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REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

FINAL FOUNDATION INVESTIGATION

VOLUME II

GROUNDWATER CONDITIONS AND PUMPING TEST RESULTS
PROPOSED CROSSING OF THE RE-ALIGNED WELLAND CANAL
MAIN STREET EAST TUNNEL

WELLAND

ONTARIO

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ABSTRACT

This report forms Volume II of the final foundation investigation report and details the stabilized piezometric groundwater conditions at the site of the proposed Main Street East Tunnel Crossing of the Re-aligned Welland Canal in Welland, Ontario. Also presented are the results of the full scale pumping test carried out in conjunction with the investigation.

The weathered portion of the bedrock underlying the site forms a confined aquifer in which the stabilized piezometric water level is at elevation 576. Due to upward seepage of groundwater from the weathered bedrock the piezometric water levels in the till and lower silt deposits which overlie the bedrock are also at about elevation 576. The piezometric water levels in the clay and upper silts are independent of the piezometric water level in the aquifer and are at present ground surface (elevation 600 to 605). The piezometric pressures in the clays and upper silts decrease slightly with depth indicating that there is some downward drainage into the underlying aquifer.

Based on the results of the full scale pumping test carried out at the site the equivalent coefficient of permeability of the 5 foot thick weathered bedrock aquifer is about 6×10^{-2} cm/sec. The effects on the overburden strata of pumping from the aquifer are discussed in the report.

INTRODUCTION

The results of a detailed investigation of the ground-water conditions at the site of the proposed Main Street East Tunnel Crossing of the re-aligned Welland Canal in Welland, Ontario are presented in this report. Included are the stabilized ground-water conditions established during the course of the final foundation investigation and the water levels established during the period of the full scale pumping test carried out in conjunction with the investigation. The hydraulic characteristics of the bedrock/till aquifer underlying the site are discussed.

The report forms Volume II of the final foundation investigation report which is presented in 3 volumes:

- Volume I - Soil and Bedrock Conditions
- Volume II - Groundwater Conditions and Pumping Test Results
- Volume III - Foundations and Excavations.

DESCRIPTION OF PUMPING TEST

General

Based on the results of the preliminary foundation investigation carried out at the site (our report 66134, dated May, 1967) and the results of pumping tests carried out in the Welland area it is known that the bedrock, till and possibly the lower silts underlying the site form an aquifer in which the

groundwater conditions are independent of those in the overlying soil strata. As discussed in the preliminary foundation investigation report the hydrostatic pressure within the aquifer is artesian with respect to the bottom of the proposed tunnel excavation and will cause uplift of the base of deep excavations unless the hydrostatic pressure in the aquifer is relieved. A full scale pumping test was therefore carried out to determine the hydraulic characteristics of the bedrock/till aquifer and to provide a practical demonstration of the effect of pumping on the groundwater level in the aquifer.

Test Well Installation

As shown on the site plan presented on Figure 2-1 the test well was located 50 feet south of the existing Main Street East centreline and some 70 feet west of the centreline of the proposed re-aligned Welland Canal. The test well was put down between November 30 and December 9, 1967 using a Bucyrus Erie track-mounted percussion drillrig supplied and operated by the F. E. Johnston Drilling Co. Ltd. The well was advanced in 12 inch diameter casing size to a depth of 117.5 feet, the upper surface of the bedrock, using conventional percussion drilling methods. The well was advanced in the bedrock without casing to a depth of 138 feet below ground surface using a standard 12 inch percussion bit. During drilling in the bedrock the casing sank

to a depth of about 119.5 feet. On December 10, 1967 the well was cleaned and partially developed by continuous bailing and the 12 inch casing was withdrawn to a depth of 118.3 feet below ground surface or 10 inches below the upper surface of the bedrock.

The pumping test was carried out using an open hole in the bedrock. Well screens were not installed since, from past experience in the Welland area together with the results of the continuous bailing in the well, it was considered that the quantity of water flowing into the well from the bedrock was such that the efficiency of the well could be decreased by installing a well screen and backfilling the annular space between the screens and the 12 inch diameter hole with filter gravel. Further, from the point of view of future construction, unscreened wells would be much more economical to install. Therefore the test pump was installed in the open hole in the bedrock at a depth of 136.5 feet below ground surface.

A 5 inch vertical turbine test pump capable of pumping at a rate of 300 I.G.P.M. from a depth of 300 feet was used for the test. The test pump was connected to a 5 inch diameter pump column supported at the top by the 12 inch diameter casing. The turbine pump was driven from ground surface by a gasoline powered

engine. The details of the test well are shown diagrammatically on Figure 2-2.

On December 12, 1967, following completion of the well and installation of the test pump, the well was developed by pumping at various rates for a period of 10 hours. This established a constant flow rate at which the actual pumping test could be carried out.

Piezometer Installations

a) Open Tube Piezometers

In order to determine the amount of drawdown achieved by pumping from the weathered bedrock and lower portion of the till open tube 'Casagrande-type' piezometers were installed at various radial distances from the well along two mutually perpendicular lines which intersected at the well. The locations of the piezometers are shown on Figure 2-1.

Because of the numerous piezometers installed in the sampled borings put down to define the soil and bedrock conditions at the site only a few additional piezometer installations were required. Originally it was intended to install piezometers at 8 additional locations, but this number was increased to 10 when difficulties were encountered with the piezometer installations. A detailed log of each installation and a brief account of the

installation procedure is given in Volume I of this report.

During boring operations at piezometer location E-2 the casing broke at a depth of 47 feet and 55 feet of casing were left in the hole. When attempts to recover the broken casing proved unsuccessful this piezometer location was abandoned. In addition, it was found that the leads to both piezometers installed in boreholes S-2 and S-3 (piezometers 52, 53, 54 and 55) and the lead to the piezometer installed in the till at the borehole E-1 location (piezometer 23) were blocked or kinked making the installations inoperative. The leads to both piezometers installed in borehole S-1 (piezometers 50 and 51) and the lead to the piezometer installed in the bedrock in borehole E-1 (piezometer 24) were found to be partially kinked. Although the water levels in these latter installations reacted normally during flushing the partial kinking made it very difficult to insert the cable from an electrical water level sounding device below the kink. Once the cable was inserted for more than a few feet below the kink it was virtually impossible to withdraw the cable.

This blockage or kinking of piezometer leads was probably caused by cobbles and small boulders which are dispersed throughout the overburden deposits. During pre-augering the

cobbles were not augered to the surface and, after withdrawal of the augers, remained in the heavy caved slurry which remained in the boreholes. When the holes were cased the cobbles were pushed aside into the slurry and remained in the annular space between the casing and the walls of the pre-augered holes. During withdrawal of the casing after the piezometers were installed these cobbles moved back into the hole and kinked the piezometer tubing. This problem was overcome during the latter stages of the investigation by placing black iron pipe around the piezometer tubing.

Due to the large number of inoperative piezometers near the well it was decided to install piezometers at 2 additional locations. These locations are identified as boreholes E-2A and S-1A and are located as shown on Figure 2-1.

To facilitate the recording of piezometric water levels during the pumping test all piezometers installed at the site during the preliminary and final foundation investigations were numbered consecutively. The number assigned to each piezometer is shown on the Record of Borehole sheets given in Volume I of this report and are summarized in Appendix A of this volume of the report.

b) "No Volume Change" Piezometers

In addition to the conventional piezometers which were installed primarily in the till and bedrock four (4) "no volume change" piezometers fitted with a steel diaphragm the deflection of which is gauged by transducers to monitor changes in piezometric pressure (numbered T.P. 101 to T.P. 104 inclusive) were installed in the clayey silt stratum underlying the site. The purpose of these piezometers was to determine any fluctuations in piezometric pressure which occurred in the relatively impervious clayey overburden during pumping from the underlying aquifer.

