64-18

GEOLOGICAL SURVEY OF CANADA

DEPARTMENT OF MINES AND TECHNICAL SURVEYS

PAPER 64-18

GEOTECHNICAL INVESTIGATIONS RED RIVER FLOODWAY, WINNIPEG, MANITOBA

(Report, 1 plate, 18 figures)

G. D. Hobson, J. S. Scott, and R. O. van Everdingen

Price, 75 cents

1964

GEOLOGICAL SURVEY

OF CANADA

PAPER 64-18

GEOTECHNICAL INVESTIGATIONS, RED RIVER FLOODWAY, WINNIPEG, MANITOBA

Part I. Hammer Seismic Survey

by George D. Hobson

Part II. Groundwater Investigations

by J.S. Scott and R.O. van Everdingen

DEPARTMENT OF

MINES AND TECHNICAL SURVEYS

CANADA

CONTENTS

Illustrations

ABSTRACT

During preliminary engineering studies of the Red River Floodway — a channel to divert floodwater from Red River around the city of Winnipeg, Manitoba — groundwater flows were encountered when a test pit was excavated in the Floodway right-of-way. The groundwater flow was being derived by upward leakage from an artesian aquifer in fractured Ordovician limestone underlying the till and lacustrine clay in which the test pit was being excavated.

Potential groundwater seepage could thus be anticipated in areas where the base of the Floodway channel comes close to the bedrock surface. A hammer seismic-refraction survey was made along the centre line of the Floodway to extend existing bore-hole information and to provide a continuous profile of the bedrock surface and of the interfaces of Pleistocene deposits overlying bedrock. The average error in determining depth to bedrock by the hammer seismic method was plus or minus 7.3 per cent of depths determined from bore-hole data.

The effect of construction of the Floodway on the regional piezometric surface of the bedrock aquifer was simulated by the construction of an electric-analog model of the continuous-solid-conductor type, using Teledeltos paper. The model was set up as a secondary flow-field or drawdown type. The type of model used gives only eventual equilibrium drawdown values, regardless of time. A time factor, corresponding to equilibrium conditions, was derived by use of a constant-head drain formula. Theoretical time- and distance-drawdown curves calculated with the formula were compared with distance-drawdown curves obtained from the model.

Total groundwater discharge along the length of the Floodway channel was predicted to be 1044 cfs immediately following construction, decreasing to 57 cfs as piezometric equilibrium is reached. These predictions are based on extrapolated values of seepage quantities entering the test pit and on a drain-discharge formula.

GEOTECHNICAL INVESTIGATIONS, RED RIVER FLOODWAY, WINNIPEG, MANITOBA

INTRODUCTION

The city of Winnipeg, Manitoba, situated at the confluence of the Red and Assiniboine Rivers, has experienced flood damage at various times throughout its history. In the spring of 1950 a major flood of the Red River caused such severe damage that an investigation was undertaken by the Federal Government to determine the possibilities of alleviating the flood problem. The major result of the investigation was the decision to construct a diversion channel that would divert Red River floodwater around the Greater Winnipeg area. Additional studies regarding the location of the diversion channel were begun in 1959 by the Water Control and Conservation Branch of the Manitoba Department of Agriculture and Conservation.

The diversion channel, known as the "Red River Floodway" (Fig. 1), will be approximately 30 miles long, 1,000 feet wide at the top, and 25 to 75 feet deep, with an average depth of approximately 30 feet. The width of the base of the channel, as shown in typical cross-sections given by Mishtak (in press), ranges from 540 feet south of Birds Hill to 380 feet north of Birds Hill. Inlet gates at an approximate elevation of 744 feet above sea-level will be situated at the south end of the Floodway; the outlet structure at the north end will be at an elevation of approximately 730 feet.

Bedrock in the area of the Floodway channel consists of highly fractured, coarse-grained limestone of the Ordovician Red River Formation. It occurs at depths ranging from 120 feet at the southern end of the channel to less than 30 feet at the northern end (see seismic profile, Fig. 5). From 10 to 30 feet of till, with local deposits of sand and gravel, overlies the bedrock. The till, in turn, is overlain throughout most of the area by lacustrine clay and silt deposited in glacial Lake Agassiz. In the Birds Hill area an extensive deposit of sand and gravel overlies the till and appears to have been deposited in the form of an outwash delta. The centre line of the Floodway intersects the sand and gravel deposit at the apex of the delta.

The Red River limestone and, locally, the lower part of the glacial till form an artesian aquifer that has served as a source of water supply for many years. The piezometric surface of the bedrock aquifer is shown in Figure 1 (in pocket). The sand and gravel deposit in the Birds Hill area forms a water-table aquifer and, as indicated by the piezometric contours in Figure 1, acts as an important source of recharge to the bedrock aquifer.

The amount of recharge to the bedrock aquifer is not known, but the continuous decrease in size of the area of flowing artesian wells southeast of Winnipeg during the past 40 years indicates that discharge from the aquifer is in excess of recharge (J.E. Charron, 1963, personal communication).

During excavation of Test Pit I, in the fall of 1961, springs began to flow from fractures in the glacial-lake deposits at the sides and base of the pit. These springs were of such a magnitude that pumping of water from the pit was necessary to permit continuance of excavation.

The unexpected occurrence of these springs, in an area underlain mainly by relatively impervious lake clay, raised questions as to the source of the water. Readings from piezometers, installed at the Test Pit I site for soil-mechanics investigations, clearly indicated that water entering the pit was being derived by upward leakage from the underlying artesian bedrock-aquifer.

The presence of the water-table aquifer in the Birds Hill area and the artesian leakage into the test pit indicated that dewatering may be a pertinent factor during the Floodway construction and that the excavation may create a further decline in the piezometric surface of the bedrock aquifer.

A request was made by the Director of the Water Control and Conservation Branch, Manitoba Department of Agriculture and Conservation, to the Director of the Geological Survey of Canada, for assistance in assessing the magnitude of the groundwater problems associated with the Floodway.

The purpose of this report is to outline the extent of groundwater investigations made and to present the results of analyses based on field data accumulated prior to construction of the Floodway.

The authors wish to acknowledge the extensive hydro-geological field data made available to them by F.W. Render, Floodway Geologist.

All data for the seismic survey were collected and originally computed by J.E. Murray (chief of party) and A.S. Ruffman, as part of a summer assignment in 1962. The cooperation of A.G. Mensforth, Chief, Floodway Division, Water Control and Conservation Branch, Manitoba Department of Agriculture and Conservation is gratefully acknowledged. The ready access to the project and divulgment of all available information pertinent to it greatly assisted the operators and interpreter in their tasks. Messrs. J. Love, F.W. Render,

L. Gray, and R. Leuzinger of the Floodway Division were most helpful in providing maps, sections, bore-hole data, etc. W. Brisbin, University of Manitoba, provided preliminary velocity data in the immediate area of the Floodway and his assistance is gratefully acknowledged.

The authors wish to thank the Director, Water Control and Conservation Branch, Manitoba Department of Agriculture and Conservation, for his permission to publish these data.

Preliminary Hydrogeological Investigations

The program of preliminary hydrogeological investigations, prepared by the Engineering Geology and Groundwater Section of the Geological Survey to assist in assessing groundwater problems associated with the Floodway, was begun early in 1962 and was essentially completed by the end of 1963. It consisted of the following four activities:

1. Inventory of water wells in the vicinity of the Floodway, to determine the configuration of the piezometric surface of the artesian aquifer and to provide a record of pre-construction water levels in both artesian and water-table wells in the area.

2. Pump tests at four sites along the Floodway right-of-way to determine coefficients of transmissibility and storage for the bedrock aquifer, and at one site in the Birds Hill sand and gravel deposit.

3. Installation of 23 permanent observation wells along the limits of the Floodway right-of-way and, in some places, beyond the limits of the right-of-way. Pump-testing and installation of observation wells were carried out under a contract let by the Water Control and Conservation Branch. Water-well inventory, tabulation of these data, supervision of observation-well installation and other hydrogeological and geological investigations have been carried out either by or under the direction of the Floodway Geologist.

4. A hammer-seismic survey along the centre-line of the Floodway, to extend existing bore-hole data and to determine areas in which the largest groundwater flows might be anticipated as a function of the proximity of the base of the Floodway to bedrock. This survey along the Floodway centre-line was carried out during the summer of 1962 by personnel of the Geophysics Division of the Geological Survey. A description and the results of this seismic survey are contained in Part I of this report.

Part I — HAMMER SEISMIC SURVEY

by George D. Hobson

Introduction

Surface elevations over the extent of the Floodway area are generally between 755 and 775 feet above sea-level, while the Birds Hill esker at chainage 500+00 rises about 35 to 45 feet above the flat surrounding Red River Valley plain. All elevations used for data computational purposes were taken directly from a centre-line topographic profile supplied by the Floodway Division.

Excessive precipitation during the summer of 1962 slowed survey progress considerably. Some areas, particularly between chainage 1144+00 and 1186+00 were inundated most of the summer. This condition more or less dictated the program for the seismic team and necessitated extensive moving and change of operating location from day to day. No extensive areas along the centre-line were omitted by employing this technique of working where and when weather conditions dictated.

Seismic exploration depends fundamentally upon the propagation of seismic waves within elastic media. Elastic waves generated by explosions, or man-made sources in general, travel downward in all directions following the physical laws of optical theory. These waves are reflected and refracted at interfaces at depth and return to the surface of the earth. The interpretation of recorded seismic data consists of determining the velocity of propagation of these elastic waves and analyzing the refraction and reflection phenomena at the interfaces or boundaries between rock layers that are characterized by different acoustic properties. The refraction phenomena were of principal interest in this investigation.

The quantity observed and recorded at each location is the time interval between the initiation of the elastic wave by an explosion or hammer blow and the first disturbance of the ground as detected by a seismometer at a known distance from the source of energy. The proportion of the energy refracted is dependent on the difference in propagation velocities on opposite sides of the acoustic boundary. The bases of the refraction technique are Snell's Law and Huygen's Principle and its successful application is dependent on the increase in velocity with depth.

It is customary to use only the first arrivals of elastic energy in the seismic refraction method but frequently it becomes necessary to utilize second and later cycles of the initial wave train. It is possible to use a later cycle consistently throughout a profile, to obtain lower velocities than true velocities, and, because of the different time-intercept on a time-distance graph, to arrive at a depth determination that is fairly accurate.

The second cycle of the primary refracted wave has been observed and interpreted frequently during this survey.

Further information on seismic refraction theory and later cycles of arriving energy may be obtained from such standard references as Dobrin (1960), and Nettleton (1940).

Instrumentation

A model FS-2 portable seismograph (Plate I) by Ronka Geophysical Instruments Limited was used throughout this survey. It is a complete reflection-refraction seismograph weighing 30 pounds and is operated by a two-man crew. All events are permanently recorded on dry electro-sensitive paper by a sweeping electric stylus. The time base for the recorder consists of a constant frequency oscillator operating at 400 cycles per second, feeding power to a small synchronous motor fitted with a clutch and actuated by a solenoid. The initiation of the energy by hammer or explosives closes the blaster circuit and delivers a pulse to this solenoid, thereby connecting the stylus assembly to the motor drive. Standard dry "lantern" batteries supply 24 volts as power supply for the instrument. Two geophones detect arrivals of energy and these signals can be correlated to the enhancement of the signal-to-noise ratio.

Instrumentally, it is of interest to note that the FS-2 instrument appears to be capable of recording both the true direct wave through the aerated zone near the surface and the ground-coupled air-wave at the surface. This cannot be done simultaneously on most other hammer instruments in which the application of special techniques is required if both waves are to be recorded.

