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THE EFFICACY OF SYSTEMS OF DRAINAGE WELLS FOR THE RELIEF OF SUBSURFACE HYDROSTATIC PRESSURES

Technical Memorandum No. 151-1

of the

U. S. WATERWAYS EXPERIMENT STATION

Vicksburg, Mississippi

February 1, 1939

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February 1, 1939

Subject: Submittal of Report on Efficacy of Systems of Drainage Wells for the Relief of Subsurface Hydrostatic Pressures.

To: The District Engineer, U. S. Engineer Office, Memphis, Tenn. (Through the President, Mississippi River Commission, Vicksburg, Miss.)

Herewith is Experiment Station Technical Memorandum No. 151-1, which constitutes the complete report of the model studies conducted at the Experiment Station to determine the efficacy of systems of drainage wells for the relief of subsurface hydrostatic pressures. This report includes text material, table, and plates. The basic data have been retained in the files of the Experiment Station. This report supersedes all previous reports of this model study.

Paul W. Thompson
Paul W. Thompson,
1st Lieut., Corps of Engineers,
Director.

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THE EFFICACY OF SYSTEMS OF DRAINAGE WELLS FOR THE RELIEF OF SUBSURFACE HYDROSTATIC PRESSURES

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PART I: INTRODUCTION

1. Preface. This memorandum constitutes a report on the study of the efficacy of certain types of drainage systems for the relief of subsurface hydrostatic pressures. The study is concerned specifically with levees founded upon relatively thin, impervious surface formations which overlie pervious strata. The proposed levees, with which this report is concerned, are to protect certain lowland sections of the city of Memphis, Tennessee, and are to be constructed upon such a foundation medium. Protection against the formation of sand boils and piping, and the consequent danger of destruction of the levee system is essential. The use of drainage systems consisting of wells penetrating into the pervious strata in the vicinity of the landside toe of the levee have been shown by this study to be feasible. The relief of the subsurface pressure is accomplished, and the discharge quantities are determinable; so that choice of the drainage system and the necessary facilities for the disposal of the water discharged may be made for the particular conditions obtaining in the levee project.

2. History. Pertinent facts concerning the history of the study which is the subject of this technical memorandum follow:

a. The study was initiated as the result of proposals submitted to the Director of the Experiment Station by the District Engineer, Memphis, Tennessee, in a letter dated January 25, 1938, Subject: "Proposed Model Studies."

b. The Experiment Station was authorized to conduct the study by the President, Mississippi River Commission, Vicksburg, Mississippi, in the 7th indorsement dated April 1, 1938, to a letter from the Director of this Station to the District Engineer, Memphis, Tennessee.

c. The study was accomplished in the Soil Mechanics Laboratory of the U. S. Waterways Experiment Station, Vicksburg, Mississippi, during the period April 1, 1938, to October 15, 1938.

d. The final report (this technical memorandum) was submitted February 1, 1939.

3. Personnel. This model study was performed in the Soil Mechanics Laboratory of the U. S. Waterways Experiment Station, Vicksburg, Mississippi. Chief of this laboratory is Spencer J. Buchanan, Engineer, who supervised the execution of the study and edited this report. Directly in charge of the model was the project engineer, William R. Perret, Junior Physicist, who supervised the performance of all tests and prepared this report. In addition, the following members of the staff of the Soil Mechanics Laboratory assisted in the performance of the tests: Raymond A. Daumer and James P. Dealy, Under Engineering Aides.

PART II: THE PROJECT

4. The problem. A brief description of the site and the phenomena which prompted this study, as well as a discussion of a possible means for accomplishing the desired result are contained in the following subparagraphs:

a. Description of the site. The specific problem for which this study has been undertaken involves the Wolf River and Nonconnah Creek levees of the Memphis Flood Control Project. These two similar levee projects are designed to protect sections of the city of Memphis, Tennessee, from floods produced principally by back water from the Mississippi River. The foundation medium upon which it will be necessary to construct those levees includes a surface stratum of relatively impervious clayey silts and silty clays varying in thickness from about 6 ft. to about 35 ft. Underlying the impervious surface stratum and exposed at the river bank is a stratum of fine sand underlain by a stratum of sandy gravels.

b. The occurrence of sandboils and piping. Sandboils are caused by the spontaneous relief of subsurface hydrostatic pressure through localized flow of the subsurface water to the ground surface at the landside of levees during high water periods. A boil usually occurs at some local weakness in the surface stratum where the more or less impervious soil fails to withstand the hydrostatic pressure developed in the more pervious underlying formations. A small weak spot, such as a crack or fissure, permits the concentration of flow from the lower strata with consequent development of high velocities at the hole and gradual enlargement of both the hole and the discharge from it. The sand boil is believed to continue to grow and to develop flow channels within the more pervious strata by general movement of material in the boil and along the channels. The formation of the flow channels is known as piping, and is believed to result directly in the undermining of the levee. The customary corrective measure is the erection of a sand bag dam around the boil so that a sufficient hydrostatic head may be built up to restrain the flow.

c. Preventive measures considered for protection against the formation of sand boils. It is generally agreed that the control of a dangerous condition is best served by preventive rather than corrective measures. With this in view, several means for preventing the formation of sand boils have been considered. Among the proposed methods, the restriction of flow within the structure and foundation material by means of bentonite or asphaltic blankets and grouts has been shown to be helpful for certain types of materials and formations.* The relief of the subsurface hydrostatic pressure by means of a drainage system situated at the landside of the levee was considered a desirable mode of protection against the formation of sand boils along the proposed Wolf River and Nonconnah Creek levees.

5. Purpose of this study. This study was initiated for the purpose of determining the feasibility of the use of a system of drainage wells for the relief of the subsurface hydrostatic pressure. It was further desired that information concerning the most efficient form of such a drainage system should be obtained in the event that feasibility was established. The study was planned to cover only the specific case presented by the projected Wolf River and Nonconnah Creek levee systems.

6. Method of investigation. It was decided that the data required for the study of the problem described above could be derived most effectively from the study of a small-scale model of a typical portion of the levee and foundation medium directly concerned. The

*A study of the efficacy of bentonite for this purpose is reported in U. S. Waterways Experiment Station Technical Memorandum No. 135-1, dated October 8, 1938.

use of small-scale models has been sufficiently well-established as a method for studying problems of fluid flow in porous media for its choice in this study to be justified, provided certain precautions and reasonable care are taken in design. It was decided to construct a model using sands and soil, so placed as to simulate the conditions of permeability and dimensions which obtained in the prototype. The free entry of water into the pervious strata at the river bank end of the model was to be attained by the use of a pervious, copper-screen bulkhead. The simulation of an infinite landward extent of the surface stratum was to be attained by use of an impervious bulkhead, which prevented all flow from the pervious strata landward from the levee, at a distance of about eight times the foundation thickness. The systems of drainage wells were to be simulated with pyralin tubing and copper screen of appropriate dimensions. The data desired from the model included the distribution of the hydrostatic pressure beneath the relatively impervious surface stratum and the rate of discharge for the various types of drainage system. The justification for the assumptions and generalizations which it was found necessary to incorporate into the design and operation of the model are discussed in detail below.