The location of the transducer piezometers together with a sketch of each piezometer showing the installation details is presented on Figure 2-3.

The piezometers were installed by augering a 4.5 inch diameter hole to a pre-selected depth using a continuous flight power auger supplied and operated by the F. E. Johnston Drilling Co. Ltd. The transducer piezometer, attached to "E" and "A" rods, was then pushed hydraulically from the bottom of the auger hole. The drill rods were left in position to facilitate recovery of the piezometers following the completion of the pumping test.

Pumping Test Procedure

The well was developed by pumping at various rates for a period of 10 hours on December 12, 1967. On December 13, 1967, following a 20 hour recovery period which allowed the groundwater to re-stabilize, the pumping test was begun.

During the first test the well was pumped at a constant rate of 51 I.G.P.M. and the water levels in the well and piezometers were recorded at frequent intervals. After 6 1/2 hours of continuous pumping, the water levels had effectively stabilized and the pump was stopped and the aquifer allowed to re-charge. During this re-charge period the rate of recovery in selected piezometers was measured.

On December 14, 1967, pumping was begun at a rate of 125 I.G.P.M. This rate was maintained for a period of 5 1/2 hours when the rate of pumping was decreased to 101 I.G.P.M. This decrease in the rate of pumping was necessitated by the fact that the water level in the well was approaching the depth of the pump intake. Pumping was continued at a rate of 101 I.G.P.M. for a period of 59 hours at which time the gasoline powered engine driving the pump broke down. Pumping was resumed at a rate of 101 I.G.P.M. at 13:00 hours on December 17, 1967 and maintained at that rate for a period of 27 hours.

Beginning at 16:00 hours on December 18, 1967, the rate of pumping was increased in stages to 200 I.G.P.M. at which rate the capacity of the well was exceeded. The pumping rate was therefore decreased in stages to 150 I.G.P.M., the maximum rate at which pumping could be maintained without "breaking suction" in the well. This rate of pumping was maintained for a period of 20 1/2 hours; then pumping was stopped.

Following each start of pumping or change in pumping rate the water levels in the piezometers were recorded and readings continued at frequent intervals. However, during long periods of pumping at a constant rate the interval between recordings was increased. The rate of recovery in the piezometers was established in a similar manner after pumping was stopped on December 19, 1967.

Following completion of the pumping test the test pump and all casing was removed and the well backfilled.

Supplementary Information

During late 1967 and early 1968 a foundation investigation and pumping test was carried out at the site of a proposed combination railroad and highway tunnel crossing of

the re-aligned Welland Canal near Townline Road in Welland, Ontario. This site is located about 13,000 feet from the Main Street East tunnel site and computations based on previous pumping tests in the area indicate that at this radial distance the drawdown of the piezometric water level in the aquifer could be as much as 4 feet. Therefore, through the co-ordination of the Department of Highways, Ontario, the pumping tests at the two sites were carried out at different times and the drawdown of the piezometric water level in the aquifer was established at each site during pumping at the other site.

GROUNDWATER CONDITIONS

General

The piezometric groundwater levels in the several soil strata encountered at the site were established by means of sealed piezometers installed during the course of the preliminary and final foundation investigations. The stabilized water levels in the piezometers were recorded at frequent intervals during the period of the field work for the final foundation investigation prior to the beginning of the pumping test. During this period all of the piezometers were flushed and their response noted. The installation details together with the final set of water level readings obtained prior to

the beginning of pumping from the well are presented on the Record of Borehole sheets included in Volume I of the final foundation investigation report.

The stabilized water levels established in the piezometers are summarized on Figure 2-4. From this figure it can be seen that the stabilized piezometric water level in the bedrock, till and lower silt deposit is consistently at about elevation 576 or some 25 to 30 feet below the existing ground surface. During the period of the field investigation it was observed that daily groundwater level fluctuations of as much as 6 inches could occur in the piezometers. However, there does not appear to be any appreciable seasonal piezometric groundwater level fluctuation in the lower silt, till and bedrock as the average piezometric level remained constant between October 1967 and January 1968 (excluding the period of the pumping test) and was the same as the average piezometric level established in March 1967.

The stabilized piezometric groundwater levels established in the clayey silt, silty clay and upper silt strata are consistently higher than the levels established in the lower silt, till and bedrock. Further, the piezometric groundwater level rises as the elevation of the piezometer tip

increases. A linear extrapolation of the range established for the hydrostatic or piezometric pressure in these upper overburden deposits indicates the presence of a slightly artesian condition with respect to the existing ground surface although no artesian condition was observed at ground surface. However, the presence of ponded surface water in local depressions and the fact that the ground surface was generally saturated indicates that the groundwater level is at or very close to ground surface.

A hydrostatic and non-artesian pore pressure distribution in the clays and upper silts (relative to an average ground surface elevation of about 602) is presented on Figure 2-4. As shown on this figure the majority of the pore pressures established at the piezometer tip elevations are less than the hydrostatic pressure.

It is possible that, with depth, the pore pressures in the clays and upper silts are reduced by underdrainage into the underlying till and bedrock. A possible reduced linear and non-artesian pore pressure distribution which is the average distribution for the range of measured hydrostatic pressures in the clays and upper silts is presented on Figure 2-4.

Although there appears to be some downward migration

of groundwater from the clays and upper silts into the independent piezometric groundwater table in the underlying lower silt. till and bedrock this minor downward movement of water is not considered to be sufficient to recharge the bedrock/till aquifer. The bedrock is considered to be in direct hydraulic communication with some large external groundwater source.

Chemical Composition

Groundwater samples from both the overburden and the bedrock were obtained during the course of the final foundation investigation to determine the chemical composition of the water. The groundwater samples from the overburden were obtained by bailing from open auger holes prior to the beginning of wash boring operations. These samples are considered to be uncontaminated by either surface water or wash water. Groundwater samples from the bedrock were obtained from the discharge from the pumped well during the period of the pumping test.

Chemical analysis of the groundwater samples were carried out by the Chemical Section, Materials and Testing Division of the Department of Highways, Ontario. The results of the chemical analyses were provided to us by the Department, memoranda dated January 16 and 23, 1968, and are presented in Appendix B of this report.

The results of the chemical analyses indicate the groundwater in the upper overburden deposits contains generally 500 to 700 parts per million of sulphates and the sulphates content increases with depth. The samples of groundwater from the bedrock contain 2,000 to 2,100 parts per million of sulphates. The pH of the water from the bedrock is 7.4 indicating that the water is almost neutral, the slight alkalinity being caused by bi-carbonates, compounds which are commonly present in water.

PUMPING TEST RESULTS

General

The piezometric groundwater level in the bedrock, till and lower silt deposits underlying the Main Street East tunnel site is apparently independent of the piezometric groundwater level in the overlying clays and upper silts. These lower waterbearing zones (i.e. bedrock, till and lower silt) may therefore be considered to form an aquifer in which the piezometric level is controlled by some large external groundwater source. However, it is probable that it is only through the bedrock that any appreciable horizontal groundwater movement occurs and that the piezometric pressure in the till and lower silt is affected by upward seepage of water

from the bedrock. This groundwater movement is arrested by the relatively impervious silty clay or clayey silt strata (vertical permeability $K_v = 10^{-5}$ to 10^{-6} cm/sec.) which overlie the lower silt and till. It further appears that the majority of the horizontal water transport occurs through the upper weathered, fractured and permeable portion of the bedrock. Consequently, it is considered that the weathered portion of the bedrock forms a confined aquifer in hydraulic communication with an external groundwater source while the till and lower silt form a "quasi-aquifer" which merely reflects the piezometric water level in the underlying aquifer.