Field and Recording Procedure

Depth determinations to Pleistocene interfaces and to bedrock were made at 229 locations from unreversed seismic refraction profiles. Geophone stations were selected approximately 600 feet apart along the length of the Floodway centre-line. The hammer positions were extended out from this recording location until the bedrock velocity was observed. In general, no correction for topographic relief was necessary in the computations. All recorded time data were transferred to a time-distance graph at each location to ensure that penetration of the seismic energy to the bedrock formation had been

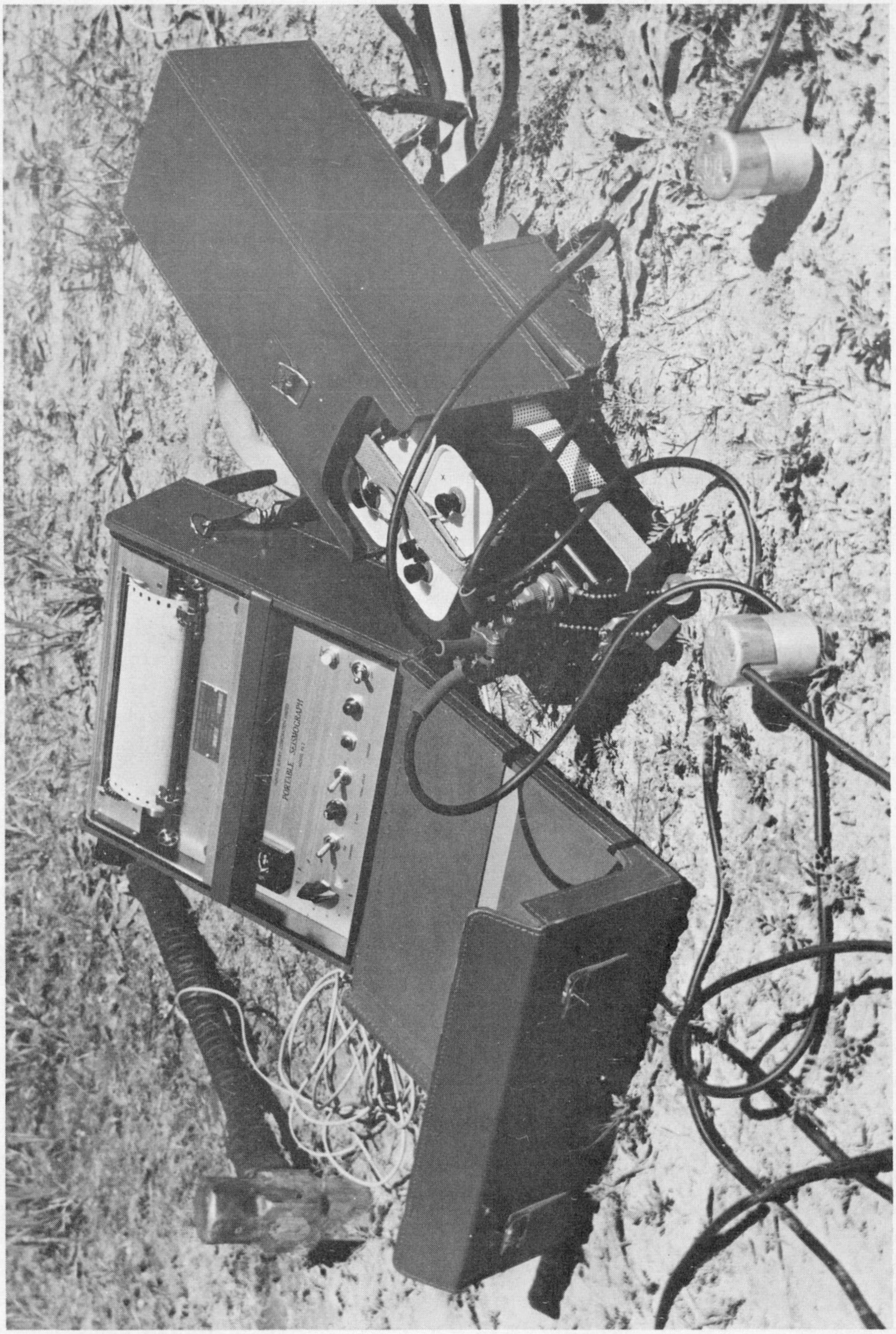


Plate I. The FS-2 seismograph.

achieved. A few reversed profiles were shot during the project, but it was not deemed necessary to the successful completion of the survey that reversed profiles should be an integral part of the field procedure. All velocities discussed later in this report therefore are uncorrected for dip.

It was never necessary to extend the hammer line beyond 400 feet from the geophone location because the breakover into the bedrock velocity was always recorded before this distance was reached. Consequently, explosives were not used during this survey. In general, all locations yielded data of excellent quality for hammer distances to 200 feet. At distances greater than 200 feet, about 20 per cent of the records deteriorated to fair or good quality, but no locations yielded unreliable, or uninterpretable or questionable data.

The signal-to-noise ratio on the records is generally high. The records at about 10 per cent of the locations indicated considerable noise picked up by the geophones, but the true seismic signal can generally be distinguished on the record.

Figure 2 is a typical record produced by the FS-2 portable seismograph near the mid-point of the length of the Floodway. It illustrates good sharp arrivals of first and later cycles of energy.

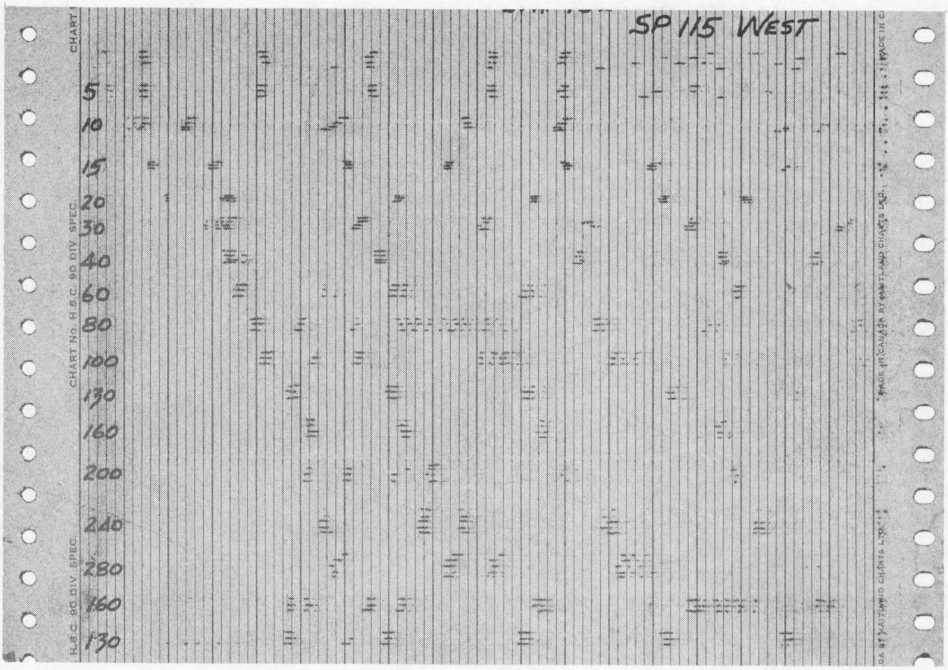


Figure 2. Typical record by the FS-2 seismograph.

Noise is at a minimum on this record, but has been recorded and can be readily detected. The field procedure of returning to selected locations to verify or amplify the time-distance plot is shown by the addition of the last two hammer positions after the profile had been completed to the required distance. The ground-coupled air-wave at the surface is recorded as the first arrival of energy at stations out to 20 feet from the geophone. The second arrival indicated on the record is the true direct wave through the aerated zone.

The time-distance plot for this sample record is shown in Figure 3. In this plot the ground-coupled air-wave is represented by the velocity line labelled "1,085 ft/sec" and the true direct wave by the curve labelled "800 ft/sec". This plot is typical of the reliability of the data and indicative of a general lack of scattering of the points used to draw the time-distance graphs. A more detailed discussion of velocities and lithology will follow. There are areas in which record quality and hence P-wave plots are of poorer quality, but fortunately these areas are localized and infrequent.

Results and Conclusions

As mentioned earlier in this report the velocities observed are obtained from unreversed refraction profiles and are therefore apparent velocities and uncorrected for dip.

Table I illustrates the accuracy of the seismic method over the project. Depth-to-bedrock determinations were made at each of eight diamond-drill holes on the centre-line of the Floodway, with an average percentage error of 7.3 as indicated in the table. Positive (+) percentage error indicates a depth determination too shallow as compared to known data, and negative (-) percentage error indicates a depth determination too deep.

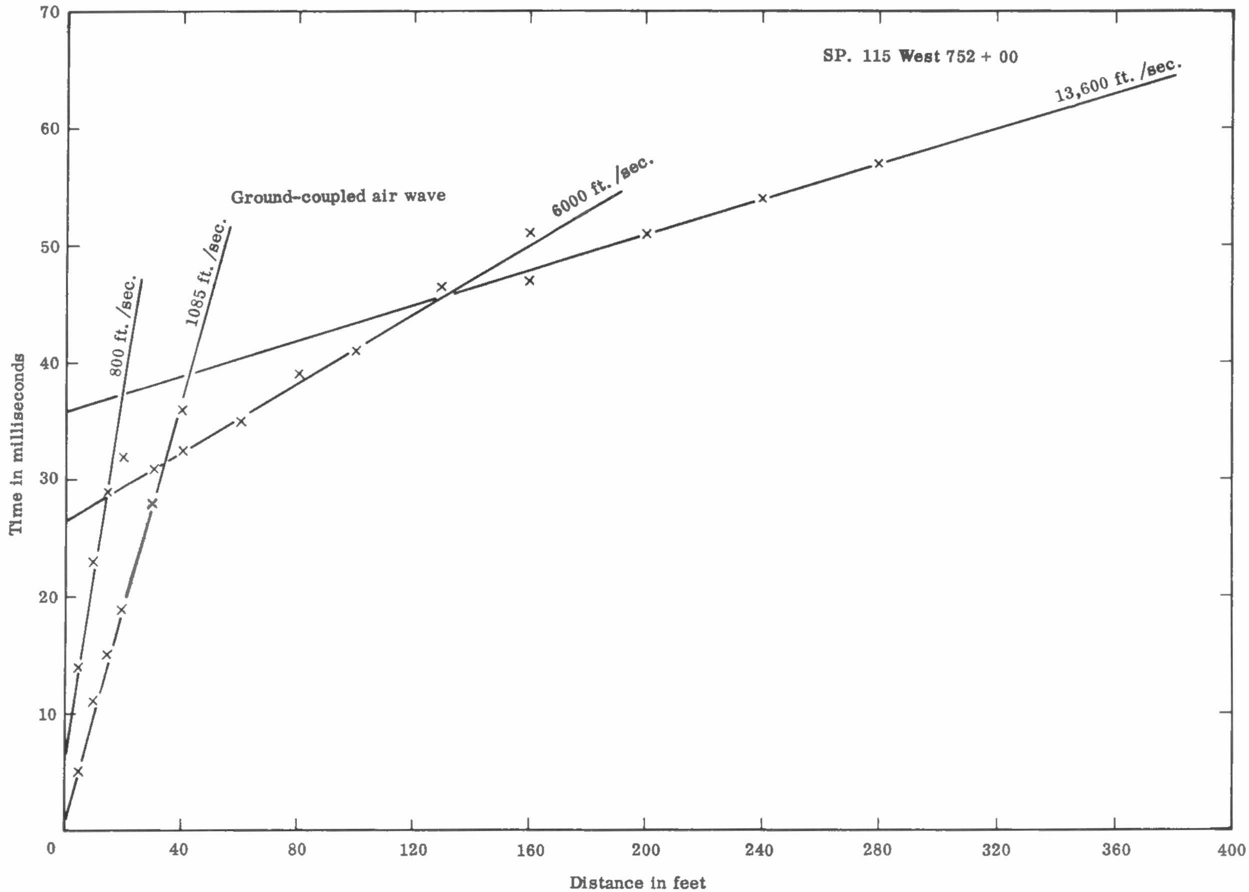


Figure 3. Typical time-distance plot

Table I

Comparison of Bore-hole and Seismic Data

D.D.H.	Bedrock by Drill	Elevation by Seismic	Drift Thickness (feet)	Percentage Error
1	675.0	684.0	88.0	- 10.2
2	675.6	674.0	89.4	+ 1.8
3	700.0	705.0	75.0	- 6.7
4	707.0	702.0	69.0	+ 7.2
5	721.0	713.3	52.0	+ 14.8
6	703.0	705.0	60.0	- 3.3
7	693.4	696.5	65.4	- 4.7
8	712.3	716.0	44.7	- 9.6
			Average	7.3

The data used to determine the percentage errors in Table I are taken from the field graphs. No attempt has been made to make a 'better fit' by adjusting velocities so that these data are indicative of the general reliability of the survey. An outside limit of 10 per cent error is suggested in conformance with seismic techniques.

A histogram of observed velocities in ft/sec vs. frequency of occurrence in arbitrary units is presented in Figure 4. This includes all velocities from the 229 locations used to compute depths to bedrock. Secondary events of velocity have not been included. A basic interpretation of the various peaks and ranges of velocities is included on the histogram.

The first peak on the histogram is that associated with the surface layer or aerated zone. This peak is at approximately 650 ft/sec and is the true direct wave. The second peak between 1,050 and 1,150 ft/sec is probably that peak associated with the ground-coupled air-wave at the surface. This air-wave was obviously not a serious problem in the computation procedure.