7. Discussion of the validity and limitations of the application of a model study to the solution of the problem. The process of constructing a model for the study of a problem such as that outlined above is essentially a process of constructing a flow system geometrically similar to that of the prototype. The flow pattern may be determined by means of dye lines or by the analysis of hydrostatic pressures mea-

sured at various locations within the model. The discharge may be measured directly. The principal characteristics to be considered in the design of such models are discussed in the following subparagraphs:

a. Geometrical similitude. Reproduction of the prototype flow net requires that the model be geometrically similar to the prototype. Thus, horizontal and vertical scale ratios must be the same. It is apparent that any distortion of the geometry of the model would introduce a distortion in the flow pattern which would make interpretation of the model data difficult and uncertain.

b. Flow type similitude. In the process of fluid flow through a pervious medium, two distinct types of flow, viscous or turbulent, are possible. The existence of either type at any point is dependent upon the energy content of the physical system at that point; that is, upon the differential hydrostatic pressure in the medium and the resistance which the medium offers to flow. The factors which control the type of flow through a stratum of pervious material are thus; the hydrostatic pressure gradient along the direction of flow, and the permeability of the material. Since the characteristics of the two types of flow are very different, it is essential to the attainment of similitude between the model and prototype flow nets that the same type of flow exists in the model as in the prototype. It has been found that for nearly all cases of fluid flow through porous media occurring in nature the combination of pressure gradient and permeability is such that only viscous flow can exist. One of the exceptions to the prevalence of viscous flow is the condition, described in paragraph 4b, in which the pressure gradient in the vicinity of a sand boil becomes so large that the flow becomes turbulent. Fortunately, the reproduction of the viscous flow which is prevalent in pervious formations may be accomplished without difficulty in a model.

c. Permeability similitude. The permeability of a porous medium represents, in a rigorous sense, a physical property of the medium only. It appears in the general form of Darcy's Law for the viscous flow of fluids through porous media as a dimensionless constant factor which characterizes the permeable condition of the medium; that is, it depends only upon geometrical properties such as the voids ratio, grain size distribution, continuity of pore channels and, in cemented media, the degree of cementation. However, in the more specific form of Darcy's Law which is customarily applied to problems involving the flow of water through pervious formations and structures, the permeability factor "k" has the dimensions of a velocity and describes the condition of the medium in terms of its effect on the flow of the particular fluid, water. In connection with the reference to Darcy's Law and the definition of the effective permeability it should be noted that the application of this law is valid only when the flow is of a viscous nature. It will be apparent from the foregoing, and from a consideration of the nature of the materials applicable to the construction of a model, that the exact reproduction of some predetermined permeability in a sand model would present difficulties too great to justify such procedure. It is possible, however, to construct the model so that a good degree of uniformity of compaction, and hence permeability, is attained. The geometrical scale ratio bears no relation to the permeability scale ratio in such a model. The model discharge may be correlated to that for the prototype on the basis of the ratios of the permeabilities as measured for each structure. In cases where the prototype includes sections having different permeabilities, it is essential that the ratio of the permeabilities of the sections in the model correspond approximately to that in the prototype. Here, too, the attainment of exact correspondence of the ratios is not justifiable. An attempt to reproduce every local variation of the prototype permeability would involve impracticable exploration of the prototype and, even if these variations could be reproduced in the model, they would produce inconsequential changes in the test data. Hence, only such general differences in permeabilities as those existing between different strata or between the core and shell of a dam are reproduced. This feature is discussed further in paragraph 10a below.

d. Capillarity. The character of the prototype which was simulated with the model employed in this study precluded the consideration of capillarity in the design of the model and in the interpretation of the results. The fact that no free-surface - that is, contact between the fluid and air at atmospheric pressure - could exist within the pervious strata of the model eliminated the possibility of any capillary effect in those strata.

PART III: THE MODEL

8. Definitions of terms. The terminology adopted throughout this study has been maintained, insofar as possible, similar to that commonly applied to levee work. However, simplicity of meaning or conditions peculiar to the model study have suggested the use of certain terms whose meanings in this report are defined in the following subparagraphs:

a. Test number, run number, part number. The designation of the various types of drainage systems and conditions of tailwater were made through the agency of test, run, and part numbers. The test numbers, 1, 1-A, 1-B, etc., refer to the type of well employed in the drainage system and defined by the accessibility. The run number refers to the chronological order in which the runs were made and, where drainage systems were employed, this number designates the spacing interval between the wells in the system. The part number designates the presence or absence of drainage facilities in the model and the operation of the model with or without tailwater.

b. Accessibility. The term accessibility defines the part of the well surface through which the subsurface water has access to the well for drainage to the surface. It is expressed as the total depth to which the well penetrates, and the location and extent of the well-point or other open areas of the well.

c. Linear units. Throughout this report all linear dimensions are given in prototype feet. In the description of the well types, paragraph 10c, the actual dimensions of the model wells are also given.

d. River stage. The elevation of the water surface at the riverside of the model is defined as the river stage. It is quoted in prototype feet referred to the original ground surface, or top of the impervious stratum, as the zero datum. In this report, results for the maximum, 37-ft., river stage only are given.

e. Pressure units. The pressures which existed beneath the surface stratum were measured with piezometers and are quoted as prototype feet referred to the same datum as the river stage.

f. Maximum excess pressure. The hydrostatic pressure which exists beneath the surface stratum due to the hydrostatic head of the river stage is corrected to the same datum as the river stage. All positive values of this pressure are referred to as excess pressure to distinguish them from the actual hydrostatic pressure which is effective beneath the impervious stratum. The maximum values quoted for each test are the highest values for the excess pressure measured beneath the surface stratum to the landside of the levee.

g. Pressure relief efficiency. The degree of relief of the pressure which is accomplished by a certain drainage system has been computed as an efficiency. The minimum actual relief accomplished is the difference between river stage and the maximum excess pressure. The ratio of this minimum relief value to the river stage is the efficiency of the system in terms of the pressure relief accomplished

h. Discharge units. The discharge from the model was measured in cubic centimeters per second. All discharge values, after correction for temperature effect, were converted to gallons per hour for convenience. No correlation to the prototype was attempted with these quantities.

i. Total available discharge. In order that a convenient means of correlation between the several tests and between the test data and prototype data might be made available in the form of an efficiency, it was necessary to determine the total discharge which was available at the wells. To determine the total available discharge, the model was altered by removal of all that portion landward from the drainage system and replacement of the wells with a pervious bulkhead, similar to the one at the river bank. It was then possible, by adjusting the river stage and tailwater to correspond to the conditions maintained during the previous tests, to determine the total discharge which was available at that point.

j. Discharge efficiencies. The discharge efficiencies are computed from the test data. They express the ratio of the discharge for a particular drainage system and river stage to the total available discharge.

9. The apparatus. (Plate 1.) The apparatus used for the study reported herein is described in the following subparagraphs:

a. The flume. The model for this study was accommodated in a long steel flume located in the main hall of the Experiment Station building. Two bulkheads, constructed of bricks and covered with a 1-in. coating of neat cement, were placed across the flume to inclose a section 32 ft. long and 3.5 ft. wide. A portion of one wall of the flume, 27 ft. long and included between the end bulkheads, is made up of glass panels 2.3 ft. high. The floor of the flume was covered with a layer of neat cement to the bottom of the glass panels. The floor and bulkheads were watertight.

b. The piezometers. The type of data required of this study made necessary the determination of the distribution of the hydrostatic pressure beneath the surface stratum. For this purpose a series of 60 piezometers was installed in the steel wall of the flume. The openings for the piezometers (Plate 4, Figure 1) were flush with the inner face of the steel wall plates and on a line 0.75 ft. above the cement floor surface. The holes were spaced at intervals of approximately 0.5 ft. The piezometers consisted of 8 mm. glass tubing about 18 in. long with a right angle bend about 1 in. from one end. These glass tubes were connected by short lengths of rubber tubing to brass nipples screwed into the steel flume wall. One end of each nipple was covered with 60-mesh copper wire screen to prevent the entrance of the model material into the piezometers. A paper scale having 100 divisions per foot was fastened with rubber cement to a board behind each piezometer. The scales were zeroed by filling the flume with water to a carefully measured depth and setting each scale so that the bottom of the meniscus in its piezometer gave the proper reading for that water level. The surface tension effect for each piezometer tube was thus automatically taken into account. The piezometers were numbered from 1 to 60; No. 1 being at the riverside of the model, No. 60 at the tail bay.

c. The circulating system. In order to prevent, insofar as possible, both the evolution of dissolved air within the model and the growth of algae, distilled water was acquired for the operation of the model. For economic reasons it was desirable that this water be used repeatedly. To accomplish this, a circulating system, which included a 1200-gal. metal-lined reservoir containing a vertical sand filter, a metal-lined constant-head box containing a second vertical sand filter, a 20-gal. metal sump, an electrically-driven pump with a limit switch, and employed brass pipe and fittings throughout, was installed.