A pumping test carried out in a confined aquifer may be analysed using either of two methods:

- i) Non-steady state condition;
- ii) steady state condition.

The non-steady state condition occurs during the period when the piezometric water level around a pumped well is lowering whereas the steady state condition assumes the water levels in piezometers to have stabilized. Although, true equilibrium may require extremely long periods of pumping practical results usually can be obtained by pumping at a steady rate for periods that range from a few hours to a few days, depending largely on the permeability of the aquifer.

For the case of the pumping test carried out at the Main Street East tunnel site the piezometric water levels had effectively stabilized during each pumping period and therefore the solution for a steady state condition was used for the analysis of the pumping test results.

The basic equation for the steady state solution of a confined aquifer pumping test is derived from the Darcy equation:

$$v = ki \text{ ----- (1)}$$

where v = velocity of flow
 k = coefficient of permeability of the aquifer
 i = hydraulic gradient

Based on equation 1 and the Dupuit assumption for the hydraulic gradient, an equation for the flow into a well may be written in terms of known or measured quantities:

$$Q = \frac{2\pi kD(h_2 - h_1)}{\ln(r_2/r_1)} \text{ ----- (2) *}$$

where Q = quantity of water pumped from well (cu.ft./min.)
 k = coefficient of permeability of aquifer (ft./min.)
 D = thickness of aquifer (ft.)
 h₂ = drawdown (feet) at a radius r₂ (feet) from the well
 h₁ = drawdown (feet) at a radius r₁ (feet) from the well

* Leonards, G.A., editor, 1962. Foundation Engineering (New York McGraw Hill.)

Equation (2) may be rewritten as:

$$Q = \frac{2\pi k D (H-h)}{\ln (R/r)} \quad \text{----- (3)}$$

where Q, k and D are as in equation (2) and

- H = Piezometric water level in aquifer prior to pumping
- h = stabilized piezometric water level in aquifer at a radial distance r from the well during pumping
- R = radius of influence of the well (the radial distance from the well at which no drawdown occurs).

For the analysis of a pumping test the rate of pumping is known, the thickness of the aquifer is known (based on borings) or assumed and the drawdown is measured in piezometers installed at various radial distances from the well. Therefore the coefficient of permeability of the aquifer may be computed.

It should be noted that equations (2) and (3) are used for both the analysis of a pumping test and the design of well systems. Therefore, for design, the thickness of aquifer used in the analysis of the pumping test is not critical provided the same relative values of aquifer thickness and coefficient of permeability are used in both cases. Should the assumed aquifer thickness be doubled the computed coefficient of permeability of the aquifer would be halved and there would be no net effect on the computed drawdown for a pumped well(s).

Effect of Pumping

a) In Bedrock

The detailed results showing the effect of the full scale pumping test on the piezometric water levels in the bedrock are presented in the Figures. Schematic sections showing the stabilized drawdown achieved at various pumping rates along sections taken parallel and perpendicular to the Main Street East centreline are given on Figure 2-5 and semi-logarithmic distance-drawdown curves are presented on Figures 2-6 to 2-8 inclusive. The observed piezometric groundwater elevations in the bedrock during the period of the pumping test are shown on Figures 2-12 to 2-15 inclusive and the rate of drawdown and recovery in each bedrock piezometer is shown on Figures 2-18 to 2-21, 2-24 to 2-27, 2-30 to 2-33 and 2-36 to 2-39.

The results of the pumping test indicate that the cone of depression of the piezometric water level in the aquifer was, in general, similar in all directions and the radius of influence of the well for the average drawdown conditions was about 3,000 feet. However, the radius of influence of the well would probably increase during a prolonged pumping period.

From the results of pressure packer permeability tests (Volume I, pages 31 and 32) it appears that the upper 3 to 10 feet

thick weathered and fractured portion of the bedrock forms a permeable zone in which the majority of horizontal ground-water movement could be expected to occur. Thus the equivalent coefficients of permeability of the aquifer computed from the pumping test result were based on a 5 foot aquifer thickness as this appears to be the average thickness of weathered rock across the site. The permeability calculated from the pumping test results represents an equivalent coefficient of permeability of the entire fractured bedrock aquifer rather than a coefficient of permeability of the sound rock matrix since the majority of the water probably flowed to the well through fractures or fissures rather than through sound rock.

The equivalent coefficient of permeability of the weathered bedrock aquifer computed from the pumping test results varies from about 4×10^{-2} to 9×10^{-2} cm/sec. with an average value of about 6×10^{-2} cm/sec. The variation on the equivalent coefficient of permeability is probably due to variations in the actual aquifer thickness at the piezometer locations.

The equivalent coefficient of permeability of the aquifer computed from the pumping test results indicates that the weathered portion of the bedrock is about 100 times more permeable than is indicated by the pressure packer permeability test results (computed equivalent coefficient of permeability of 1×10^{-3} to

5×10^{-4} cm/sec.). This apparent inconsistency may be due to the presence of numerous gypsum inclusions within the bedrock. During coring operations in boreholes the gypsum was ground to a fine powder or slurry which was carried by the drilling water into the fractures within the bedrock. During pressure packer testing in the bedrock the gypsum was for a short period forced under pressure into the fractures. This reduced or stopped water flow through some small fractures thus reducing the computed equivalent coefficient of permeability. This effect did not occur in the well since, during the course of well development and the pumping test, water was withdrawn from the well thus flushing the gypsum from the fractures.

As was previously noted the cone of depression of the piezometric water level in the aquifer was, in general, similar in all directions. However, in a area slightly east of the pumped well, boreholes T-124 and T-123, the observed drawdown was considerably less than would be anticipated from the other results. At the borehole T-123 location 3 piezometers were installed in the bedrock. Piezometers numbered 34 and 35 (installed respectively 24 and 45 feet below bedrock surface) indicated drawdown of the order anticipated from adjacent piezometers whereas piezometer number 33 (installed 9 feet below bedrock

surface) indicated relatively little drawdown. Both piezometer number 29 (installed in borehole T-124) and piezometer 33 were flushed prior to and following the pumping test and were observed to respond properly although following the test the response of piezometer 29 was slow. It appears, however, that both piezometers were functioning during the period of the pumping test and the relatively small drawdown measured in the installations was due to a less permeable zone of weathered bedrock in the area of boreholes T-124 and T-123.

Immediately east of the pumped well (piezometers 24 and 26) appreciable greater drawdowns were measured than would be anticipated from the general trend determined at the site. These large drawdowns could be caused by the presence of a continuous void passing through the piezometer locations and in direct communication with the pumped well. A 2 foot thick void was encountered in borehole E-1 and although no void of this magnitude was encountered in either the well or borehole E-2A connecting thin fissures or voids could exist at these locations.

b) In Till

The detailed results showing the effect of the full scale pumping test on the piezometric water levels in the till are given in the figures. From the distance-drawdown plots

presented on Figures 2-9 to 2-11 inclusive, it can be seen that two distinctly different drawdown conditions were recorded in the till; either the drawdown in the till was the same as the drawdown in the bedrock or the measured piezometric water level in the till was not appreciably effected by the pumping.

The groundwater elevations in the till during the period of the pumping test and rate of drawdown and recovery curves for piezometers in which the stabilized drawdown was about the same as the drawdown in the bedrock are given in the figures. At these piezometer locations one or more of the following facts are noted on the borehole logs:

- (i) A sand and gravel zone was encountered within the generally silty till.
- (ii) Numerous boulders were encountered within the till.
- (iii) The till was found to be relatively thin and the piezometer was therefore installed close to the upper surface of the bedrock.