Over the length of the Floodway project it appears to be impossible to correlate seismic velocities with the near-surface Pleistocene clay materials other than to indicate, in places, the presence of variable clay layers or sandy clay. This can be done only locally. The range of velocities between 2,000 and 4,000 ft/sec has been indicated as sandy clay on the histogram. Although the velocity interval of 4,000 to 6,000 ft/sec has been designated as clay it does not

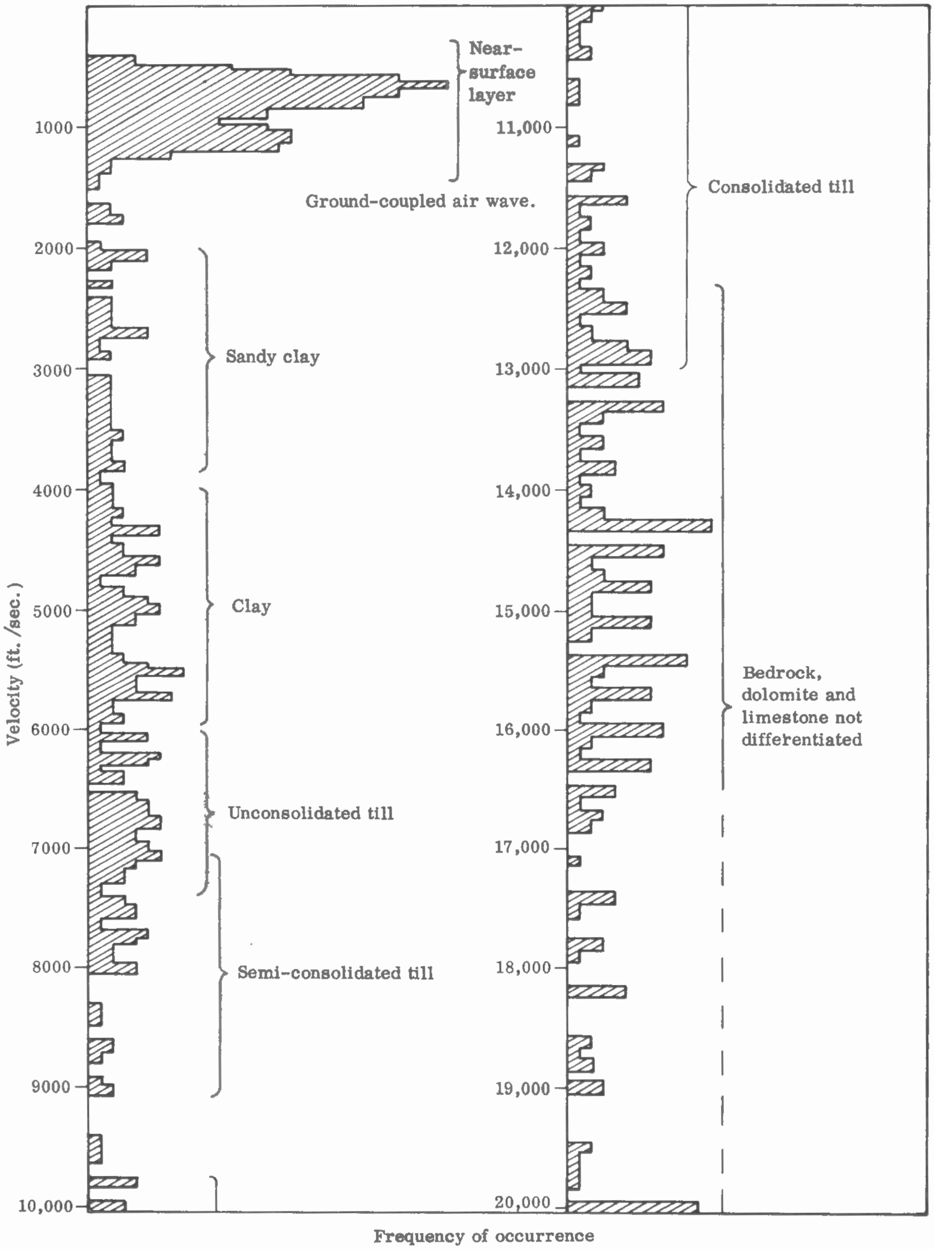


Figure 4. Histogram of observed velocities

appear to be possible to differentiate between various types of clay over the project area, except to suggest the possible presence of sandy clay. This is accomplished by noting singular low velocities particularly located in the geologic section. However, the lacustrine clays in the area may not permit differentiation by seismic methods. There may be an overlap of velocity ranges for clay and till over the project area when a comparison is made between known stratigraphy in the boreholes and seismic velocities recorded at those particular locations.

An attempt has been made to correlate observed velocities with till materials for purposes of making the prognosis of materials to be moved above the grade of the Floodway bottom. Unconsolidated, semi-consolidated, and consolidated tills can be differentiated, although there is an indicated overlap of velocity ranges for the first two of these. However, it is believed that they can be differentiated within the seismic section on a depth basis.

A similar overlapping of velocity ranges for bedrock and consolidated till is also indicated on Figure 4, but, similarly it is believed that over the Floodway project these two materials can be differentiated on a depth basis. Bedrock velocities scatter over a considerable range of values,as would be expected from unreversed profiles, but appear to predominate in the range between 14,200 and 16,000 ft/sec. No differentiation between dolomite and limestone can be made, as is apparent on Figure 4.

The wide spread of velocities bracketed to indicate various Pleistocene materials need not be alarming, because this range is frequently encountered. The range of velocities indicated as bedrock is always encountered when reversed profiles are not shot to account for dipping strata. The indicated narrow range of 14,200 to 16,000 ft/sec is probably indicative of the true bedrock velocity.

The following tabulation sets out velocities that can be anticipated for various materials over the Greater Winnipeg Floodway area.

$<$ 1,500 ft/sec	- surface layer, loam, silt etc.
2,000-3,800 ft/sec	- sandy clay
2,000-6,000 ft/sec	- clay
6,000-7,400 ft/sec	- unconsolidated till
7,050-9,050 ft/sec	- semi-consolidated till
9,800-13,000 ft/sec	- consolidated till
$>$ 12,200 ft/sec	- bedrock

This velocity - material relationship has been adhered to in compiling the prognosis. The above tabulation is, of course, a generalization for there will be exceptions governed by near-surface conditions and apparent velocities.

W. Brisbin (personal communication) has observed comparable velocities for these materials at Birds Hill esker (Fig. 5, in pocket) and the test pit (chainage 1,000+00, Fig. 5) during experimental hammer seismic surveys employing reversed refraction profiles. He spent considerable time and effort at these two locations on the Floodway in an attempt to correlate seismic velocities with Pleistocene materials. Brisbin has also been unable to differentiate between the dolomitic and limestone bedrock formations by seismic methods in this area.

Figure 5 is a section of the surficial geology along the centre-line of the Red River Floodway, as interpreted from this hammer seismic survey. A prognosis of material that may be moved above the grade of the Floodway bottom is derived from a correlation of seismic velocities with Pleistocene lithology. Low seismic velocities are indicative of unconsolidated easily moved materials, whereas high seismic velocities suggest the presence of tills of varying degrees of consolidation, which may require different techniques and equipment for excavation. The greatly exaggerated scale of Figure 5 (vertical exaggeration X200) gives the appearance of a sawtooth bedrock surface. This scale of presentation has been maintained to conform with that used by the Floodway Division for other plans and sections. Actually, there is no reason to believe that bedrock should be a plane surface, for cross-sections on the centre-line profile showing the locations and logs of test holes indicate that there may be changes in vertical elevation of 4 to 6 feet over 200 feet horizontal on the top of a till formation. This same section shows considerable change in thickness of the clay layers down to the top of the till formation. The sawtooth irregular appearance of this exaggerated section is therefore not unrealistic.

The average thickness of the near-surface aerated low-velocity layer is 9.3 feet over the length of the Floodway with local thinning and thickening as indicated on the section. Average maximum thickness of this layer is about 15 feet, whereas the average minimum thickness is about 5 feet. Excavation to depth will be more rapid in areas of a thick aerated surface layer than in areas where clays and tills are shallow in depth. Areas in which the very low true direct wave of velocity about 650 ft/sec could not be recorded are infrequent, indicating that, in general, the near-surface layer should be moved easily by earth-moving equipment. This surface layer in the principal area where only the ground-coupled air-wave was recorded, chainage 90+00 to 170+00, is about 9 feet thick and probably will not affect excavation progress.

The average thickness of overburden over the length of the Floodway is 61.6 feet. In no place will bedrock penetrate the base of the Floodway. In general, the overburden over the extent of the

Floodway south of the Birds Hill esker is relatively simple geologically. At Birds Hill, tills within the overburden will begin directly to influence excavation techniques and costs. North of Birds Hill the near-surface layers of drift are less homogeneous than in the southerly direction and will also influence excavation techniques and costs.

The writer suggests that the general area of the contact between bedrock dolomite and limestone is at chainage 1,329+00 on the well inventory base-map (F.W. Render, personal communication). The seismic data neither deny nor confirm this suggestion because there does not appear to be any definite distinction between these two bedrock materials based on a study of seismic velocities. There is an indication of a bedrock 'high' between chainages 1,300+00 and 1,400+00, but this is not necessarily indicative of a change in lithology.

Perusal of the section in Figure 5 indicates several locations at which bedrock apparently approaches and, in some cases appears to penetrate, the base of the Floodway. At chainage 15+00, the apparent approach of bedrock to the Floodway bottom is probably caused by the presence of a very dense till below the clay and immediately overlying the bedrock. Heavier tracked equipment, as opposed to rubber-wheeled vehicles, may be required to rip this material. The bedrock profile at chainage 40+00 also appears to indicate a definite penetration of the bottom of the Floodway, but this too is an erroneous indication, for the velocity of 10,500 ft/sec recorded in this area is too low to be a bedrock velocity and must be interpreted as consolidated till. It is, however, a direct and positive indication that some difficulty may be encountered in the earth-moving process. Bore-holes to bedrock for the Canadian National Railways bridge crossing at chainage 38+00 confirm that the relatively high final velocities observed at the neighbouring locations are associated with a dense consolidated till, which definitely will require heavy equipment to move. A similar situation exists at chainage 270+00 where consolidated till approaches within 5 feet of the Floodway bottom. This is another area in which it may be more difficult to move earth.

At Birds Hill esker, we note the presence of a two-layer seismic case in that the low-velocity material has been completely removed from the gravel pit. This is the only example of a two-layer case over the entire project. Beneath the Birds Hill esker and extending for considerable distance to the south, from chainage 490+00 to 630+00, the bedrock definitely approaches but does not penetrate the base of the Floodway. The penetration indicated at chainage 498+00 is erroneous, but once again indicates the presence of consolidated till at the base of the grade. The bedrock 'high' does not appear to be due to a velocity 'pull-up' as a result of the topographic surface relief, but conversely that the topographic high feature is set upon a bedrock high. We note a general decrease in the final velocity observed under the Birds Hill esker, indicating that the highest and final velocity recorded at each location should be interpreted as a consolidated till at the base of the Floodway. Bedrock in this area is represented by a broken line. This apparent approach of bedrock to the base of the Floodway and the definite indication of the presence of consolidated till at the base are factors that must be considered in the groundwater studies of the area.

A generalized bedrock profile would indicate a bedrock high, elevation approximately +715 feet, under the general area of Birds Hill at chainage 500+00, approaching to within about 50 feet of the mean land surface. The bedrock surface generally dips southward from this point to depths of about 100 feet below surface at the southern end of the Floodway, elevation approximately +655 feet and chainage 1,400+00, and dips northward from the same point to depths of about 45 feet below surface, elevation approximately +710 feet and chainage 70+00, at the northern extremity of the Floodway.

Part II — GROUNDWATER INVESTIGATIONS
by J.S. Scott and R.O. van Everdingen

The following three groundwater problems were studied in connection with the Red River Floodway:

1) influence of the excavation on water levels in bedrock wells in the area (drain effect);

2) time required to reach maximum drawdown; and

3) amount of groundwater discharge into the Floodway channel.

Electric-Analog Model

For the study of the first problem an electric-analog model of the continuous-solid-conductor type was built, using Teledeltos paper. This type of model is not the most suitable for the problem, but the paper and instrumentation were readily available and the scarcity of basic data did not justify the use of a more sophisticated method. The paper model is indicated in Figures 7 and 8 by the letter C.

The Servomex field-plotter FP 92 (a null-type instrument) was used as both power supply and measuring instrument (A in Fig. 7). A potential divider consisting of a bank of 17 parallel potentiometers of 0.25 Megohm supplied the required potentials to the model (B in Fig. 7).

Representation of the present piezometric surface for the Red River limestone on Teledeltos paper is virtually impossible. To try to apply the changes resulting from excavation of the Floodway channel to such a model would be a hopeless task. The model was therefore set up as a secondary flow-field model or drawdown model (de Jong, 1962a; van Everdingen and Bhattacharyya, 1963).

It was assumed that excavation of the channel to a certain elevation will lower the piezometric surface at that point to the same elevation. Thus, the projected Floodway channel is represented in the model by a number of electrodes. The electric potentials of these correspond to the difference in elevation between the present piezometric surface and the future grade of the channel (this difference being equivalent to drawdown).

The maximum drawdown of 50 feet is assumed for the Birds Hill area, and represented by a 100 per cent potential electrode. The potentials applied to the other electrodes are percentages of this maximum, simulating the amount of drawdown expected at the location of each of these electrodes.

Electrodes having a certain length and potential differences of 5 and 10 per cent were used, although there is a continuous variation in the amount of expected drawdown along the channel.

The Red River is simulated in the model by a zero-per cent equipotential electrode. This arrangement implies that no drawdown resulting from the Floodway excavation is expected beyond Red River. It must be realized that this may not be the case, especially in the southern half of the area. Figure 6 shows the influence of Red River on the piezometric surface in the aquifer, near the Floodway inlet structure. Hydraulic connection between river and aquifer seems to be incomplete, which may well indicate that some drawdown beyond the river will result from the excavation. As more detailed data are not available, the model was set up on the above-mentioned simplifying assumption.

Part C in Figure 8 shows the arrangement of, and the potentials applied to the various electrodes. The solid lines between the river and the Floodway and the broken lines beyond the Floodway indicate the potential distribution as measured on this arrangement of the model.

It will be noted in Figure 8 that the broken lines for 10, 20, 30 and 40 per cent intersect the model edges at right angles, implying that there is no secondary flow in a direction normal to these edges. It is possible to test the validity of this implication by enlarging the area covered by the model, but this would create difficulties in using the model. Boundary problems like this can be solved, as indicated by de Jong (1962b) by applying a conformal transformation to the area outside the model.