10. The model. (Plate 2, Figure 1.) The essential features of the model used for this study are described in the following subparagraphs:

a. Scale and dimensions. Study of the dimensions of the flume and the prototype showed that the most satisfactory linear scale ratio (model to prototype) would be 1 to 50. The scale of permeabilities was not definitely prescribed, but approximate values of the permeability of the two pervious substrata in the prototype were furnished by the Memphis Engineer Office.

The application of the chosen scale to the prototype structure and foundation dimensions resulted in a model having the following dimensions:

Dimension	Prototype	Model
Over-all length, feet	1270	25.4
Over-all height, feet	88	1.76
River bank to riverside toe, feet	500	10
Riverside to landside toe, feet	370	7.4
Levee to impervious bulkhead, feet	400	8
Riverside slope	1 on 3.5	1 on 3.5
Crown width, feet	10	0.2
Landside slope	1 on 5.97	1 on 5.97
Height of levee, feet	38	0.76
Length on centerline of levee, feet	175	3.5
Total foundation thickness, feet	50	1.0
Thickness of top stratum, feet	10	0.2
Thickness of middle stratum, feet	18	0.36
Thickness of bottom stratum, feet	22	0.44
Permeability:* levee and top stratum	Impervious	0.005×10^{-4}
Permeability: middle stratum	150×10^{-4}	220×10^{-4}
Permeability: lower stratum	2000×10^{-4}	3700×10^{-4}

It should be noted in connection with the permeabilities and dimensions of the prototype and model, as given above, that certain con-

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*All permeabilities are given in cm. per sec. The permeabilities quoted for the model are those determined during the construction of the model by means of the voids ratios obtained from undisturbed samples of the three strata and the predetermined voids ratio versus permeability curves. These values are not valid for all tests.

straints are placed upon the results by the values used. The permeabilities quoted for the prototype, as indicated above, are those supplied by the District Engineer, Memphis, and are understood to be approximations or general values applicable to the particular stratum represented. It is natural that variations should exist in the materials and their permeabilities in the strata of the prototype. However, such variations are assumed to be averaged to result in the general values used. Anisotropy did not exist in the model because of the careful control exercised during construction. The values of permeabilities for the prototype served principally to define the order of magnitude for the model permeabilities. It will be seen in Part V of this report that variations in the actual values of the permeability of the materials does not affect the efficacy of the drainage system, insofar as either the pressure relief or the discharge are concerned. Similarly, it will be seen that the dimensions of the several strata are effective only in their relation to the accessibility of the wells. However, it was considered desirable that the dimensions of the model be such that it was a reproduction of the generalized prototype for which the dimensions were furnished by the Engineer Office, Memphis.

b. Materials. A study of the prototype conditions and the materials available for model construction showed that a satisfactory choice could be made from the materials at hand. In making this choice, a series of tests were conducted upon samples of six types of sands and a sample of the sandy gravel which forms the lower of the three strata of the prototype. These tests were devised to furnish information concerning the flow characteristics for the material with particular reference to the establishment of the critical range within which the type of flow will change from viscous to turbulent. The results of these tests indicated that, for the conditions prevailing in both the prototype and the model, the flow would be of a viscous nature with the probable exception of the flow in the vicinity of a boil or within a relatively small region immediately surrounding the wells of the drainage system. From this information the choice of materials for the two pervious sub-strata was determined from permeability data. It was decided to simulate the lower gravel stratum with Standard Ottawa Sand which has a grain size distribution such that about 99 per cent, by weight, occurs between diameters 0.833 to 0.589 mm. The upper sand stratum was represented with a fine sand obtained locally and referred to as National Park Sand. This sand has a grain size distribution such that 95 per cent, by weight, occurs between the diameters 0.417 and 0.104 mm. The relatively impervious surface stratum and the levee were represented with Vicksburg loess.

c. Placement. The sand strata were placed under water in order to avoid the inclusion of air with the sand. The pervious riverside bulkhead and the impervious landside bulkhead were placed in the flume with their top edges at the elevation to which the surface of the impervious stratum was to be molded. Lines were drawn on the glass panels by means of carefully regulated water depths to show the limits of each stratum and the location of the levee section. An excess of Standard Ottawa Sand was then placed under water in thin layers and rodded to insure both freedom from trapped air and uniformity of compaction. The surplus sand was then removed so that the top surface of the lower stratum of the model coincided with the guide line drawn on the glass panel. The upper sand stratum was laid in a similar manner except that in this case it was necessary to have the fine National Park Sand thoroughly wet before introducing it into the water in the flume. It was found desirable to place this sand in thin layers through a thin layer of water in order that the stratification due to non-uniform settlement of the grains might be held at a minimum. This sand stratum was rodded thoroughly to destroy any stratification which may have occurred during placement. As in placing the first stratum, an excess amount of the fine sand was placed and compacted and the surplus was removed. The construction of former models has shown that the most desirable procedure for conditioning loess to simulate an impervious material is to maintain the water content at about 1 or 2 per cent below the plastic limit. In placing the surface stratum and levee, the water content of all the material was carefully maintained at the proper value. The material was mixed to the proper consistency, broken up through a 1/4-in. screen, and compacted by hand in thin layers. Tamping was found to distort the lower pervious strata and so could not be resorted to for compaction. Special care was necessary to insure that a water-tight bond existed between the surface stratum and both the flume walls and the impervious landside bulkhead.

d. Permeabilities. Undisturbed samples were taken from each stratum at several locations upon completion of the molding of that stratum. The voids ratios were determined for each of these samples and the weight of a unit volume of the surface stratum was computed. These data permitted the determination of the approximate permeabilities for the three strata. The permeability of the lower, most pervious strata (Standard Ottawa Sand) was found to be approximately 3700 x 10-4 cm. per sec.; that of the upper stratum, to be approximately 220 x 10-4 cm. per sec. The ratio of these permeabilities is about 1 to 17 which agreed sufficiently well with the given prototype ratio of 1 to 13. The permeability of the relatively impervious surface stratum was found to be about 0.005×10^{-4} cm. per sec. Weight of a unit volume of the surface stratum was found to be about twice that of a unit volume of water. This latter quantity is of value in determining safe excess pressures.

e. The drainage system. The drainage systems tested consisted of a series of wells spaced at 20-ft. intervals along a line parallel to the centerline of the levee and 10 ft. to the landside of the landside toe. (Plate 2, Figure 2.) These wells were of two principal types and penetrated through the impervious top stratum to various depths within the pervious substrata. Both types of drainage wells tested were so designed that the displacement of the foundation material by the flow of water was prevented. Hence, the development of piping as the result of excessive velocities through the porous strata could not occur. Tests were also made to determine the effects of landside borrow pits which penetrated to the pervious strata at certain distances from the levee toe. The two principal types of wells used in the drainage system, as well as the borrow pits, are described in the following subparagraphs:

(1) Original drainage wells. The drainage wells originally tested were planned to simulate pipes 1 ft. in diameter driven to a depth of 6 ft. below the base of the impervious surface stratum and back-filled with gravel. The wells were to project 2 ft. above the ground surface. The model wells consisted of a piece of pyralin tubing 0.5 in. I. D. and 0.36 ft. long. A piece of 60-mesh bronze screen was cemented across the bottom of these wells which penetrated 0.12 ft. below the base of the impervious surface stratum.