Each of the above facts pertains only to the vicinity of the actual boring in which it was noted. These facts are not considered to indicate horizontal movement of water through the till to the pumped well. Each fact would however be sufficient to permit vertical downward movement of groundwater from the till into the underlying bedrock aquifer. This downward drainage

could, during the period of the pumping test, result in stabilization of the piezometric groundwater levels in the till and bedrock.

As shown on Figures 2-9 to 2-11 inclusive the observed drawdown in a number of piezometers installed in the till was relatively minor (less than 3 feet at the maximum pumping rate). Based on the grain size distribution curves for the silty till the coefficient of permeability of the silty till is less than about 1×10^{-4} cm/sec. This relatively low permeability is sufficient to allow the transmission of pore pressure through the till but is insufficient to allow appreciable movement of groundwater during the period of the pumping test. Pore pressures within the till could be reduced during pumping to about the piezometric pressure within the bedrock aquifer but little or no change would occur in the volume of water within the till where it is silty, and hence within open tube piezometers. Water levels observed in the piezometers are therefore, not necessarily representative of the lowest piezometric water level in the silty portion of the till.

c) In Lower Silt Deposit

Little or no drawdown was observed in the piezometers installed in the lower silt deposit. This however is to be

expected as the silts overlie the silty till and, as discussed previously, little drawdown was observed in the till. However, as with the till it is probable that some reduction in pore water pressure occurred although there was insufficient ground-water movement within the lower silt deposit to reflect this pressure change in the open tube piezometers.

d) In Clays and Upper Silts

The drawdown achieved in the clays and upper silts by pumping from the underlying bedrock aquifer was monitored during the period of the pumping test by "no volume change" or transducer piezometers. The observed drawdown in the clayey silt stratum during the period of the pumping test (expressed in feet of water) is presented on Figure 2-42. As can be seen from this figure the pore water pressure within the clayey silt stratum was reduced by as much as 6 feet of water during the pumping test. Little correlation between the rate of drawdown and the rate of pumping is apparent and where recharge did occur in the piezometers there was a considerable time lag between the end of pumping and the beginning of recharge. From figure 2-42 it can further be seen that the pore pressure reduction achieved in piezometers T.P. 101 and T.P. 103, installed immediately above a silt to sandy silt zone, was greater than the pore pressure reduction achieved in piezometer T.P. 102

installed beneath the silt zone. This suggests that drainage of the upper portion of the clayey silt is through the sandy silt zones whereas lower portions of the clayey silt and the silty clay drain downward into the underlying till.

The measured drawdown or reduction in pore water pressure in the clays and upper silts is not considered to be sufficiently reliable for quantitative analysis but is considered to indicate that some reduction of piezometric levels may be anticipated in the clays and upper silts during prolonged pumping periods.

Supplementary Information

It is understood that no drawdown of the piezometric water level in the bedrock aquifer was observed at the site of the proposed combination railroad and highway tunnel at Townline Road during the period of the pumping test at the proposed Main Street East tunnel site. Similarly no drawdown of the piezometric water level in the bedrock aquifer was observed at the Main Street East tunnel site during the prolonged pumping period required to dewater a test shaft at the Townline Road tunnel site. However, it is understood that at the Townline Road tunnel site pumping was carried out from a gravel stratum

overlying the bedrock rather than from the bedrock.



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APPENDIX A
INDEX TO PIEZOMETERS

PIEZOMETER NUMBER	BORE HOLE NUMBER	PIEZOMETER LOCATION			STRATUM for which water level recorded	GROUNDWATER LEVEL PROFILE during Pumping Test (Figure)	RATE OF DRAWDOWN & RECOVERY CURVES (Figure)	PIEZOMETER DESTROYED ON
		STATION	OFFSET (feet)	RADIAL DISTANCE from WELL (feet)				
1	T-4	25+22	72.5Rt.	64	Upper silt	*	*	
2	T-4	25+22	72.5Rt.	64	Till	*	*	
3	T-4	25+22	72.5Rt.	64	Bedrock	2-12	2-18, 24, 30, 36	
4	T-125	24+03	53 Lt.	206	Till	**	**	
5	T-125	24+03	53 Lt.	206	Bedrock	2-12	2-18, 24, 30, 36	
6	T-126	22+64	54 Rt.	318	Lower silt	**	**	
7	T-126	22+64	54 Rt.	318	Till	**	**	
8	T-126	22+64	54 Rt.	318	Bedrock	2-15	2-21, 27, 33, 39	
9	T-3	21+72	46 Lt.	420	Clayey silt	*	*	
10	T-3	21+72	46 Lt.	420	Upper silt	*	*	
11	T-3	21+72	46 Lt.	420	Bedrock	2-15	2-21, 27, 33, 39	
12	T-9	18+93	67 Rt.	690	Clayey silt	*	*	
13	T-9	18+93	67 Rt.	690	Upper silt	*	*	
14	T-9	18+93	67 Rt.	690	Till	**	**	
15	T-107	17+30	48.5Rt.	852	Till	**	**	
16	T-107	17+30	48.5Rt.	852	Bedrock	2-12	2-18, 24, 30, 36	
17	T-2	15+94	53 Rt.	988	Clayey silt	*	*	
18	T-2	15+94	53 Rt.	988	Upper silt	*	*	
19	T-2	15+94	53 Rt.	988	Till	**	**	
20	T-1	11+05	54 Lt.	1,480	Clayey silt	*	*	
21	T-1	11+05	54 Lt.	1,480	Upper silt	*	*	
22	T-1	11+05	54 Lt.	1,480	Till	**	**	
23	E-1	25+91	50 Rt.	9	Till	-	-	Dec. 14/67
24	E-1	25+91	50 Rt.	9	Bedrock	2-13	2-25	
25	PIEZOMETER 25 was not installed							
26	E-2A	26+10	50 Rt.	28	Bedrock	2-13	2-19, 25, 31, 37	
27	T-124	26+48	45 Rt.	67	Upper silt	*	*	
28	T-124	26+48	45 Rt.	67	Till	2-16	2-22, 28, 34, 40	
29	T-124	26+48	45 Rt.	67	Bedrock	2-14	2-20, 26, 32, 38	
30	T-5	28+06	50 Lt.	254	Clayey silt	-	-	Oct. 26/67
31	T-5	28+06	50 Lt.	254	Upper silt	-	-	Oct. 26/67
32	T-5	28+06	50 Lt.	254	Bedrock	-	-	Oct. 26/67
33	T-123	28+91	71 Rt.	310	Bedrock	2-14	2-20, 26, 32, 38	
34	T-123	28+91	71 Rt.	310	Bedrock	2-14	2-20, 26, 32, 38	
35	T-123	28+91	71 Rt.	310	Bedrock	2-14	2-20, 26, 32, 38	
36	T-122	29+88	78 Lt.	425	Clayey silt	*	*	
37	T-122	29+88	78 Lt.	425	Till	2-16	2-22, 28, 34, 40	
38	T-122	29+88	78 Lt.	425	Bedrock	2-13	2-19, 25, 31, 37	
39	T-121	31+07	49 Rt.	525	Till	2-16	2-22, 28, 34, 40	
40	T-121	31+07	49 Rt.	525	Bedrock	2-13	2-19, 25, 31, 37	
41	T-6	33+03	53 Rt.	721	Clayey silt	*	*	
42	T-6	33+03	53 Rt.	721	Clayey silt	*	*	
43	T-6	33+03	53 Rt.	721	Bedrock	2-13	2-19, 25, 31, 37	
44	T-7	37+06	56 Lt.	1,130	Clayey silt	*	*	
45	T-7	37+06	56 Lt.	1,130	Till	*	*	
46	PIEZOMETER 46 was not installed							
47	T-101	39+88	51 Rt.	1,416	Bedrock	2-13	2-37	
48	T-8	43+76	62 Rt.	1,794	Clayey silt	-	-	Oct. 26/67
49	T-8	43+76	62 Rt.	1,794	Till	2-16	2-22, 28, 34, 40	
50	S-1	25+83	59 Rt.	9	Till	2-17	2-29	
51	S-1	25+83	59 Rt.	9	Bedrock	-	-	Dec. 1/67
52	S-2	25+83	74 Rt.	24	Till	-	-	Dec. 2/67
53	S-2	25+83	74 Rt.	24	Bedrock	-	-	Dec. 2/67
54	S-3	25+83	148 Rt.	98	Till	-	-	Dec. 7/67
55	S-3	25+83	148 Rt.	98	Bedrock	-	-	Dec. 7/67
56	S-4	25+91	449 Rt.	399	Till	2-17	2-23, 29, 35, 41	
57	S-4	25+91	449 Rt.	399	Bedrock	2-15	2-21, 27, 33, 39	
58	PIEZOMETER 58 was not installed							
59	S-1A	25+83	65 Rt.	15	Bedrock	2-12	2-18, 24, 30, 36	
60	N-1	25+89	50 Lt.	100	Till	2-17	2-29, 35	
61	N-2	25+89	250 Lt.	300	Bedrock	2-15	2-27, 33, 39	
62	T-104	30+01	168 Rt.	435	Clayey silt	*	*	
63	T-104	30+01	168 Rt.	435	Till	2-17	2-29, 35, 41	