A potential current in a field with a certain coordinate system will keep its characteristics when a conformal transformation is applied. All angles in the transformed field will have the same values as in the original field. The transformation principle used here can be described as follows. A point P outside a circle with radius R, at a distance r from the centre of the circle can, through transformation, be represented by a point P', inside the circle, at a distance r' from the centre, such that

$$r \times r' = R^2.$$

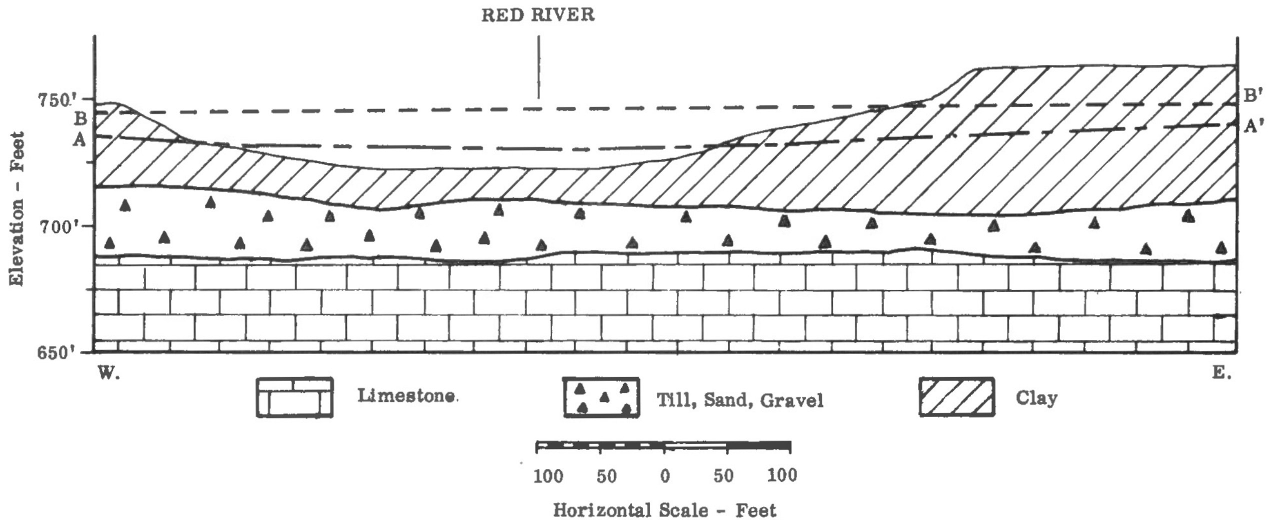
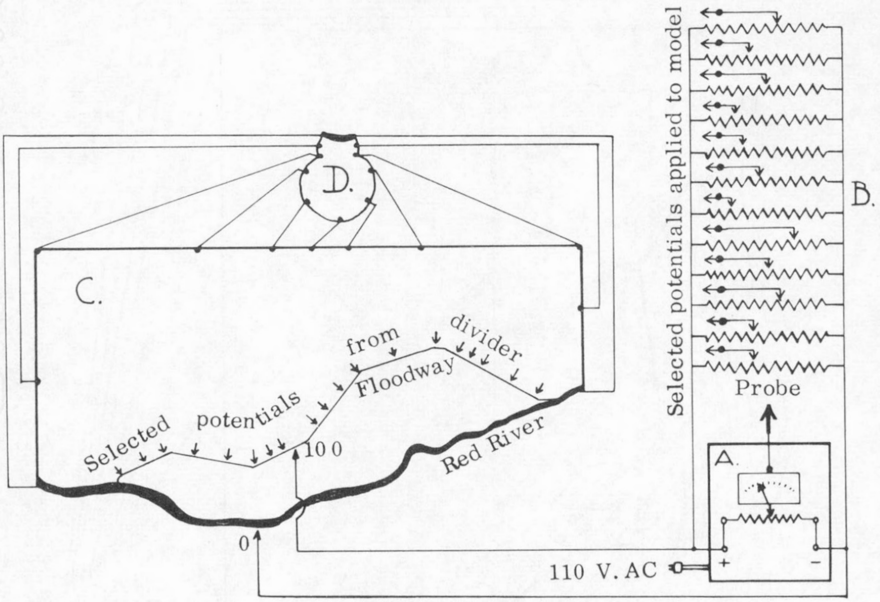
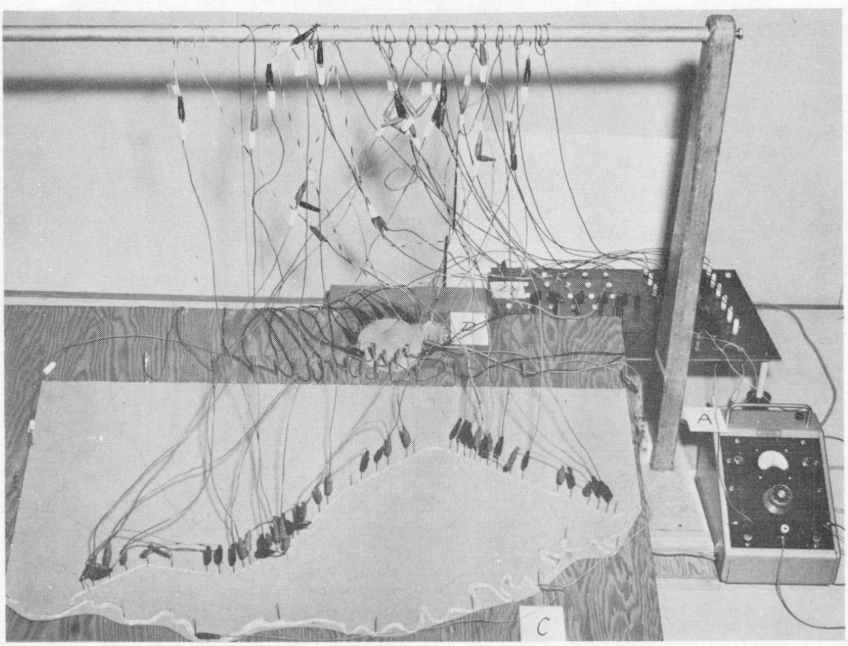


Figure 6. Cross-section of Red River at inlet-control-structure site showing influence of river valley on the piezometric surface of the bedrock aquifer. A-A', piezometric surface of bedrock aquifer determined from bore-holes. B-B', westward projection of regional piezometric surface

Figure 7. Electric-analog model of the Red River Floodway.



A. Schematic circuit diagram.



B. Model arrangement.

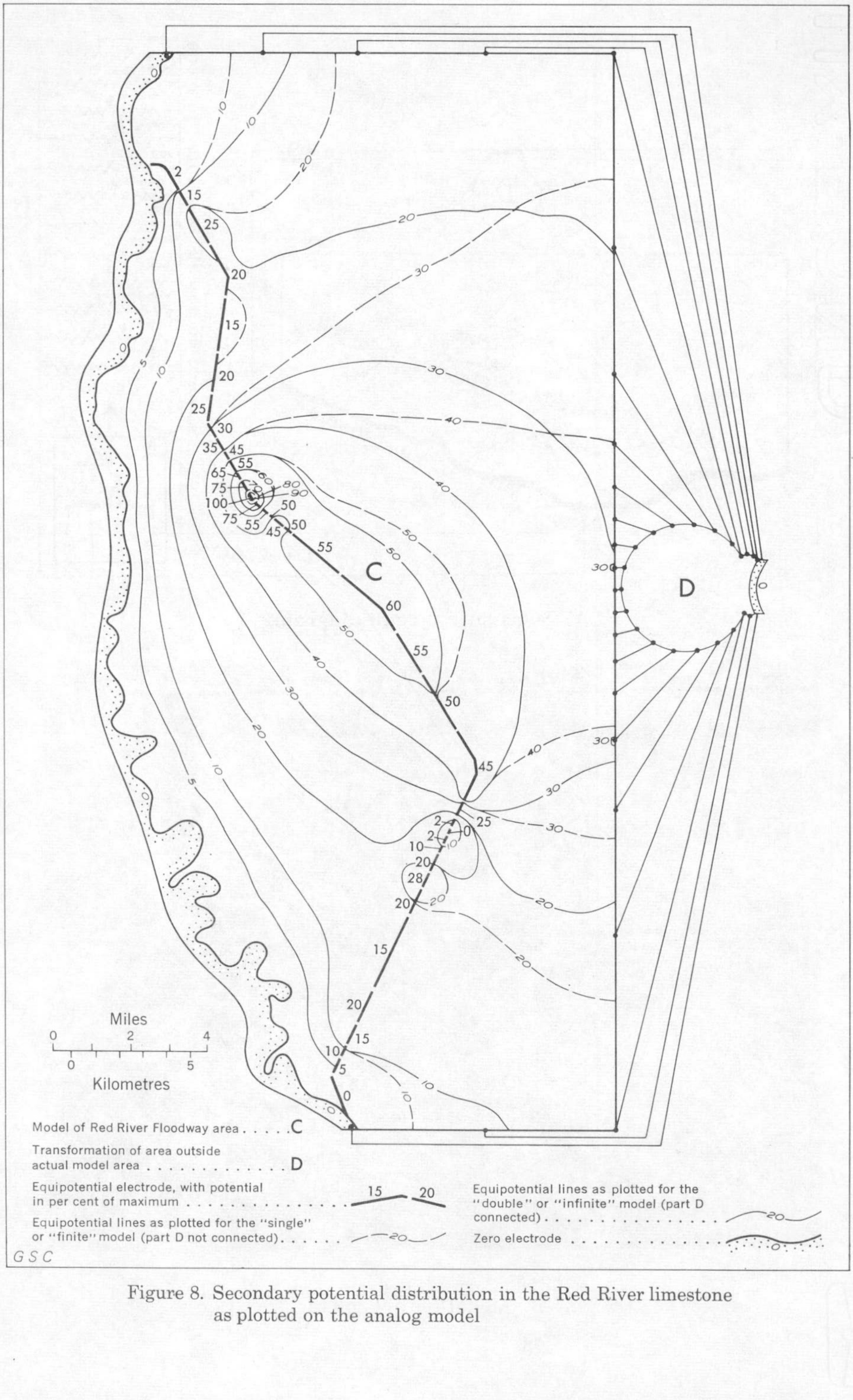


Figure 8. Secondary potential distribution in the Red River limestone as plotted on the analog model

The transformation applied to the area outside the main Floodway model C resulted in the second model, part D, connected to part C in 22 different points (see Figs. 7 and 8). Using this somewhat more complicated arrangement the potential distribution indicated by the solid lines in Figure 8 was obtained.

Next, the potential distribution in Figure 8 (solid lines) was adjusted for the inaccuracy caused by the use of discontinuous potential variation along the Floodway channel. Only minor changes were necessary, as can be seen from a comparison of Figures 8 and 9. In addition, potential percentages were converted into feet of drawdown as indicated in Figure 9, which represents predicted drawdown in feet.

The eventual piezometric surface that will result when equilibrium conditions are established after completion of the excavation, is shown in Figure 10. It was derived by subtracting the predicted drawdown from the original piezometric surface.

Figure 9 gives the maximum possible drawdown. The ultimate values may be limited by recharge and they will almost certainly be limited by head losses through the till and lacustrine clay between the Floodway grade and the limestone aquifer. This implies also that the point of zero-drawdown on the east side of the Floodway will probably lie closer to the channel than is predicted by the model results.

The type of model used in this study only gives the eventual equilibrium drawdown values, regardless of the time factor. Predictions on the time necessary for reaching equilibrium, and on the amount of groundwater discharge into the Floodway channel at various times, thus had to be arrived at in another way.

Time Factor, Derived from the Constant-Head Drain Formula

The time required to reach the condition of equilibrium, indicated by the analog model, was approximated by the use of a constant-head drain equation given by Stallman (in Ferris et al., 1962), derived from the solution of an analogous heat-flow problem.