(2) Well-point drainage wells. A second type of well, radically different from that originally considered, was tested after it was found that the first type was inadequate. These new wells incorporated well-points and were designed to give a high degree of flexibility in testing different degrees of accessibility. They consisted of pyralin tubing 0.5 in. I. D., 0.22 ft. long which projected 0.02 ft. above the ground surface and extended completely through the impervious surface stratum. A tube of 60-mesh copper screen 0.5 in. diameter and 0.36 ft. long was cemented to the bottom of the pyralin tubing. A second tube of 30-mesh copper screen, 0.5 in. diameter and 0.40 ft. long with a bottom of 30-mesh screen was soldered to the 60-mesh screen. The joint between the two screen sections was enlarged so that a brass tube of proper diameter and length could be placed on the upper section of the

well and seated so as effectively to case-off the upper sand stratum. This type (Plate 3, Figure 3) represented a well of 1 ft. diameter penetrating 48 ft. below the ground surface. The lower 38 ft. consisted of a well point so arranged that it could be tested in sections. Thus (Plate 3, Figure 2) with the casing in place, only the lower 20 ft. of the well point was accessible and only the lower sand stratum was drained. Furthermore, by introducing a suitable quantity of sand into the well, the accessible section of the well point could be reduced so that the well penetrated only 10 ft. into the lower sand stratum and drained only the upper 10 ft. of that stratum. Removal of the casing and variation of the quantity of sand introduced into the well made possible the representation of other degrees of accessibility.

(3) Borrow pits. (Plate 2, Figure 3.) Several tests were conducted to determine the effect of landside borrow pits upon the distribution of the excess pressure beneath the surface stratum. The borrow pits were simulated by the removal of the surface stratum at the landside of the model. Three locations of the borrow pit were tested, representing pits which penetrated to the upper pervious stratum at distances of 300 ft., 100 ft., and 10 ft. from the landside toe of the levee and extended, in each case, to the impervious bulkhead.

PART IV: THE TESTS

11. Procedure. The procedure for conducting all the tests for this model study was devised and employed. The essential features of this procedure are described in the following subparagraphs:

a. Regulation of river stage. The river stage was regulated and maintained by means of an over-flow drain at the riverside of the model. The water-surface elevation could be maintained with an error of less than 0.002 ft. or 0.1 prototype ft. In all tests, it was desired to determine the behavior of the model for a 37-ft. river stage; that is, one foot freeboard. However, all tests were conducted so as to include intermediate stages of 10 ft., 20 ft., and 30 ft. in order that more complete information might be obtained. In some of the more critical tests, where it was believed that the model might be seriously damaged at the higher stages, intermediate steps of 5 ft., and in several cases 2.5 ft., were employed to prevent such damage.

b. Pressure observations. As stated in paragraph 9b above, a series of piezometers was employed to indicate the distribution of the hydrostatic pressure beneath the impervious surface stratum. These piezometers were situated 0.05 ft. below the base of the surface stratum. The standard procedure adopted for all tests consisted of reading all even-numbered piezometers 15 minutes after the establishment of a river stage and repeating the observation at 30-minute intervals until the individual readings indicated that an equilibrium condition had been established. In general, only three observations were necessary, and the last observation was considered to represent the pressure distribution for the particular river stage and drainage system. From time to time, the observations were extended over much longer periods to verify the establishment of equilibrium. In order that variations in the pressure distribution in the vicinity of the drainage system might be more accurately observed, both even- and odd-numbered piezometers were read in that region of the model.

c. Discharge measurements. The water level in the tail bay of the model was maintained constant by means of an over-flow drain. For most tests, it was maintained at approximately the ground-surface elevation. The water which was discharged through the drainage system flowed out through the over-flow drain into the collecting sump. This discharge was measured before and after each set of pressure observations, and equilibrium was considered to be established only when the discharge rate was found to be constant or very nearly so. The discharge rate was measured as the volume discharged per unit time for the entire system.

d. Adjustment of the well spacing and accessibility. The drainage systems tested in this model study differed in the distance between the wells and in the accessibility of the wells. A total of nine wells were used in the model at a minimum interval of 20 ft., center-to-center. The other intervals tested were 40 ft., 60 ft., 80 ft., and 175 ft. The increased spacing was accomplished by plugging the tops of appropriate wells. The plugs used for this purpose consisted of corks through which glass tubes, open at each end, projected. These tubes served as piezometers for checking, approximately, the hydrostatic pressure at the wells. The method adopted for varying the accessibility of the wells was described in paragraph 10c(2) above. The spacing intervals and accessibilities tested are presented in Table I.

e. Temperature corrections. The viscosity of water and hence its rate of viscous flow varies considerably with temperature. The temperature of the water in the model was measured at the riverside bulkhead and at the wells for each discharge observation. It was found to vary widely from test to test, and in many cases during the operation of a single test. Therefore, it was deemed desirable to correct all discharge rates to

20° C. by means of a table computed from Table 170 of the "Smithsonian Physical Tables," 8th edition. The temperatures used for these corrections were the mean of the entrance and exit temperatures.

f. Piezometer scale corrections. During the period throughout which this study was conducted, the atmospheric temperature and humidity varied through wide ranges. These changes, especially the humidity variations, caused the paper scales which were employed for reading the piezometers to expand and wrinkle in such a manner that some of the observations were affected. Correction curves for each piezometer were compiled, and pressure data were corrected wherever the scale error was greater than the experimental error.

12. The test data. The data obtained from the tests conducted for this model study consist of the hydrostatic pressure distribution and the rates of discharge. These data are presented and described in the following subparagraphs:

a. The data. All the data relevant to this model study are contained in Table I, page 24. These data have been selected and arranged so that the characteristics of each type of drainage system tested are readily comparable with those of all the other systems. Since it is obvious that any drainage system which will accomplish the safe relief of the excess pressure at the maximum river stage will be safe for lower stages, no data for the lower stages are quoted in Table I. Similarly, the discharge measurements for the lower stages were considered to be irrelevant, since the design of any pumping or storage system for the accommodation of the discharge will necessarily be based upon maximum conditions. A series of typical pressure distribution graphs have been assembled in Plate 4. Figure 1 of this plate represents a sectional view of the model showing the location of the various important parts. Figure 2 represents the pressure distribution for river stages corresponding to 10 ft., 20 ft., 30 ft., and 37 ft., when the drainage system in operation consists of wells spaced at 20-ft. intervals and having a total depth of 48 ft. of which the lower 20 ft. is accessible to the subsurface water. Figure 3 illustrates the effect of the variation of the spacing interval between wells upon the pressure distribution. The tests were conducted using intervals of 20 ft., 40 ft., 60 ft., and 80 ft. between the wells. The wells were similar to those described for Figure 2. Figure 4 illustrates the effect of changes in the accessibility of the wells upon the pressure distribution. The tests from which these data were obtained involved; (1) the use of wells 48 ft. deep which were accessible throughout their lower 38 ft. and 20 ft., respectively, (2) the use of wells 38 ft. deep and accessible throughout their lower 10 ft.; and (3) the use of wells 28 ft. deep and accessible throughout their lower 18 ft. The wells were spaced at 20-ft. intervals in these tests. Figure 5 represents the pressure distribution which occurs for three locations of the edge of the landside borrow pit. Borrow pits which penetrated through only the impervious surface layer at distances of 300 ft., 100 ft., and 10 ft. from the landside toe of the levee and extended in each case to the impervious landside bulkhead were tested.

b. Accuracy of the data. The degree of accuracy attained in the measurements which produce certain data must govern to a considerable extent the interpretation of those data. In the model study which is the subject of this report, the accuracy of all the measurements was maintained well within the limits prescribed by the nature of the model materials, the scale, and the operating conditions. The determination of the hydrostatic pressure involved estimation to the nearest 0.001 ft. on a 100-divisions-per-foot scale. That this procedure was allowable was shown by the close correspondence of repeated and independent observations by several of the operators. This degree of accuracy was not retained, however. The piezometer readings were converted to the nearest 0.005 ft. in order that they might be more nearly of the same order of accuracy as the model prescribed. The discharge measurements, involving a volume of water and a time interval, were such that the probable error in both measurements was of the same order of magnitude, and always less than one per cent of the whole quantity measured. All measurements were repeated as stated in paragraphs 11b and 11c above. All pressure measurements which were considered stable varied less than 0.003 ft. Discharge measurements were checked to 0.03 cc. per sec. in quantities not less than 7 cc. per sec. and as high as 17 cc. per sec. It is evident that the accuracy of the measurements is at least as good as that attained in the construction of the model and quite sufficient for the purpose of this study.