* Piezometer not considered reliable in analysis of pumping test results.

** Drawdown in piezometer less than 2 feet - insufficient data available to use in analysis of pumping test results.

APPENDIX B

CHEMICAL COMPOSITION OF GROUNDWATER

GROUNDWATER in OVERBURDEN

(Samples obtained by bailing in open auger holes)

Depth at which Sample obtained	p.p.m. SO ₄	Relative degree of SO ₄ attack on concrete
8	164	Positive
8	516	Positive
14	681	Positive
36	713	Positive

GROUNDWATER in BEDROCK

(Samples obtained from discharge from pumped well)

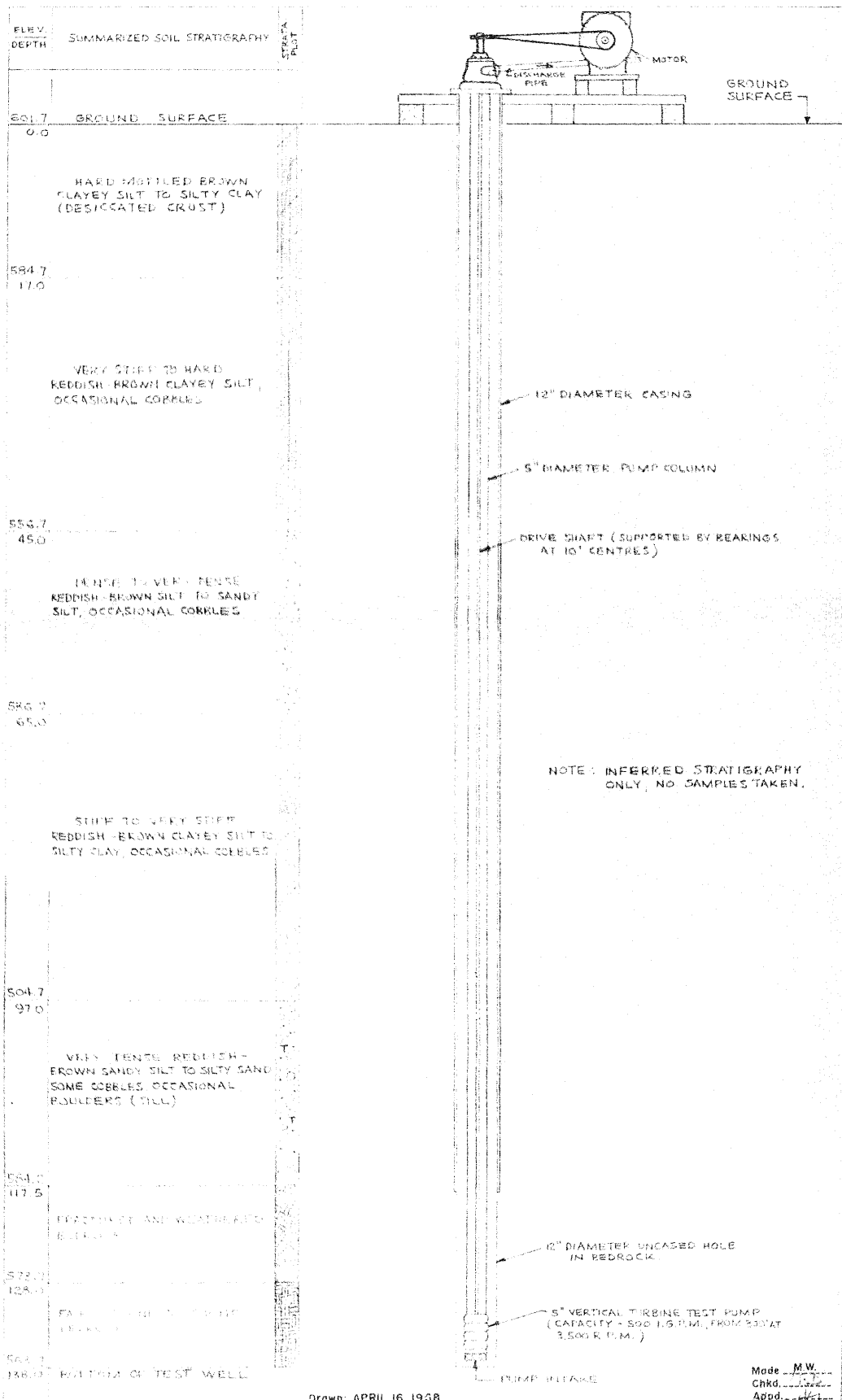
Time at which Sample Obtained	p.p.m. SO ₄	pH	Relative degree of attack on concrete
16:35 hours Dec. 13/67	2,045	7.4	Severe
22:30 hours Dec. 13/67	2,040	7.4	Severe
17:30 hours Dec. 14/67	2,040	7.4	Severe
10:15 hours Dec. 15/67	2,026	7.4	Severe
23:00 hours Dec. 15/67	2,035	7.4	Severe
15:00 hours Dec. 16/67	2,040	7.4	Severe
08:00 hours Dec. 18/67	2,035	7.4	Severe
13:00 hours Dec. 18/67	2,010	7.4	Severe
16:35 hours Dec. 18/67	2,095	...	Severe
22:30 hours Dec. 18/67	2,060	...	Severe
12:00 hours Dec. 19/67	2,060	...	Severe