According to Stallman the distribution of drawdown in an artesian aquifer at any distance X from a stream or drain, subsequent to a sudden change in stage, is given by the expression

$$\Delta = \Delta_o \left[1 - \frac{2}{\sqrt{\pi}} \int_0^{\frac{x}{2\sqrt{Tt/s}}} e^{-u^2} \, du \right] \qquad \ldots\ldots\ldots\ldots(1)$$

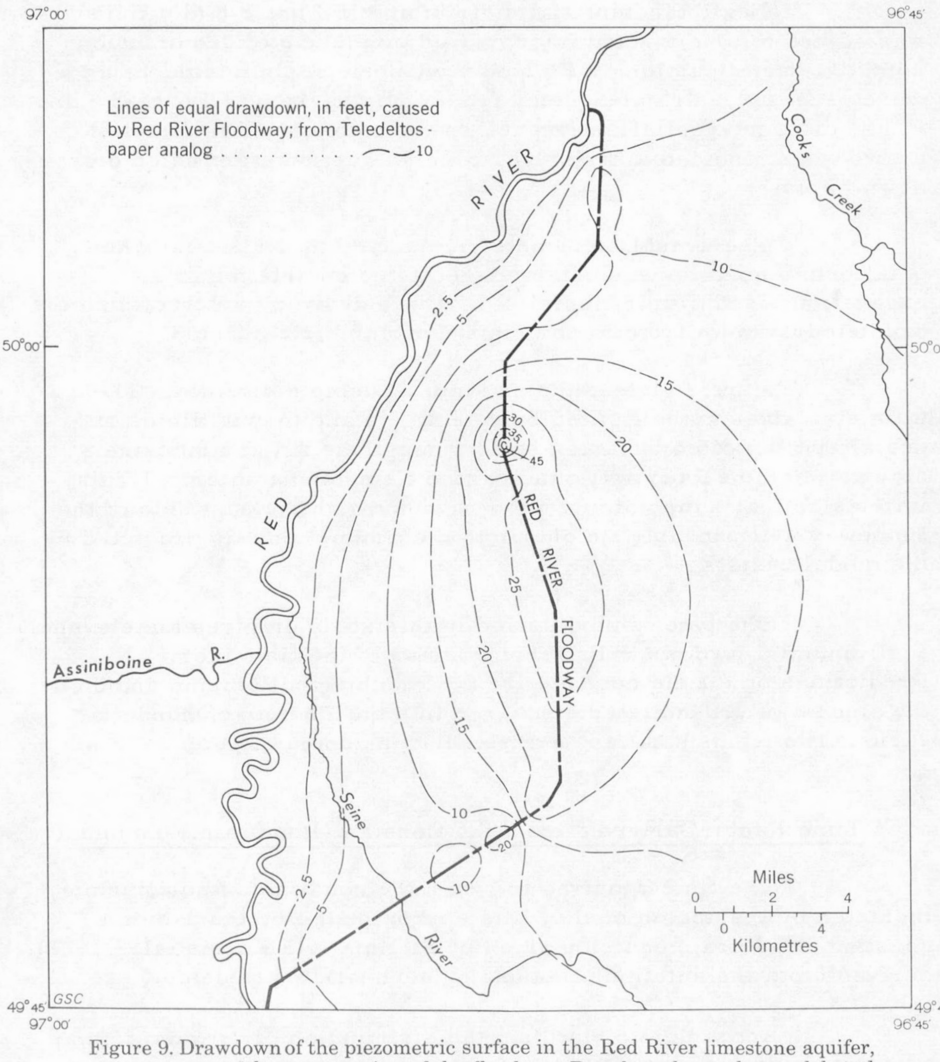


Figure 9. Drawdown of the piezometric surface in the Red River limestone aquifer, caused by construction of the floodway. Based on the analog-model study

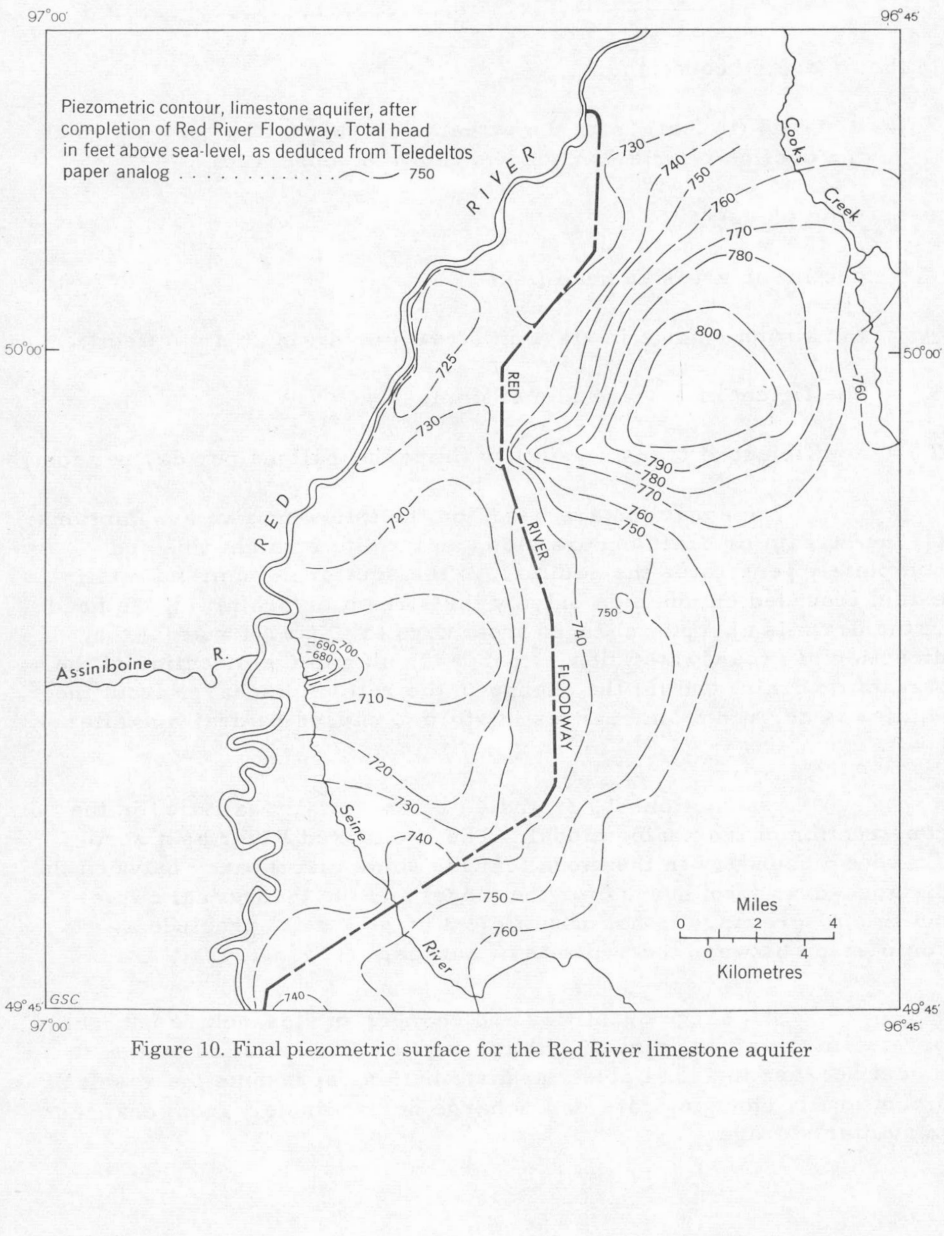


Figure 10. Final piezometric surface for the Red River limestone aquifer

or

$$\Delta = \Delta_o \, D(u)h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where D(u)h, replacing the quantity in brackets, is termed the drain function of u for the constant-head situation, and where

$$u^2 = \frac{1.56 \, X^2 \, S}{Tt} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

In the foregoing equations:

X = distance (in feet) from the stream or drain to the point at which the decline in artesian head is known or observed;

t = time (days);

Δ = decline in artesian head (feet);

Δ_o = the abrupt change in stage in stream or drain at $t = 0$ (feet);

S = coefficient of storage (dimensionless);

T = coefficient of transmissibility (imperial gallons per day per foot).

The equations are based on the following five assumptions: (1) the stream or drain occurs along an infinite straight line and completely penetrates the aquifer; (2) the aquifer is semi-infinite in extent (bounded on one side only by the stream or drain); (3) the head in the drain is abruptly changed from zero to Δ_o at $t = 0$; (4) the direction of groundwater flow is perpendicular to the direction of the stream or drain; and (5) the change in the rate of discharge from the aquifer is derived from changes in storage caused by drainage after $t = 0$.

Assumptions 1, 3, and 4 are essentially satisfied in the construction of the analog model. The use of Red River as a zero-drawdown boundary in the model causes some discrepancy between the distance-drawdown curves for the model and the theoretical curves, but this discrepancy is not of sufficient magnitude to preclude a comparison between the two sets of curves.

Discharge quantities and changes in storage are not incorporated in the analog model, but assumption 5 is satisfied by the model because the final potential distribution represents the result of a continually changing rate in discharge derived solely from changes in aquifer storage.

Thus the drain equation can be used for the calculation of the time factor corresponding to the piezometric distribution given by the analog model. For this calculation it is necessary to obtain suitable values of the coefficients of transmissibility (T) and storage (S) for the aquifer affected by the drain.

The main artesian aquifer that will be affected by the drain is formed by the upper part of the fractured limestone that occurs at varying depths below the base of the drain (see Fig. 5). All material between the base of the drain and the top of the limestone must be included as part of the aquifer, because of the hydraulic connection of the limestone aquifer with the base of the drain through fractures in the intervening till and lacustrine clay.

Values of the coefficients of transmissibility and storage for the limestone aquifer are available from pump tests at several sites along the Floodway right-of-way, but these tests did not yield values of T and S for the material overlying the limestone. Values of T and S for the aquifer below the base of the drain could be calculated, however, from data obtained during dewatering of Test Pit I (see Fig. 11).

On April 16, 1962, Test Pit I was flooded and the water level in the pit reached an elevation of 771.8 feet by April 23, 1962. The water level in the pit then steadily declined to an elevation of 769.9 feet by August 21, 1962. On that date dewatering of the pit began, continuing until October 13, 1962 (see Fig. 12).

During the dewatering period, drawdown of water levels was recorded in observation wells Nos. 011, 12, 19, and 21, completed in bedrock, and surrounding the test pit (see Figs. 11, 13, and 14). Coefficients of transmissibility and storage were obtained by treating the test pit as a well and analyzing the drawdown data by matching of the curves with the type curve solution to the Theis non-equilibrium equation.

It was assumed that the influx of water into the pit was derived from the limestone aquifer by upward leakage through fractures in the overlying till and clay, similar to the flow conditions anticipated for the Floodway.

The test pit did not have vertical walls and did not completely penetrate the aquifer. Therefore the drawdown curves for the observation wells deviate from the type curve for the Theis non-equilibrium equation, which requires a fully penetrating vertical well as a condition for its application.

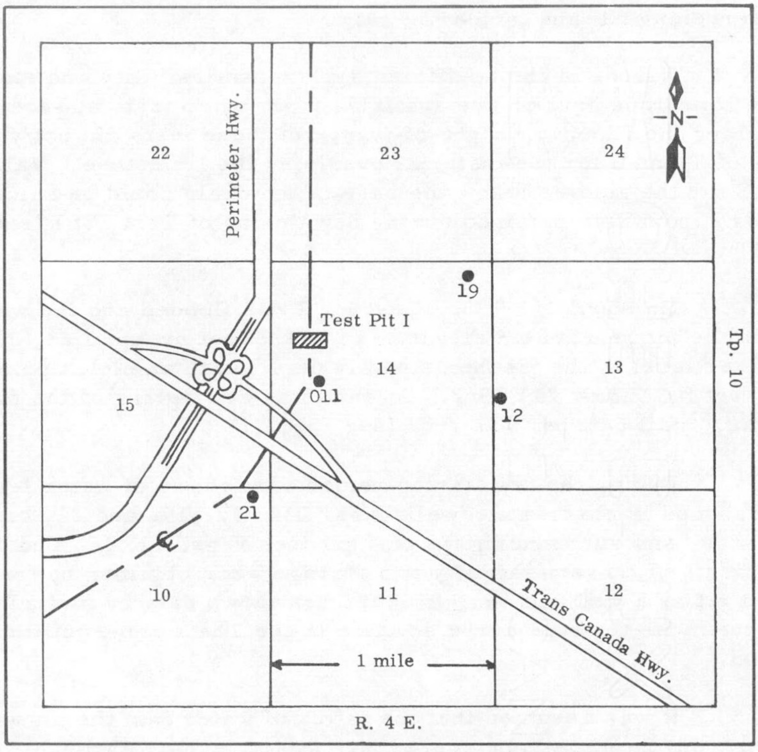


Figure 11. Location of Test Pit 1 and Observation Wells 011, 12, 19, and 21

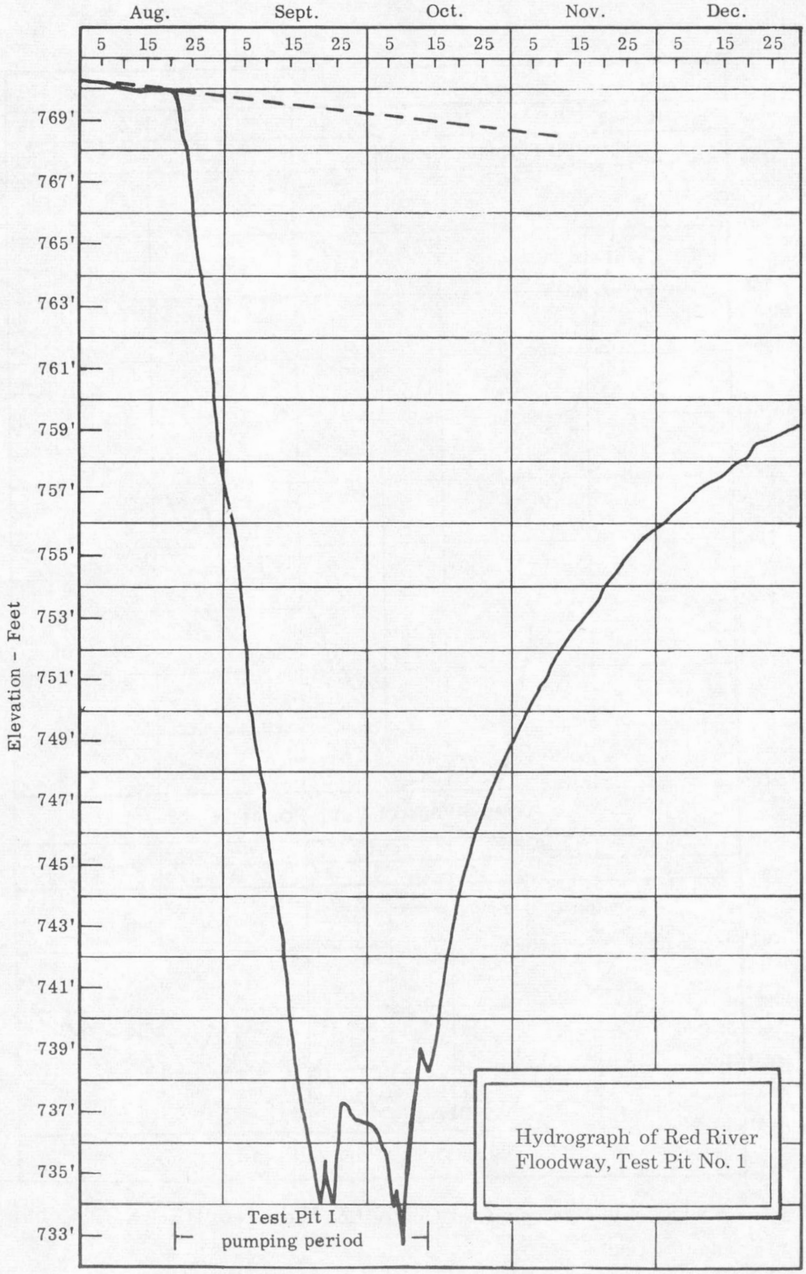

Aug.
Sept.
Oct.
Nov.
Dec.
5 15 25
Elevation - Feet
769'
767'
765'
763'
761'
759'
757'
755'
753'
751'
749'
747'
745'
743'
741'
739'
737'
735'
733'
Test Pit I
pumping period
Hydrograph of Red River
Floodway, Test Pit No. 1