TABLE I

EFFICACY OF DRAINAGE SYSTEM
FOR THE
RELIEF OF SUBSURFACE HYDROSTATIC PRESSURES

TEST NO.	RUN NO.	PART NO.	DRAINAGE SYSTEM WELL SPACING	DRAINAGE SYSTEM ACCESSIBILITY	MAXIMUM RIVER STAGE	MAXIMUM EXCESS PRESSURE	PRESSURE RELIEF EFFICIENCY	CORRECTED MODEL DISCHARGE		DISCHARGE EFFICIENCY
			FT.	FT.	FT.	FT.	%	CC/SEC	GAL/HR	%
1	1	1	NO DRAINAGE SYSTEM		10.0	9.0	--	--	--	--
1	2	1			7.5	7.5	00.0			
1	2	1			10.0	9.0	--	--	--	--
1	1	2			*22.5	7.5	00.0	--	--	--
1	1	3	20	TOTAL DEPTH	15.0	--	--	--	--	--
1	3	3	20	16 FT.	10.0	9.5	--	VERY SMALL		@ 1%
1	3	4	20	OPEN:BOTTOM	*25.0	9.8	--	--	--	--
				ONLY						
1-A	10	3	20	TOTAL DEPTH	37.0	4.0	89.1	14.29	13.61	98.4
1-A	8	3	40	48 FT.	37.0	4.3	88.4	14.00	13.31	96.4
1-A	9	3	60	OPEN:LOWER	37.0	5.7	84.6	13.22	12.57	91.8
1-A	11	3	80	20 FT.&BOTTOM	37.0	6.3	83.0	13.21	12.57	91.0
1-B	1	3	20	TOTAL DEPTH	37.0	4.5	87.9	14.11	13.39	97.2
1-B	2	3	40	38 FT.	37.0	5.4	85.5	13.42	12.79	92.5
1-B	3	3	60	OPEN:LOWER	37.0	7.2	80.5	12.63	12.00	87.0
1-B	4	3	80	10 FT.& BOTTOM	37.0	8.5	77.0	12.28	11.67	84.5
1-C	1	3	20	TOTAL DEPTH	37.0	6.5	82.5	12.52	11.89	86.3
1-C	2	3	40	28 FT.	37.0	8.2	77.8	11.84	11.25	81.5
1-C	3	3	60	OPEN:LOWER	30.0	9.9	63.7	8.33	7.93	**
1-C	4	3	80	18 FT.& BOTTOM	25.0	9.9	56.4	6.23	5.91	**
1-D	1	3	20	TOTAL DEPTH	37.0	2.5	93.2	14.27	13.56	98.3
1-D	2	3	40	48 FT.	37.0	3.0	91.9	14.21	13.50	97.9
1-D	3	3	60	OPEN:LOWER	37.0	4.3	88.4	13.97	13.27	96.1
1-D	4	3	80	38 FT.& BOTTOM	37.0	5.0	86.5	13.68	13.02	94.1
1-D	5	3	175		37.0	7.6	79.4	12.40	11.82	85.4
1-E	1	3	20	TOTAL DEPTH	37.0	3.5	90.5	14.00	13.31	96.4
1-E	2	3	40	38 FT.	37.0	4.1	88.9	13.70	13.02	94.4
1-E	3	3	60	OPEN:LOWER	37.0	5.8	84.3	13.15	12.49	90.6
1-E	4	3	80	28 FT.& BOTTOM	37.0	6.3	83.0	13.06	12.42	89.9
1-D	1	4	20	TOTAL DEPTH	37.0	1.0	95.9	9.54	9.45	**
1-D	2	4	40	43 FT.	37.0	1.3	94.5	9.37	8.90	**
1-D	3	4	60	OPEN:LOWER	37.0	1.9	92.0	9.25	8.83	**
1-D	4	4	80	38 FT.& BOTTOM	37.0	2.7	88.7	9.10	8.68	**
1-E	1	4	20	TOTAL DEPTH	37.0	1.7	92.9	9.36	8.87	**
1-E	2	4	40	38 FT.	37.0	2.0	91.7	9.26	8.83	**
1-E	3	4	60	OPEN:LOWER	37.0	3.0	87.5	8.85	8.38	**
1-E	4	4	80	28 FT.& BOTTOM	37.0	3.8	84.2	8.73	8.30	**
1-C	1	4	20	TOTAL DEPTH	37.0	3.7	84.6	8.53	8.08	**
1-C	2	4	40	28 FT.	37.0	4.5	81.2	8.05	7.63	**
1-C	3	4	60	OPEN:LOWER	37.0	7.6	68.4	6.91	6.58	**
1-C	4	4	80	18 FT.& BOTTOM	37.0	8.0	66.6	6.83	6.51	**
2	1	1	20	TOTAL DEPTH	37.0	2.9	92.2	14.09	13.39	97.0
			40	48 FT.	37.0	3.3	90.3	13.90	13.24	95.8
BORROW PIT			60	OPEN:LOWER	37.0	3.8	89.8	13.52	12.87	93.2
OPEN:300 FT. TO			80	38 FT.& BOTTOM	37.0	4.1	88.9	13.51	12.87	93.2
400 FT.FROM			175		37.0	6.0	83.8	12.82	12.19	88.4
LANDSIDE TOE			NO DRAINAGE SYSTEM		37.0	10.3	72.2	10.15	9.65	69.9
2	1	2	20	TOTAL DEPTH	37.0	3.0	91.9	14.20	13.46	97.8
BORROW PIT			40	48 FT.	37.0	3.2	91.4	13.97	13.27	96.1
OPEN:100 FT. TO			60	OPEN:LOWER	37.0	3.5	90.5	13.79	13.09	94.9
400 FT. FROM			80	38 FT.& BOTTOM	37.0	3.9	89.5	13.62	12.94	93.8
LANDSIDE TOE			175		37.0	4.7	87.3	13.10	12.49	90.2
			NO DRAINAGE SYSTEM		37.0	6.6	82.2	11.86	11.29	81.7
2	1	3	BORROW PIT:OPEN 10 FT. TO 400 FT. FROM LAND-SIDE TOE		37.0	4.6	87.5	13.13	12.47	90.5

1. DIMENSIONS IN PROTOTYPE FEET
2. PART NO. 4 OF ALL TESTS WAS CONDUCTED WITH 13.0 FEET TAILWATER
3. DISCHARGE EFFICIENCIES CALCULATED ON BASIS OF MEASURED TOTAL AVAILABLE DISCHARGE AT CENTERLINE OF DRAINAGE SYSTEM; Q = 14.51 CC PER SEC.