ATTACK ON CONCRETE BY SOILS AND WATER CONTAINING VARIOUS SULPHATE CONCENTRATIONS

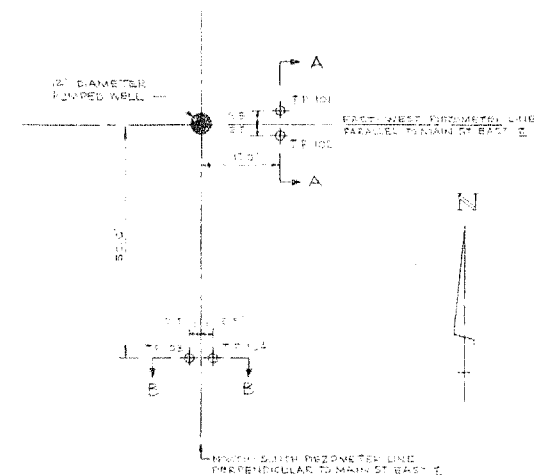
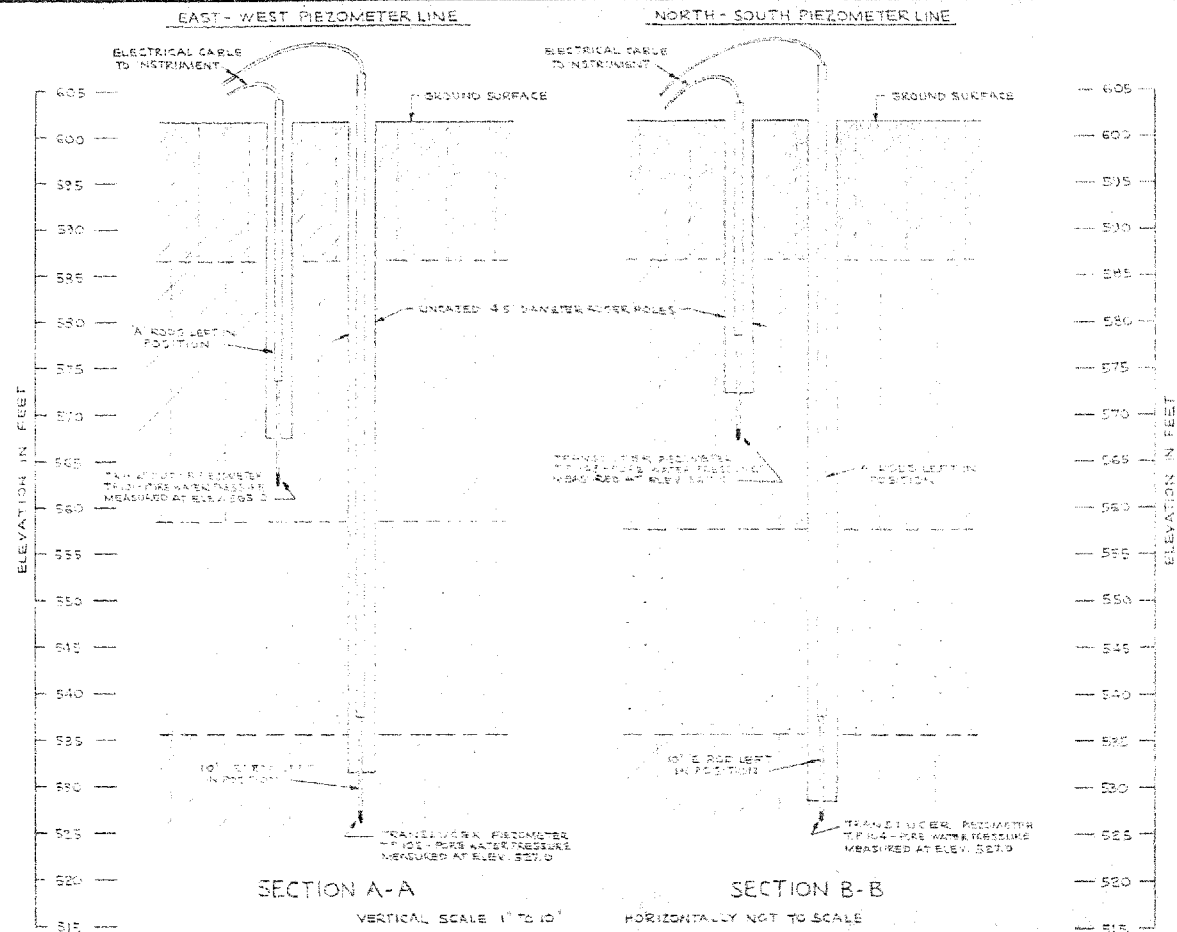
<u>Relative Degree of SO₄ Attack</u>	<u>% Water Soluble SO₄ in Soil Samples</u>	<u>p.p.m. SO₄ in Water Samples</u>
Negligible	0.00 - 0.10	0 - 150
Positive	0.10 - 0.20	150 - 1000
Considerable	0.20 - 0.50	1000 - 2000
Severe	over 0.50	over 2000

DETAILS OF PUMPED WELL

FIGURE 2-2



Drawn: APRIL 16, 1938.

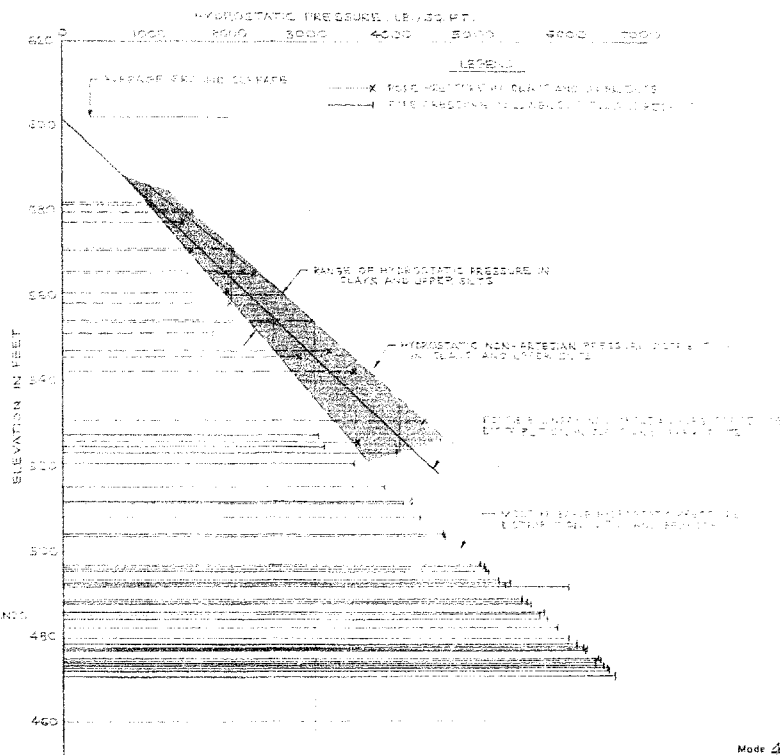


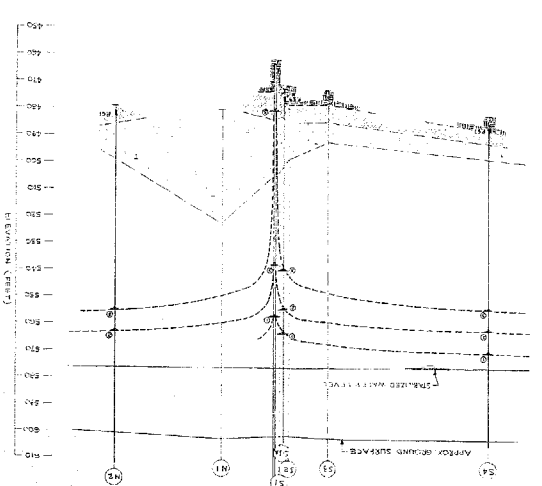
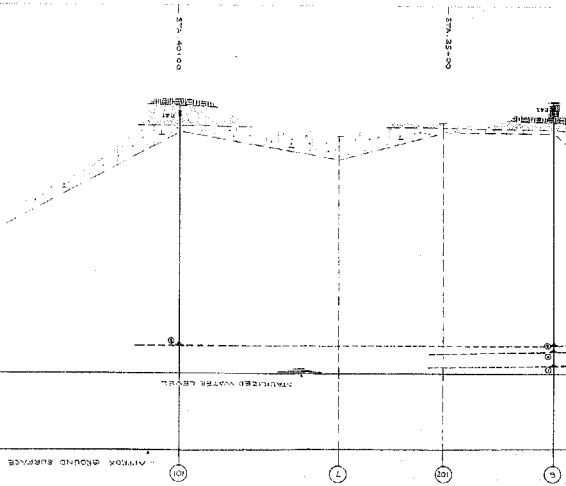
Drawn: FEB. 15, 1968

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Mode *Hand*
Chkd. *Hand*
Appd. *Hand*

FIGURE 2-4.