Figure 12

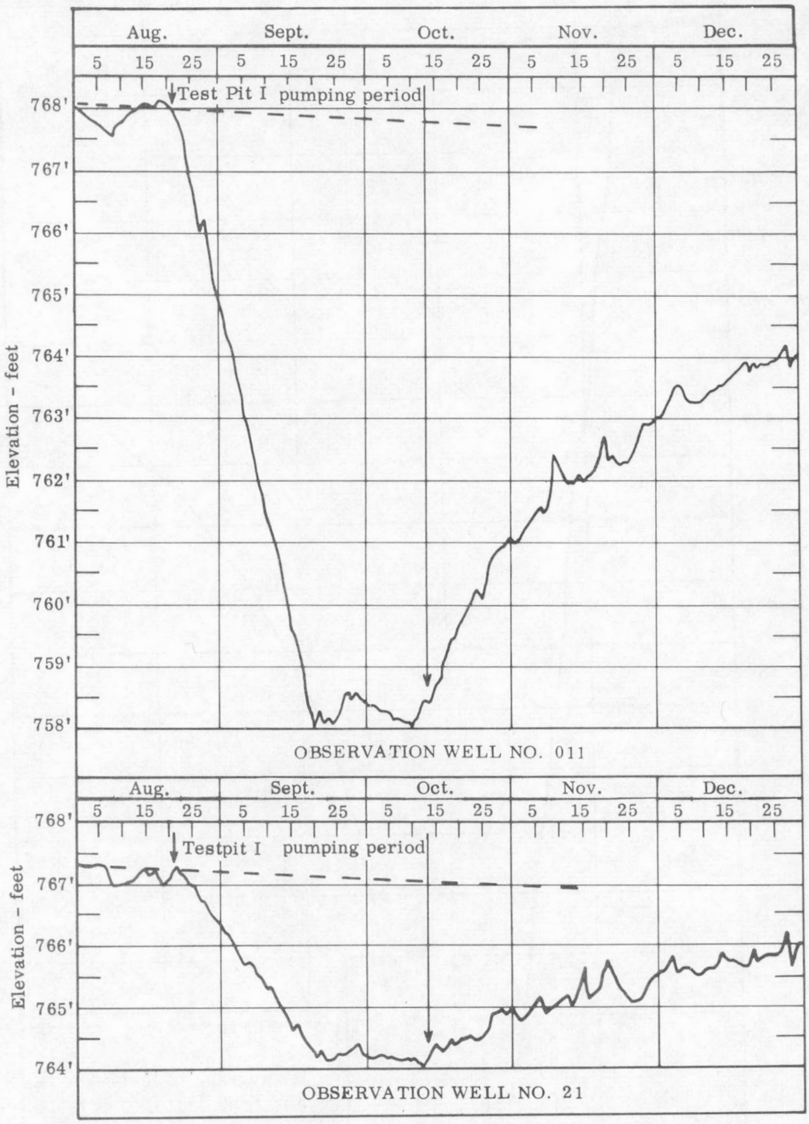


Figure 13. Hydrographs of Observation Wells No. 011 and No. 21

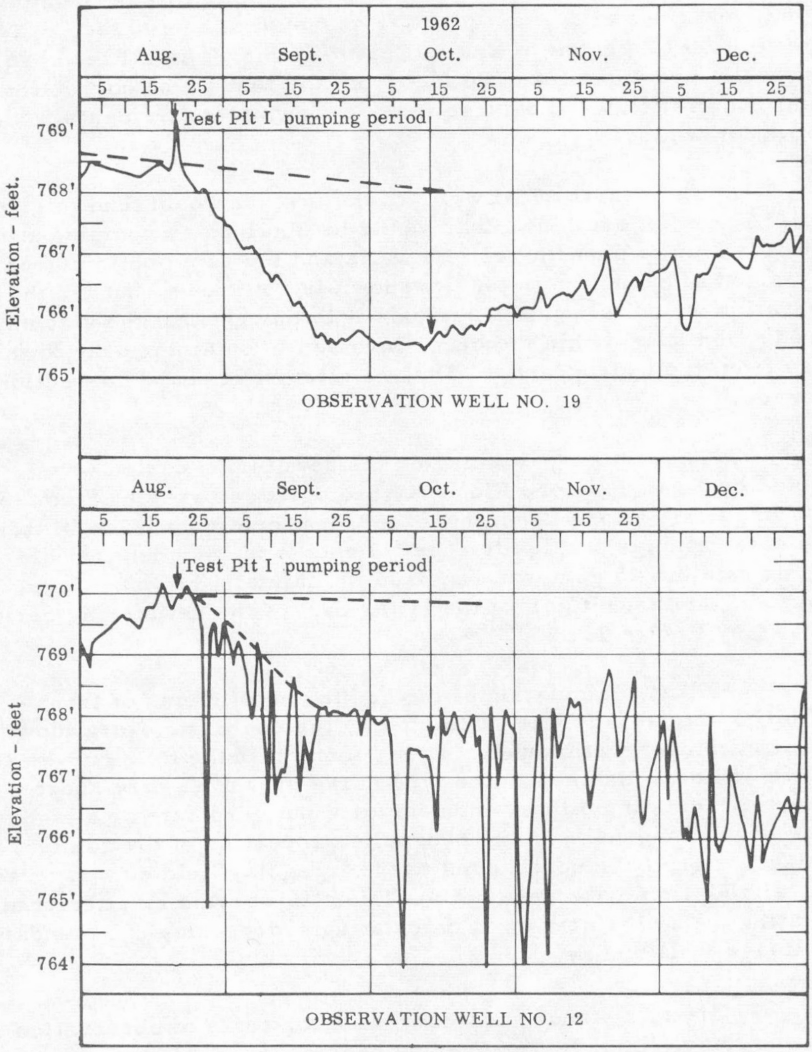


Figure 14. Hydrographs of Observation Wells No. 12 and No. 19

The effect of partial penetration of the test pit is minimized because of the distance between the observation wells and the test pit. Butler (1957, p. 162) pointed out that the effect of top-partial penetration is generally negligible beyond a radius of

$X = 2m \sqrt{Pv/Ph}$ where X is the distance from the observation well to the pumped well, m is the thickness of the aquifer, and Pv and Ph are, respectively, vertical and horizontal permeability of the aquifer. For the conditions at the test pit m is assumed to be 100 feet, while the ratio Pv/Ph is equal to or smaller than unity. As all the observation wells are more than 200 feet from the test pit no correction for partial penetration was required for drawdown measurements in the observation wells.

The non-vertical walls of the test pit would tend to give smaller values of drawdown than would be obtained by pumping at the same rate from a pit with vertical walls and the same bottom area. The amount of reduction in the drawdown is not known, but for the observation wells used in the analysis it is thought that the amount of the reduction falls within the error involved in obtaining drawdown values from the hydrographs. Thus no attempt at shape correction was made.

The pumping rate during the dewatering period was reported by F.W. Render, Floodway geologist, as ranging from 150 to 200 imperial gallons per minute (personal communication, 1963). In the calculation of the aquifer coefficients it is assumed that a pumping rate of 200 gpm was required to maintain the water level at the base of the test pit during the latter part of the dewatering period, as shown by Figure 12.

First, calculation of the aquifer coefficients of transmissibility and storage was attempted by the use of time-drawdown data for each observation well. It was found that the field curves so obtained could be matched to the type curve only over very short segments, indicating either a non-uniform pumping rate or a high degree of inhomogeneity in the hydraulic properties of the aquifer. Both factors probably contributed to the irregular field curves obtained. The available field data were not sufficient to warrant an attempt at separating the two effects or at calculating hydrogeological boundaries from image-well theory.

Next, distance-drawdown data for pairs of observation wells at various times were superimposed on the Theis type curve and appropriate match points were selected for each curve (see Figs. 15 and 16). Aquifer coefficients were calculated from the match point data by the Theis non-equilibrium equation (Tables II and III). Two observation wells are the minimum number required for the distance-drawdown method and the use of data from only two wells precluded the interpretation of boundary effects. However, it is thought that the average values for the aquifer coefficients of transmissibility and storage, based on a number of determinations, are satisfactory for use in the constant-head drain equation.

Distance-drawdown data from the observation wells Nos. 12 and 19 (on a line parallel to the hydraulic gradient) gave average values of $T = 30,800$ gpd/ft and $S = 3.9 \times 10^{-3}$ (see Fig. 15). Values of $T = 17,100$ gpd/ft and $S = 5.5 \times 10^{-3}$ were obtained from the distance-drawdown data for observation wells Nos. 11 and 21 (see Fig. 17). These two wells are aligned approximately at right angles to the hydraulic gradient. This fact, along with inhomogeneity of the aquifer is presumed responsible for the apparent variation in aquifer parameters.

Values of $T = 30,000$ gpd/ft and $S = 4 \times 10^{-3}$ were used in the constant-head drain equation for the calculation of the time factor.

From equations (2) and (3) it may be seen that log Δ will vary with log $D(u)h$ and log u^2 will vary with log x^2/t. Values of $D(u)h$ and u^2 are given in tabular form by Stallman (in Ferris et al., 1962, p. 127), from which a logarithmic type curve for $D(u)h$ versus u^2 was prepared.

Several values of drawdown were calculated from equations (2) and (3) and plotted logarithmically versus X^2/t. A predicted distance-drawdown curve, corresponding to an assigned time, was obtained by superposing the calculated values on the type curve and tracing an equivalent curve through the plotted points. Finally the predicted-drawdown versus distance curves were compared with the curves obtained from the analog model, in order to arrive at an estimate of the time necessary to reach equilibrium.

For the area west of Birds Hill, where a maximum change in drain stage of 50 feet is anticipated, the analog-model curve falls between predicted-drawdown curves corresponding to $t = 10$ days and $t = 100$ days (see Fig. 17). It may be concluded from this that the time factor implied in the model curve is of the order of 60 days. The effect of the zero-drawdown boundary at the Red River is shown by the rapid increase in the slope of the model curve beyond values of $X^2 = 10^8$ (see Fig. 17). If Red River were not treated as a zero-drawdown boundary the model curve would probably be similar in slope to the curves based on the drain formula; it would still remain situated between the two predicted-drawdown curves.