* TESTS CONDUCTED WITH 15.0 FEET TAILWATER

** TESTS FOR WHICH NO DISCHARGE EFFICIENCY WAS COMPUTED BECAUSE OF LOW MAXIMUM RIVER STAGES OR PRESENCE OF TAILWATER

PART V: THE RESULTS

13. Discussion of the results. The data obtained from the tests described in Part IV of this report are applicable to the design of an adequate drainage system for the relief of the subsurface hydrostatic pressure at the site of the Wolf River and Nonconnah Creek levee project. These results are discussed in the following subparagraphs with respect to their significance and limitations. Reference is made throughout this discussion to Table I and to Plate 4.

a. Effectiveness of drainage system. The preliminary tests, which were operated without drainage facilities, served to verify the fact that the pressure which exists under the impervious surface stratum in this case is purely a hydrostatic pressure induced throughout the pervious substrata by the river stage and equivalent to it. The pressure beneath the surface stratum was found to reach a steady state within a short time after the establishment of a river stage and to be uniform throughout the length of the model. The data for Part No. 1 of Test No. 1, quoted in Table I, shows that for both Run No. 1 and Run No. 2 the establishment of a 10.0-ft. river stage resulted in the rupture of the surface stratum to the landside of the levee. In both cases the maximum excess pressure recorded beneath the surface stratum at the landside of the levee was less than that expected from the existing river stage. However, equilibrium had not been established and the rupture of the impervious stratum prevented its establishment. The data quoted for the 7.5-ft. river stage for this test, as well as that for the 5.0-ft. river stage (not quoted), indicate the equivalence between the river stage and excess pressure. In the latter cases, the pressure was uniform throughout the entire length of the model. It is of interest to note here that the specific gravity of the saturated surface stratum was found to be approximately 2.03. Hence, it would be expected that an excess pressure of about 10 ft. would very nearly counterbalance the weight of the surface stratum and cause failure by rupture at any weak or thin spot in the stratum.

The drainage system tested in Part No. 3 of Test No. 1 consisted of wells constructed according to the original plans, described in paragraph 10c(1) above, and spaced at intervals of 20 ft. along the center line of the system. Run No. 1 of this test resulted in failure by rupture of the surface stratum in the vicinity of the wells upon establishment of a 15.0-ft. river stage. However, Run No. 3 of this test, for which the maximum river stage was 10.0 ft., showed a pressure relief of 0.5 ft. While these results showed definitely that the drainage system being tested was inadequate for the accomplishment of safety against sand boils, they did indicate that relief of subsurface pressure could be attained by this means. Consideration of the foundation medium and the wells which were tested led to changes in the design of the wells. It is evident from consideration of the permeabilities and dimensions of the two pervious strata that the upper, less pervious of these can carry only a small portion of the total flow through the model. This portion was estimated to be about 5 per cent. The wells tested penetrated to only 1/3 of the total thickness of this stratum and were open at the bottom only, so that the area through which the water had access to them was small. It was estimated that only about 1 per cent of the total quantity of water available at the wells was discharged from them. It was apparent that the accomplishment of satisfactory relief of the subsurface pressure would require the use of wells which were accessible to a considerably larger portion of the subsurface water. The improved type of drainage well described in paragraph 10e(2) above, resulted from the foregoing considerations and was incorporated into the model in all further tests. The flexibility of the new wells permitted tests to be conducted with numerous degrees of accessibility. The specifications for, and the results of, the tests conducted with these wells are given in Table I. It will be noted that for Test No. 1-A, the pressure relief efficiency varies from 98.4 to 91.0 per cent as the spacing interval is increased from 20 ft. to 80 ft. These two efficiencies vary through somewhat similar ranges for the other tests. Test No. 1-C illustrates the effect of insufficient drainage facilities for both the 60-ft. and the 80-ft. spacing intervals; maximum river stages of only 30.0 ft. and 25.0 ft. respectively, were allowable in these cases. Part No. 4 of these tests, as well as Part No. 2 of Test No. 1, which were operated with tailwater covering the landside of the model to a depth of 13.0 ft. (15.0 ft. in Test No. 1), showed that the tailwater reduced the effective hydrostatic head of the river stage by an amount equal to its stage height. Pressure relief and discharge efficiencies were similar to those expected for the same effective hydrostatic head in the absence of tailwater. The effect of the borrow pits was similar to that of the drainage systems, except that as the borrow pit was located at successively greater distances from the levee greater excess pressures existed beneath the surface stratum in the vicinity of the landside of the levee. This is best shown in Figure 5 of Plate 4 in which the pressure distribution curves for the three borrow pit locations are presented. The use of a drainage system in conjunction with a borrow pit produced a pressure relief similar to that already discussed. It should be mentioned that the flow within the borrow pit was distributed over the entire area so that no boiling or piping occurred there. This is believed to be due to the exposure of a large area of the less pervious stratum which prevented concentration of the flow.

b. Significance of the pressure relief and discharge efficiencies. The efficiencies derived from the results of this model study furnish primarily a means for comparing the several tests. The extent to which they can be carried over, unaltered, to the prototype is not known. Probably, the use of these efficiencies in designing a drainage system should not be strictly quantitative. Certainly, it is safe to use them qualitatively. The pressure relief efficiency, alone, defines the real effectiveness of the system in accomplishing safety against the production of sand boils. The discharge efficiency serves the purely economic function of aiding in the design of the means for disposing of the discharged water. The advantage of these two efficiencies lies in the fact that they are dimensionless and thus may be used without regard to scale ratio or the dimensional units involved and their utility will depend only upon the extent to which true similitude between model and prototype has been attained.

c. Application of the results of this study to a specific problem. For the purpose of demonstrating the processes involved in the application of the results of this study to a practical problem, the two cases discussed below have been chosen. The process must necessarily appear to be quantitative. However, the results should be regarded as indicating the minimum specifications for the drainage system rather than the optimum system. It is assumed that at some site all the dimensions, slopes and permeabilities for a levee system and foundation exist as described in paragraph 10a above. It will be further assumed that two distinct cases exist, one in which the surface stratum has a thickness of 8 ft., and the other in which that thickness is 18 ft.

(1) Case I. It is believed that safety against the formation of sand boils may be accomplished if the excess pressure at the landside is not permitted to exceed one-half the thickness of the surface stratum. A factor of safety is thus provided in view of the data obtained from Test No. 1, Run No. 2, Part No. 1. Thus, the maximum allowable excess pressure for a surface stratum 8 ft. thick would be 4 ft. Since the maximum river stage, allowing 1 ft. freeboard on the levee, is 37 ft., the requisite pressure relief is 33 ft. It is, therefore, necessary that a drainage system having a pressure relief efficiency of 89 per cent be installed. Perusal of Table I shows that several of the systems tested have efficiencies of 89 per cent or higher. Of these systems, all but one,

Test No. 1-A, consist of wells having exceptionally long well-points. Since that part of the well will be costly, the well type used in Test No. 1-A will be considered. The drainage system thus planned would consist of wells 48 ft. deep having 20 ft. of well-point at the bottom and spaced along the centerline of the drainage system at 20-ft. intervals. Each well would drain from a strip of the pervious strata 20 ft. wide extending to the river bank. The total available discharge would be 3460 gal. per hr. per well and since the discharge efficiency for this system was found to be 98.4 per cent, it follows that the discharge to be expected for the system would be 3400 gal. per hr. per well or 17,000 gal. per hr. per 100 ft. of levee system. It should be noted that this discharge is for the maximum river stage only; for lower stages it would decrease. An alternative drainage system consisting of wells only 38 ft. deep spaced 20 ft. apart and having only 10-ft. well-points, might be adopted since such a system, Test No. 1-B, was found to have a pressure relief efficiency of 87.9 per cent which would involve an excess subsurface pressure of only about 4.5 ft.; probably safe for the given 8 ft. stratum. This system has a discharge efficiency slightly lower than that considered above.