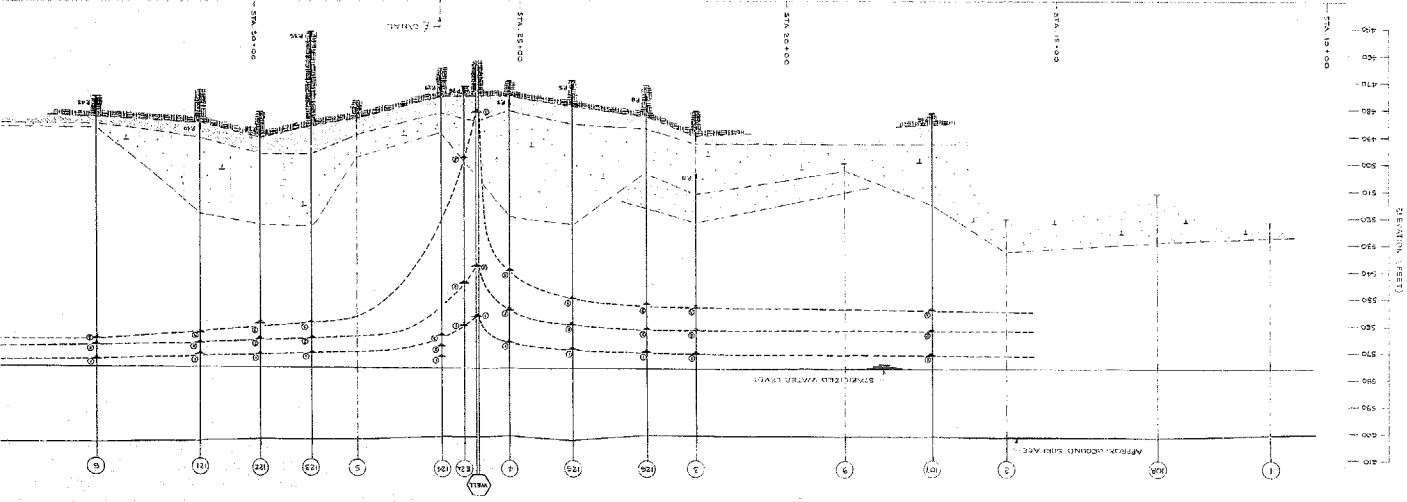


HORIZONTAL SCALE 1" = 100'
VERTICAL SCALE 1" = 20'

- SIMPLIFIED SOIL STRATIGRAPHY**
- CLAY, SILT, SAND AND SILT DEPOSITIONAL DEPOSITS (GROUND WATER LEVEL, INTERFERENCE OF AQUIFERS)
 - LOWER SILT
 - SILT SAND TO SANDY SILT TILL
 - WEATHERED BEDROCK
 - SOUND BEDROCK

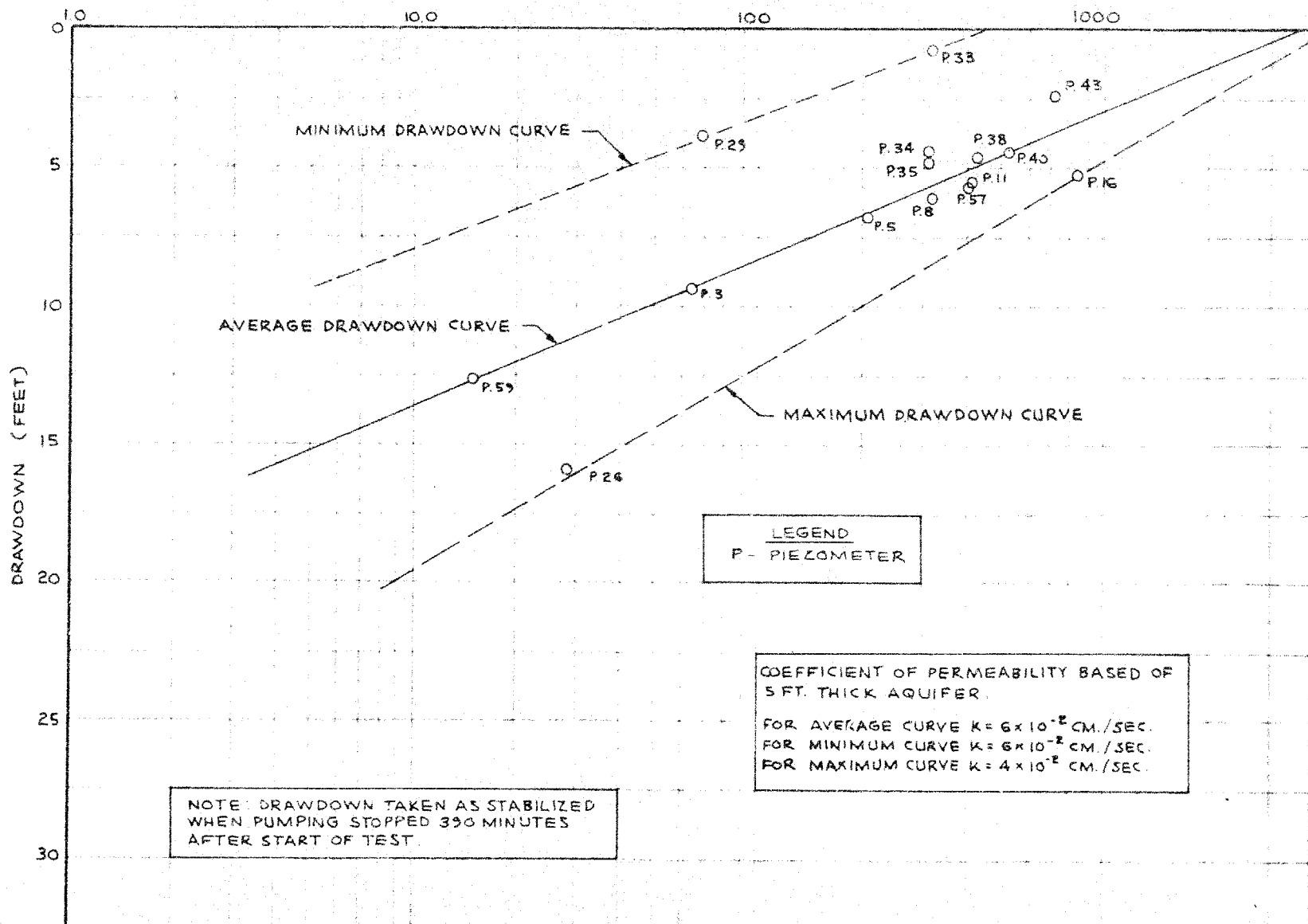
- LEGEND**
- BOREHOLE IN ELEVATION
 - PUMPED WELL IN ELEVATION
 - STABILIZED WATER LEVEL IN PNEUMATIC WELL (PNEUMATIC NUMBER)
 - PNEUMATIC IN ELEVATION (PNEUMATIC NUMBER)

FIGURE 2-5 SCHEMATIC SECTIONS SHOWING FINAL DRAWDOWN IN BEDROCK FOR VARIOUS PUMPING RATES "Q"