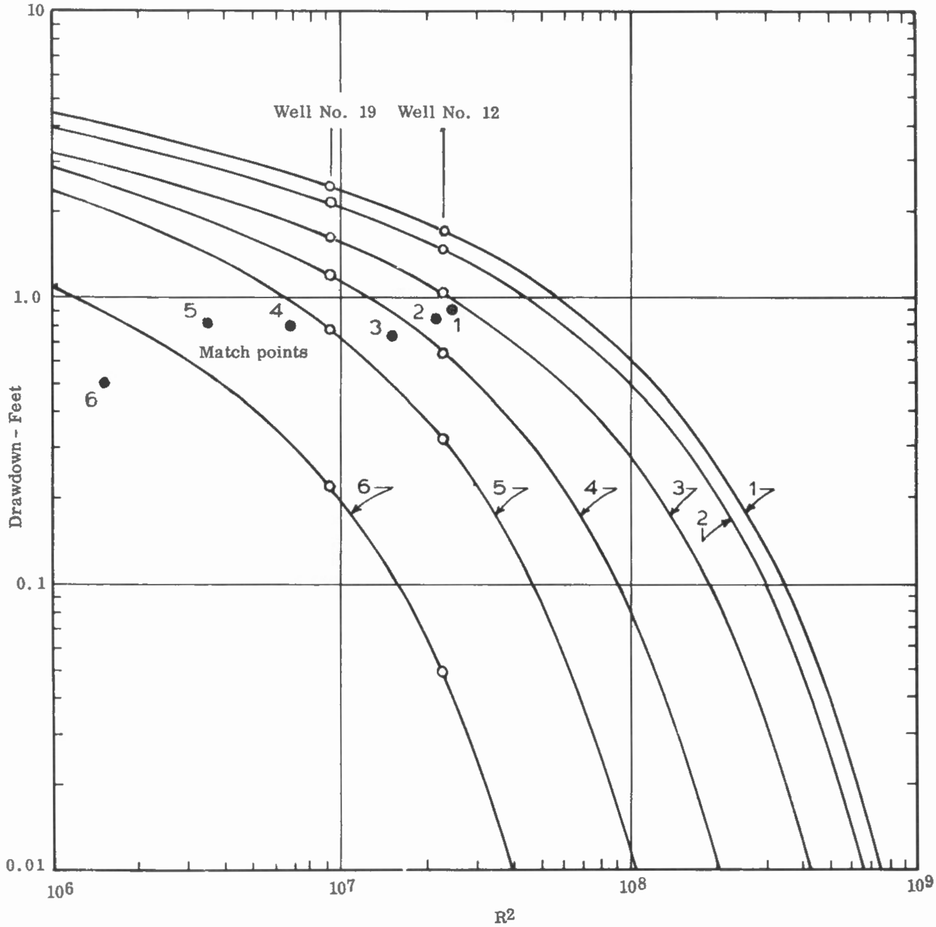


Figure 15. Red River Floodway, Test Pit No. 1, distance-drawdown data, Wells No. 19 and No. 12

Curve	Date	W(u)	u	Δ feet	R^2	t days
Curve 1	Sept. 20	1.0	0.1	0.92	2.8×10^7	30
Curve 2	Sept. 15	1.0	0.1	0.86	2.1×10^7	25
Curve 3	Sept. 10	1.0	0.1	0.74	1.5×10^7	20
Curve 4	Sept. 5	1.0	0.1	0.80	6.6×10^6	15
Curve 5	Aug. 31	1.0	0.1	0.82	3.4×10^6	10
Curve 6	Aug. 25	1.0	0.1	0.50	1.5×10^6	4

$$T = \frac{114.6\ Q.\ W(u)}{\Delta}$$

$$S = \frac{T.u.t.}{1.56\ R^2}$$

Q = 200 I G.P.M. (approx.)
Δ = drawdown, feet
t = time - days
R = distance - feet

Curve 1. T = 114.6 x 200/0.92 = 24,900 gpd/ft.
Curve 2. T = 114.6 x 200/0.86 = 26,600 gpd/ft.
Curve 3. T = 114.6 x 200/0.74 = 31,000 gpd/ft.
Curve 4. T = 114.6 x 200/0.80 = 28,600 gpd/ft.
Curve 5. T = 114.6 x 200/0.82 = 28,000 gpd/ft.
Curve 6. T = 114.6 x 200/0.50 = 45,800 gpd/ft.

$T_{avg.}$ = 30,800 gpd/ft.

Curve 1. $S = 24{,}900 \times 0.1 \times 30/1.56 \times 2.8 \times 10^7 = 1.65 \times 10^{-3}$
Curve 2. $S = 26{,}600 \times 0.1 \times 25/1.56 \times 2.1 \times 10^7 = 2.03 \times 10^{-3}$
Curve 3. $S = 31{,}000 \times 0.1 \times 20/1.56 \times 1.5 \times 10^7 = 2.65 \times 10^{-3}$
Curve 4. $S = 28{,}600 \times 0.1 \times 15/1.56 \times 6.6 \times 10^6 = 4.18 \times 10^{-3}$
Curve 5. $S = 28{,}000 \times 0.1 \times 10/1.56 \times 3.4 \times 10^6 = 5.30 \times 10^{-3}$
Curve 6. $S = 45{,}800 \times 0.1 \times 4/1.56 \times 1.5 \times 10^6 = 7.82 \times 10^{-3}$

$S_{avg.} = 3.9 \times 10^{-3}$

Table 2. Match-point data and calculation of coefficients of transmissibility and storage from Observation Wells Nos. 19 and 21

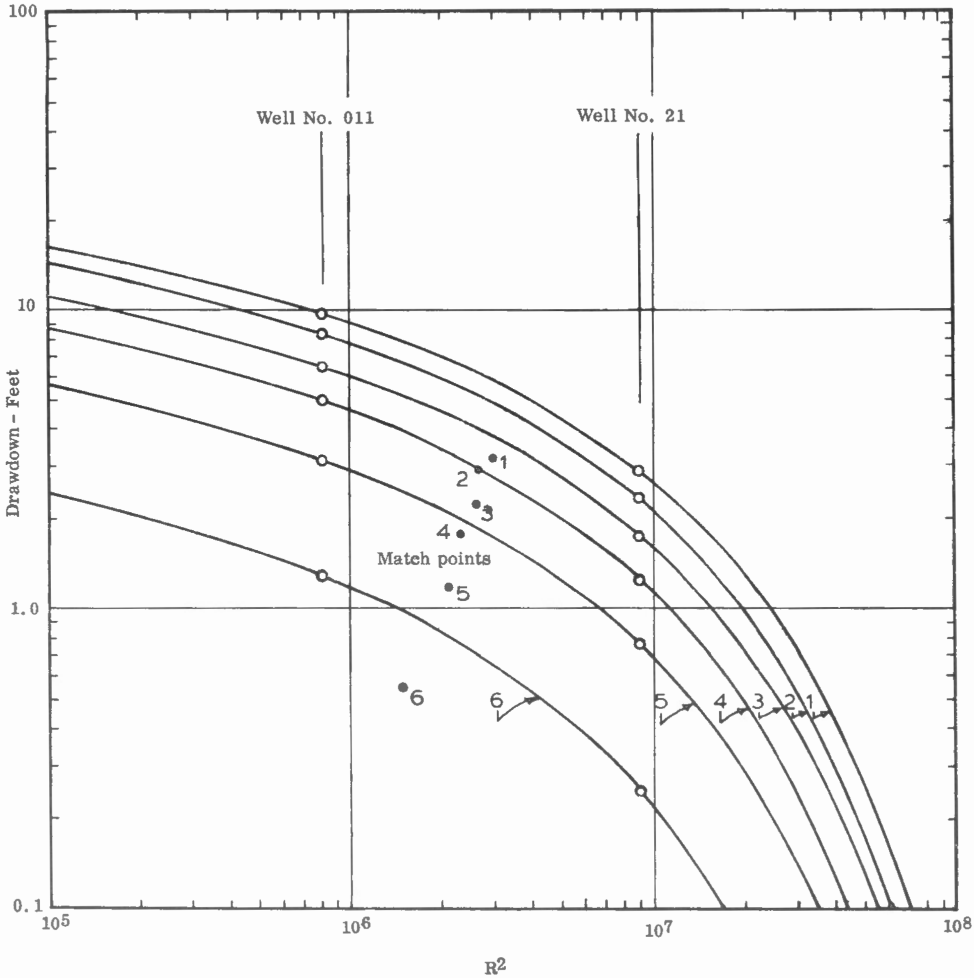


Figure 16. Red River Floodway, Test Pit No. 1, distance-drawdown data, Wells No. 011 and No. 21

Curve	Date	W(u)	u	$\triangle$ feet	R^2	t days
Curve 1	Sept. 20	1.0	0.1	3.2	3×10^6	30
Curve 2	Sept. 15	1.0	0.1	2.9	2.7×10^6	25
Curve 3	Sept. 10	1.0	0.1	2.25	2.65×10^6	20
Curve 4	Sept. 5	1.0	0.1	1.80	2.30×10^6	15
Curve 5	Aug. 31	1.0	0.1	1.20	2.10×10^6	10
Curve 6	Aug. 25	1.0	0.1	0.55	1.50×10^6	4

$$T = \frac{114.6\ Q.\ W(u)}{\triangle}$$

$$S = \frac{T.u.t.}{1.56\ R^2}$$

Q = 200 I. G. P. M. (approx.)
$\triangle$ = drawdown - feet
t = time - days
R = distance - feet

Curve 1. T = 114.6 x 200 x 1.0/3.2 = 7150 gpd/ft.
Curve 2. T = 114.6 x 200 x 1.0/2.9 = 7900 gpd/ft.
Curve 3. T = 114.6 x 200 x 1.0/2.25 = 10,200 gpd/ft.
Curve 4. T = 114.6 x 200 x 1.0/1.80 = 12,700 gpd/ft.
Curve 5. T = 114.6 x 200 x 1.0/1.20 = 19,000 gpd/ft.
Curve 6. T = 114.6 x 200 x 1.0/0.55 = 45,600 gpd/ft.

Tavg = 17,100 gpd/ft

Curve 1. S = 7150 x 0.1 x 30/1.56 x 3 x 10^6 = 4.6 x 10^{-3}
Curve 2. S = 7900 x 0.1 x 25/1.56 x 2.7 x 10^6 = 4.7 x 10^{-3}
Curve 3. S = 10,200 x 0.1 x 20/1.56 x 2.65 x 10^6 = 4.9 x 10^{-3}
Curve 4. S = 12,700 x 0.1 x 15/1.56 x 2.30 x 10^6 = 5.3 x 10^{-3}
Curve 5. S = 19,100 x 0.1 x 10/1.56 x 2.10 x 10^6 = 5.8 x 10^{-3}
Curve 6. S = 45,600 x 0.1 x 4/1.56 x 1.5 x 10^6 = 7.8 x 10^{-3}

Savg. = 5.5 x 10^{-3}

Table 3. Match-point data and calculation of coefficients of transmissibility and storage from Observation Wells Nos. 011 and 21

Similarly Figure 18 shows a comparison between the analog-model drawdown curve and predicted-drawdown curves for the area west of Test Pit I. In this area the anticipated change in stage is 25 feet and the equilibrium condition shown by the model curve is estimated to occur in approximately 400 days.

No time estimate was calculated for the analog-model drawdown curve for the area east of Birds Hill. This is an area of water-table conditions affected by recharge and the data are not amenable to analysis by the constant-head drain formula.

Groundwater Discharge into the Floodway

Groundwater discharging into the Floodway will be derived, by upward leakage, from the artesian aquifer in the underlying limestone bedrock, and from the water-table aquifer in the Birds Hill area. Data available are not sufficient to evaluate the contribution to discharge by the water-table aquifer. However, the following calculations, based on the influx of groundwater into Test Pit I, are assumed to give discharge values of the right order of magnitude to accommodate the combined discharge from both aquifers.

If an average discharge of 175 gallons of water per minute entered the test pit during the later part of the dewatering period, then a basic discharge rate of 0.55 gpm/linear foot of the test pit is obtained by dividing the total influx by the length of the base of the test pit.

The arrangement of the analog model assumes that the drain completely penetrates the aquifer, in which case groundwater entering the drain will be derived from lateral flow through the aquifer mainly from the east side (up-gradient) of the drain. In this case the amount of water entering the drain would be controlled by the transmissibility of the aquifer and the magnitude of the hydraulic gradient.

Therefore, an estimate of groundwater discharge was made for 1-mile segments of the Floodway, using average gradients that would control groundwater flow (Table IV, column 4). The groundwater gradients were calculated by dividing the head difference between the piezometric surface and the base of the Floodway by the vertical distance between the base of the Floodway and the top of the bedrock (Table IV, columns 2 and 3).