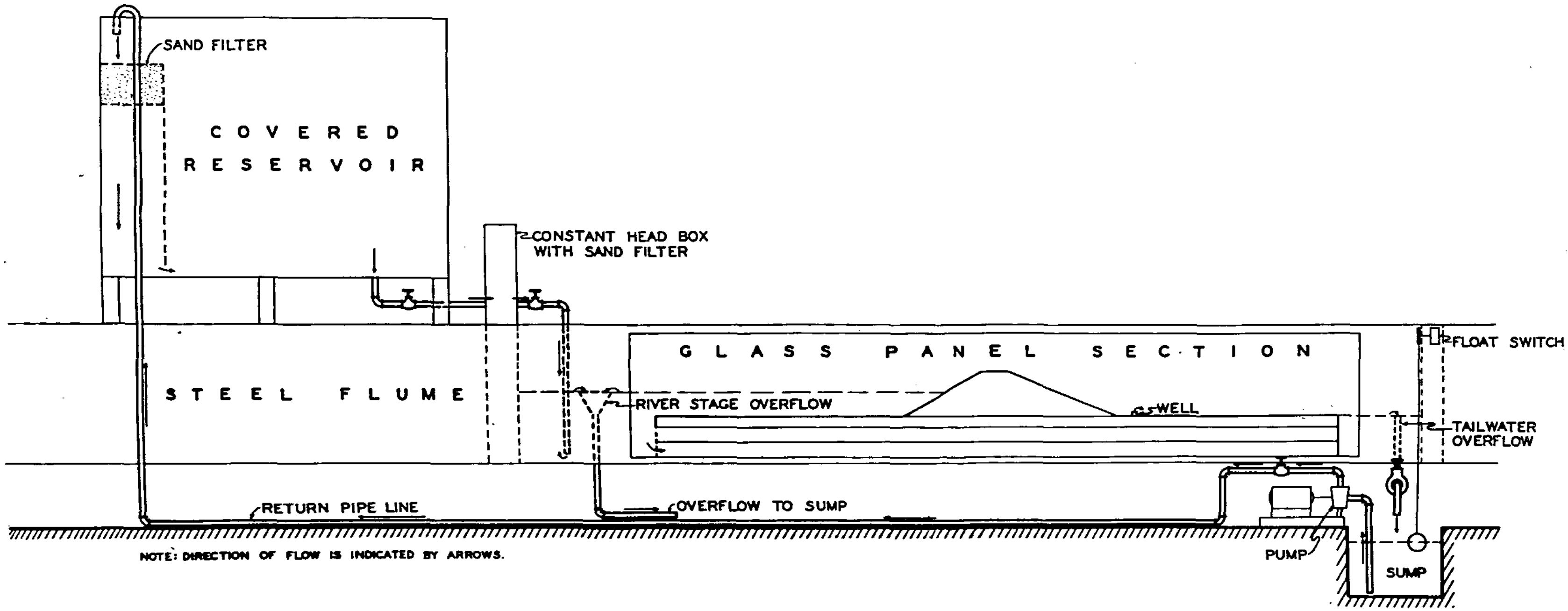
(2) Case II. In this case, the surface stratum is considered to be 18 ft. thick. The maximum allowable excess pressure is 9 ft. and the necessary relief is 28 ft. The accomplishment of this pressure relief requires that the drainage system have a pressure relief efficiency of 75.7 per cent. Among the systems of drainage wells tested in this study, the most satisfactory for the accomplishment of the necessary pressure relief is that used

in Test No. 1-B, Run No. 4. This drainage system consists of wells 38 ft. deep having a 10-ft. well-point and spaced at 80-ft. intervals. It has a pressure relief efficiency of 77.0 per cent and a discharge efficiency of 84.5 per cent. If we assume no alteration in the dimensions of the pervious strata at this location, the expected discharge per well will be about 11,700 gal. per hr. per well or 14,600 gal. per hr. per 100 ft. of levee system.

14. Conclusions. The results of the experiments conducted during this study have shown that the use of a drainage system consisting of wells for the relief of subsurface hydrostatic pressure is feasible. They have also shown that, for the particular problem involved in this study, a practical, economical solution may be effected with any one of several drainage systems. It is believed that a further, more general program of study along lines similar to those developed during the course of this study, would permit the treatment of other types of subsurface formations in a manner similar to that adopted in this report.

END OF MEMORANDUM PROPER

(Plates follow)



EFFICACY OF SUB-SURFACE DRAINAGE SYSTEMS

SCHEMATIC DIAGRAM OF FLUME AND CIRCULATING SYSTEM

U.S. WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

SUBMITTED:	RECOMMENDED:	APPROVED:
JR. PHYSICIST	ENGINEER	1ST. LIEUTENANT C.E.

DRAWN BY J.D.G. CHECKED BY W.R.P.	TO ACCOMPANY REPORT DATED: FEB. 1, 1939	FILE NO. 195/151.1/51

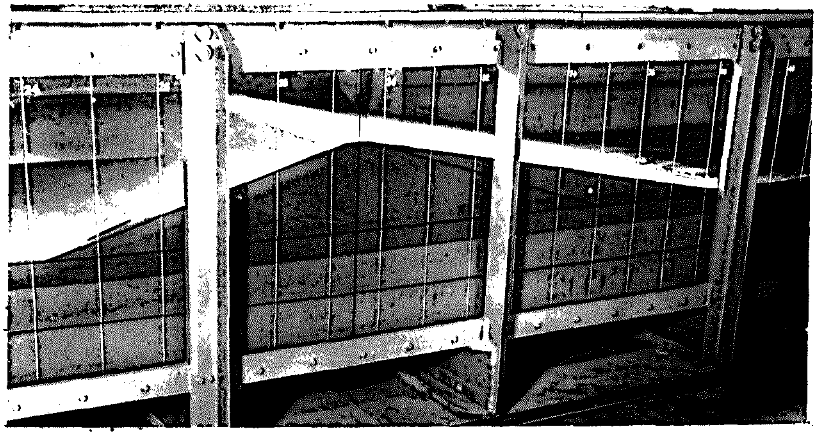


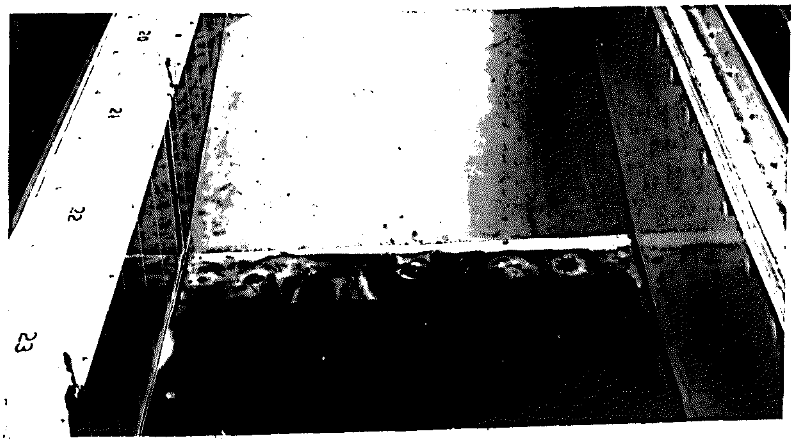
FIGURE 1

GENERAL VIEW

Foundation and levee section at maximum river stage. Vertical lines on glass panels show location of piezometers.

FIGURE 2

DISCHARGE FROM WELLS



Wells spaced at 20-ft intervals. Dye introduced into wells.

FIGURE 3

EDGE OF BORROW PIT

Location with respect to wells and levee toe. Note standpipe plugs in wells.