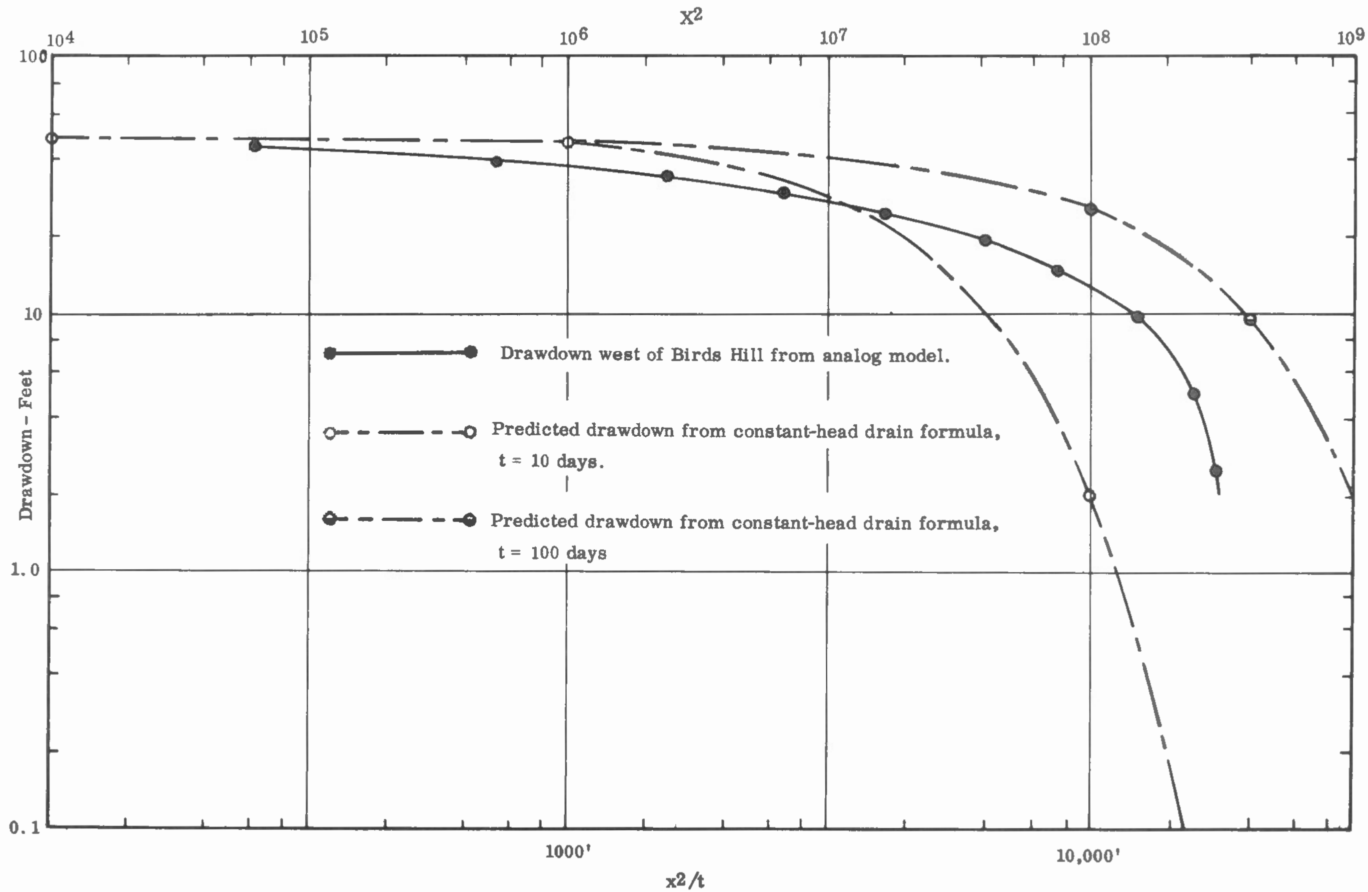


Figure 17. Comparison of distance-drawdown curves for area west of Birds Hill

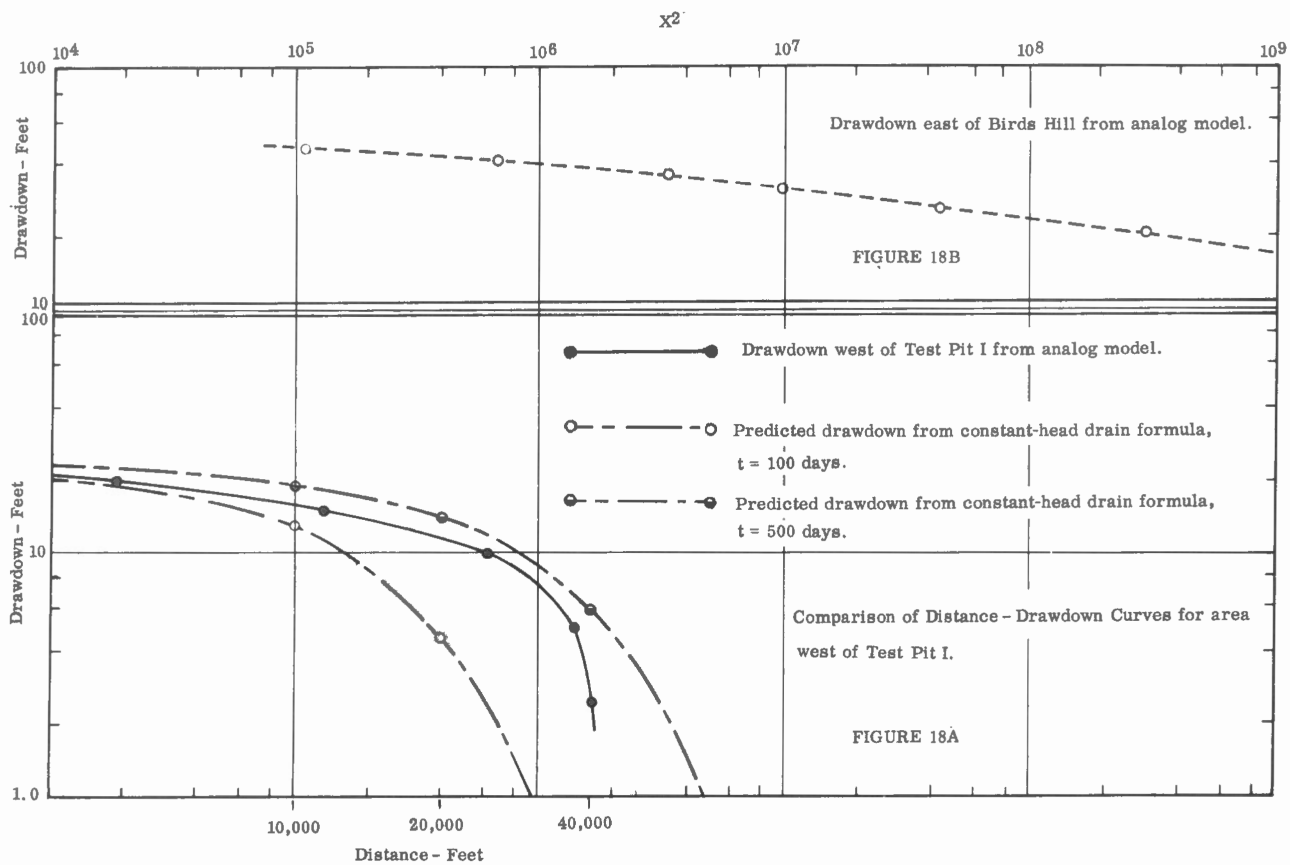


Figure 18 A. Comparison of distance-drawdown curves for area west of Test Pit No. 1
Figure 18 B. Drawdown east of Birds Hill from analog model

At Test Pit I the basic discharge rate of 0.55 gpm/ft was obtained under a gradient of 0.65 ft/ft. Thus anticipated discharge for any 1-mile segment of the Floodway is given by the expression

$$Q_{line}\ (\text{gpm/mile}) = \frac{\text{average gradient}}{0.65} \times 0.55 \times 5{,}280.$$

The values resulting from the calculations are shown in Table IV, column 6, amounting to a total discharge of 79,565 gpm or 212 cfs.

Flow of water into the test pit indicated, on the other hand, that upward flow of groundwater from the bedrock aquifer toward the base of the Floodway is the most likely condition to occur. In this case the amount of discharge will be a function of the vertical permeability of the material overlying the bedrock, as well as of the width of the Floodway channel. Using a discharge value of 0.0055 gpm/ft^2 (obtained by dividing the basic discharge value of 0.55 gpm/ft by the width of the pit), a second set of discharge values was calculated from the expression:

$$Q_{area}\ (\text{gpm/mile}) = \frac{\text{average gradient}}{0.65} \times 0.0055 \times 5{,}280 \times \text{width of channel}$$

(widths of the channel are given in Table IV, column 5). The results of the calculations, shown in Table IV, column 7, give a total predicted discharge of 391,173 gpm or 1,044 cfs for the entire channel.

Actual discharge quantities immediately after completion of the construction will probably lie somewhere between the values of Q_{line} and Q_{area}.

Discharge quantities will decrease with time because of continuously decreasing gradients resulting from the decline in artesian head. Stallman (in Ferris et al., 1962) gives the following equation for calculating quantities of discharge from a drain at any time after the change in stage:

$$Q_b = 2.15 \times 10^{-3} \times \Delta_o \times \sqrt{TS/t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which Q_b = base flow of the drain (gpm/ft)
Δ_o = change in drain stage (feet)
T = coefficient of transmissibility (gpm/ft)
S = coefficient of storage
t = time (days)

At time t = 1 day for Δ_o = 25 feet the amount of discharge per foot of channel calculated from equation (4) is 0.59 gpm/ft, in fair agreement with the amount of 0.55 gpm/ft derived from the test-pit dewatering data.

Table IV

Estimated Groundwater Discharge for 1-mile Segments of the Red River Floodway

1	2	3	4	5	6	7
Mile No.	Average Head Difference (feet)	Average Vertical Distance from Grade to Bedrock (feet)	Average Gradient	Width of Channel (in feet)	Q line Discharge in gpm/mile	Q area Discharge in gpm/mile
(south) 1	0	70	-		-	-
2	4	70	0.057		255	1,375
3	10	80	.125		559	3,016
4	9	60	.150		670	3,619
5	6	65	.092		411	2,220
6	8	60	.133		594	3,209
7	11	38	.290		1,295	6,997
8	10	45	.222		992	5,356
9	6	30	.200		894	4,825
10	24	38	.630	540	2,815	15,200
11	24	37	.650		2,904	15,683
12	25	38	.658		2,940	15,876
13	26	37	.704		3,145	16,985
14	26	24	1.08		4,825	26,057
15	26	28	.929		4,151	22,414
16	27	15	1.80		8,042	43,429
17	28	20	1.40		6,255	33,778
18	26	15	1.73		7,730	41,740
19	25	23	1.09		4,870	26,298
20	35	18	1.94	420	8,668	36,406
21	28	32	.875		3,910	14,856
22	18	31	.580		2,591	9,847
23	14	31	.451		2,015	7,657
24	8	26	.308		1,376	5,229
25	8	25	.320	380	1,430	5,433
26	10	24	.416		1,859	7,063
27	10	29	.345		1,541	5,857
28	12	23	.522		2,332	8,863
29	3	27	.111		496	1,885
(north) 30	0	17	-		-	-
				Total:	79,565 (212 cfs)	391,173 (1,044 cfs)
				After 400 days - Total:	4,339 (11.6 cfs)	21,336 (57 cfs)

At the end of 400 days, total predicted-discharge values are:

Q_{line} = 4,339 gpm or 11.6 cfs,

Q_{area}= 21,336 gpm or 57 cfs.

Discharge quantities may tend to increase, on the other hand, as a result of further opening of fractures by erosion. The magnitude of this effect can not be evaluated at the present time.

Both the effect of the Floodway on the regional piezometric surface and the problem of influx of groundwater into the channel could be restricted mainly to the construction period. After completion of the Floodway, maintenance of a fixed water level in the channel would help to establish a new equilibrium position of the piezometric surface. Complete recovery is impossible because of quantities of groundwater removed from storage during the construction period, and because of unavoidable discharge through evaporation and leakage from the channel.

Conclusions

1. Groundwater flows are to be anticipated during construction, particularly where the Floodway channel or appurtenant structure excavations come close to bedrock.

2. The analog model assumes that the piezometric surface will be lowered to the grade level of the Floodway and that no head loss occurs between the base of the Floodway and the bedrock aquifer. The drawdown of the piezometric surface, as shown by the model is therefore the maximum possible drawdown that could occur by gravity drainage. Greater drawdowns could only result from pumping from wells or excavations in the bedrock.

3. The average transmissibility value of T = 30,000 gpd/ft, obtained from Test Pit I dewatering data, is appreciably lower than transmissibility values for the bedrock aquifer in the same area, which reflects the influence of the till and lacustrine clay.

4. An admissible hypothesis for the time factor implied in the analog-model solution was obtained by a constant-head drain equation. A value of t of approximately 60 days was adopted for the area west of Birds Hill (drawdown at the channel Δ_o = 50 feet), and of approximately 400 days for the area west of Test Pit I (drawdown at the channel Δ_o = 25 feet).

5. Predicted quantities of groundwater discharge that will be intercepted by the Floodway are between 200 and 1,000 cfs immediately after construction of the channel.

6. Most of the predicted drawdown of the piezometric surface will occur during the construction period.

7. Both the drawdown and discharge effects of the Floodway could be reduced by maintaining a static water level in the channel, if feasible.

Predictions made in this report will be checked by field observations during and after construction of the Floodway. Flow quantities in the channel could be measured by weirs suitably placed along the channel, or by a total-flow measurement at the outlet.

REFERENCES

Butler, S.S.
1957: Engineering hydrology; Prentice Hall Inc., Englewood Cliffs, N.J., 356 pp.

Dobrin, M.B.
1960: Introduction to geophysical prospecting; McGraw-Hill.

Ehrlich, W.A., Poyser, E.A., Pratt, L.E., and Ellis, J.H.
1953: Reconnaissance soil survey of Winnipeg and Morris map sheet areas; Man. Dept. Agriculture, Soils Rept. 5.

Elson, J.A.
1961: Soils of the Lake Agassiz region; Soils in Canada, Roy. Soc. Can., Spec. Pub. 3; Univ. Toronto Press.

Ferris, J.G., Knowles, D.B., Brown, R.H., and Stallman, R.W.
1962: Theory of aquifer tests; U.S. Geol. Surv., Water-Supply Paper 1536-E, pp. 69-174.

de Jong, J.
1962a: Electrische analogie modellen voor het oplossen van geo-hydrologische problemen; Water, vol. 46, No. 4, pp. 43-45.

1962b: Een eenvoudige methode voor het nabootsen van een oneindig potentiallveld; Water, vol. 46, No. 12, pp. 185-186.

Mishtak, J.
(in press): Preliminary soil mechanics aspects of the Red River Floodway; Can. Geotech. J.

Nettleton, L.L.
1940: Geophysical prospecting for oil; McGraw-Hill.

Upham, W.
1895: The glacial Lake Agassiz; U.S. Geol. Surv., Mon. 25.

1910: Birds Hill, an esker near Winnipeg, Manitoba; Bull. Geol. Soc. Amer., vol. 21, July.

van Everdingen, R.O., and Bhattacharyya, B.K.
1963: Data for groundwater model studies; Geol. Surv. Can., Paper 63-12, 31 pp.

Walton, W.C.
1962: Selected analytical methods for well and aquifer evaluation; Illinois State Water Surv., Bull. 49, 81 pp.

ROGER DUHAMEL, F. R. S. C.
QUEEN'S PRINTER AND CONTROLLER OF STATIONERY
OTTAWA, 1964

Price 75 cents Cat. No. M44-64/18