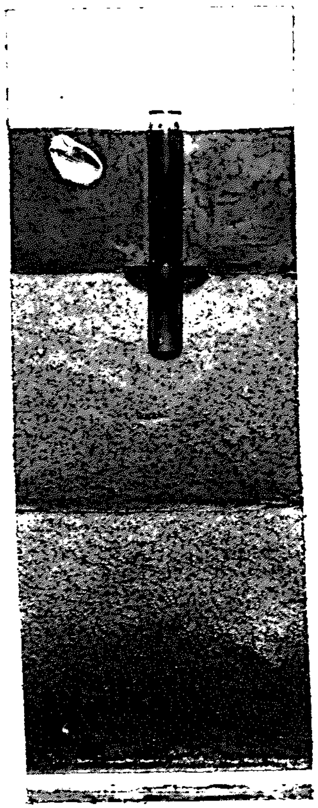


FIGURE 4

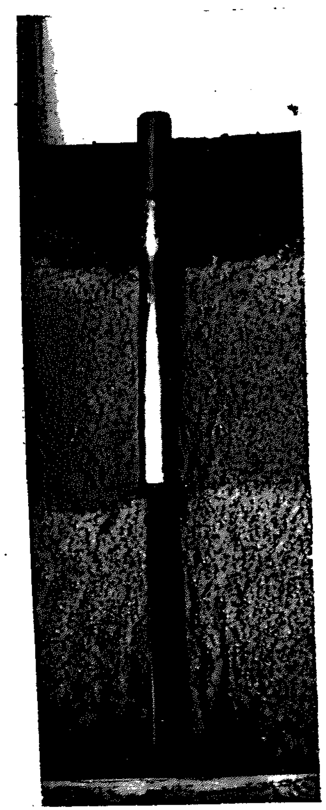


FIGURE 5

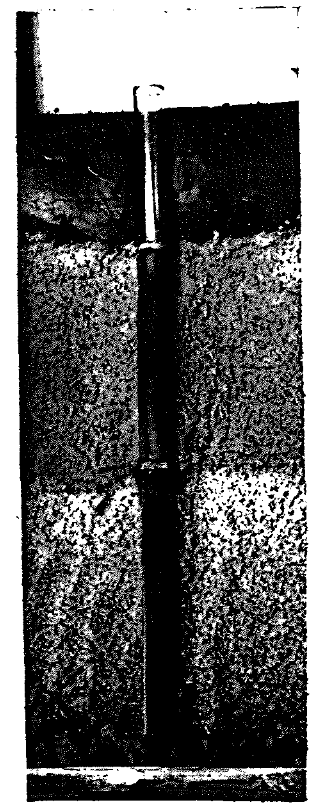


FIGURE 6

FIGURE 4. Original Well Type. Total Depth: 16 ft. Accessible at bottom only.

FIGURE 5. Well-point Type. Total Depth: 48 ft. Accessible: Lower 20 ft. Note casing.

FIGURE 6. Well-point Type. Total Depth: 48 ft. Accessible: Lower 38 ft. Note two sizes of screen.

PLATE 3

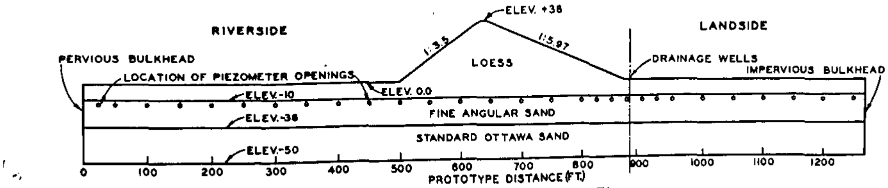


FIG. 1 ELEVATION OF MODEL

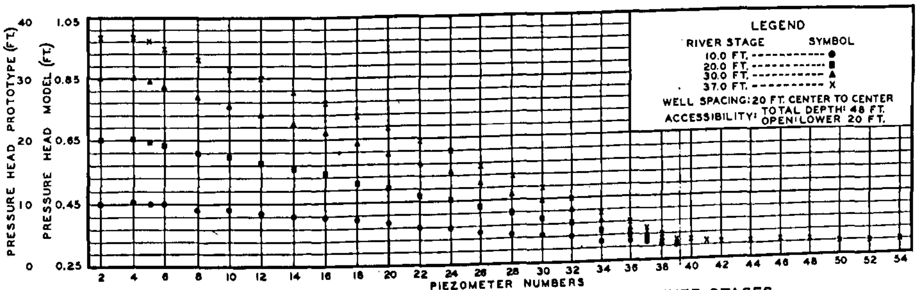


FIG. 2. PRESSURE DISTRIBUTION FOR VARIOUS RIVER STAGES

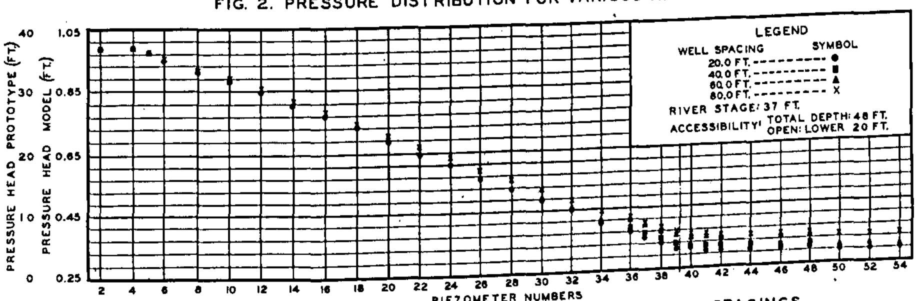


FIG. 3 PRESSURE DISTRIBUTION FOR VARIOUS WELL SPACINGS

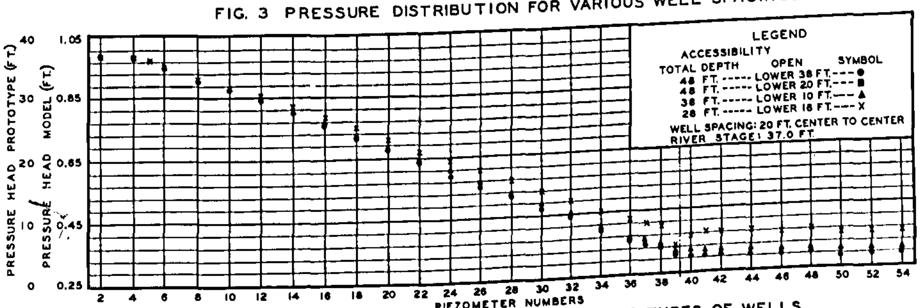


FIG. 4. PRESSURE DISTRIBUTION FOR VARIOUS TYPES OF WELLS

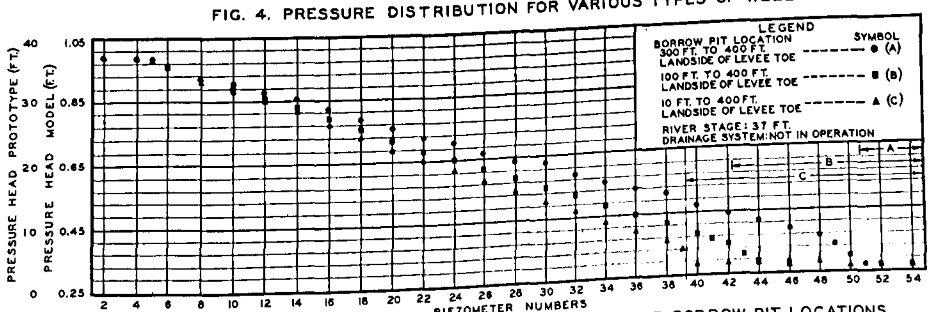


FIG. 5. PRESSURE DISTRIBUTION FOR VARIOUS LANDSIDE BORROW PIT LOCATIONS

NOTE:

PIEZOMETER NO. 2 DENOTES THE WATER SURFACE ELEVATION AT EACH RIVER STAGE.

IN CASES OF COINCIDENCE OF DATA, ONLY THE UPPER AND LOWER EXTREME VALUES ARE INDICATED.

EFFICACY OF SUB-SURFACE DRAINAGE SYSTEMS

DISTRIBUTION OF HYDROSTATIC PRESSURES

U. S. WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

SUBMITTED:	RECOMMENDED:	APPROVED:
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DRAWN BY C.H.L. CHECKED BY W.R.P.	TO ACCOMPANY REPORT DATED: FEB 1, 1939.	FILE NO. 195/151.1/52