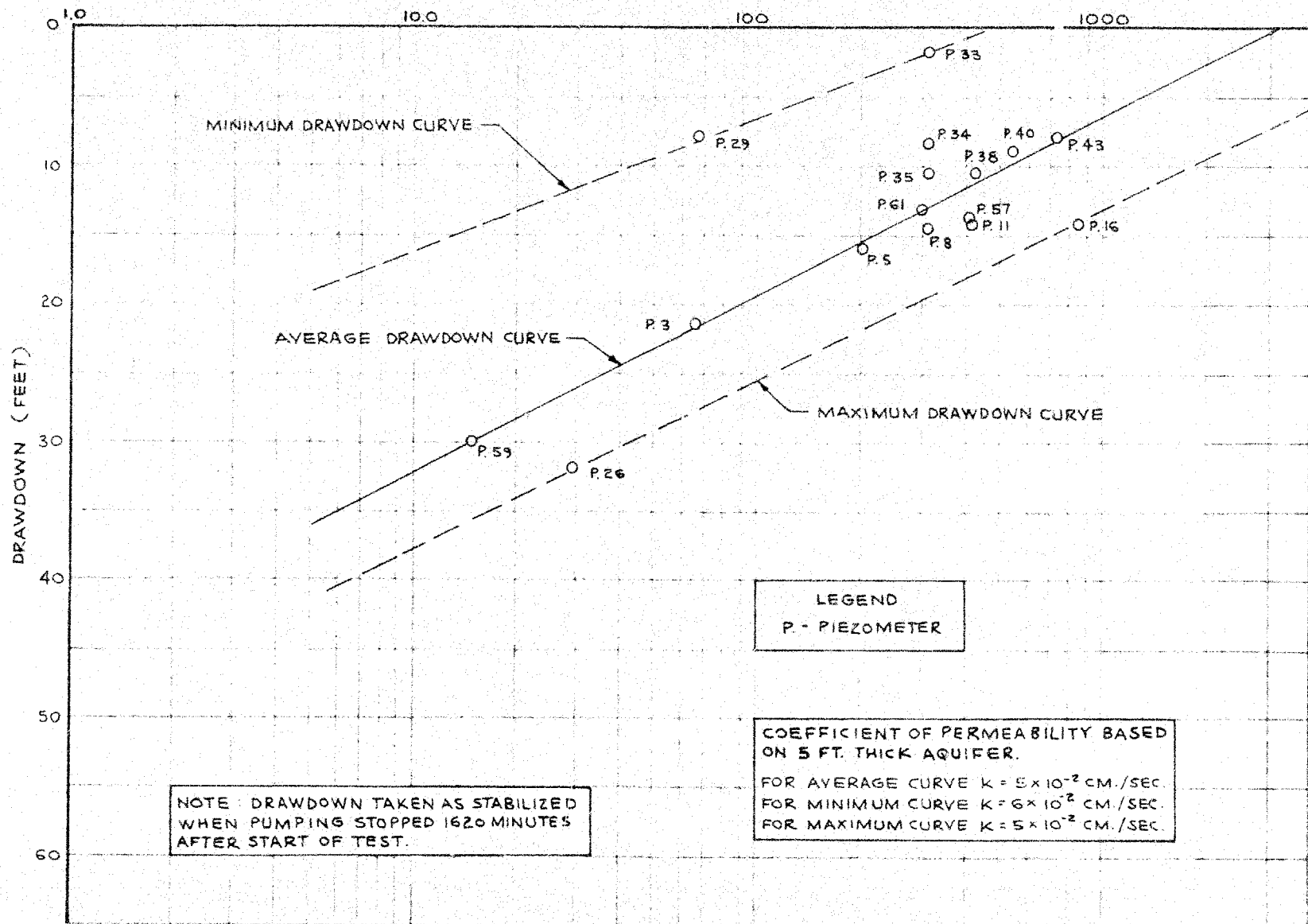


SCHEMATIC SECTION PARALLEL TO MAIN STREET EAST CENTRELNE

RADIAL DISTANCE FROM PUMPED WELL (FEET)



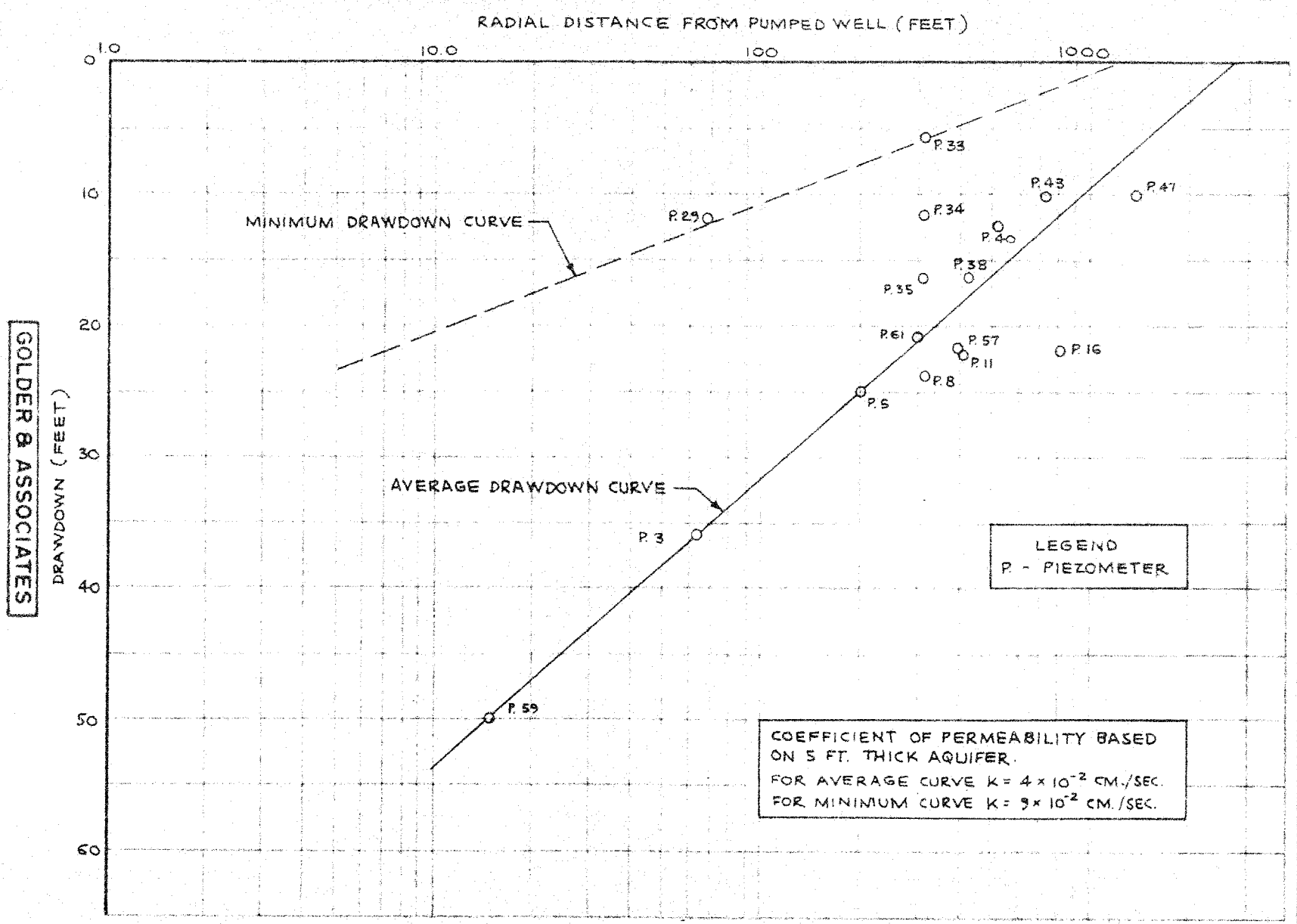
RADIAL DISTANCE FROM PUMPED WELL (FEET)



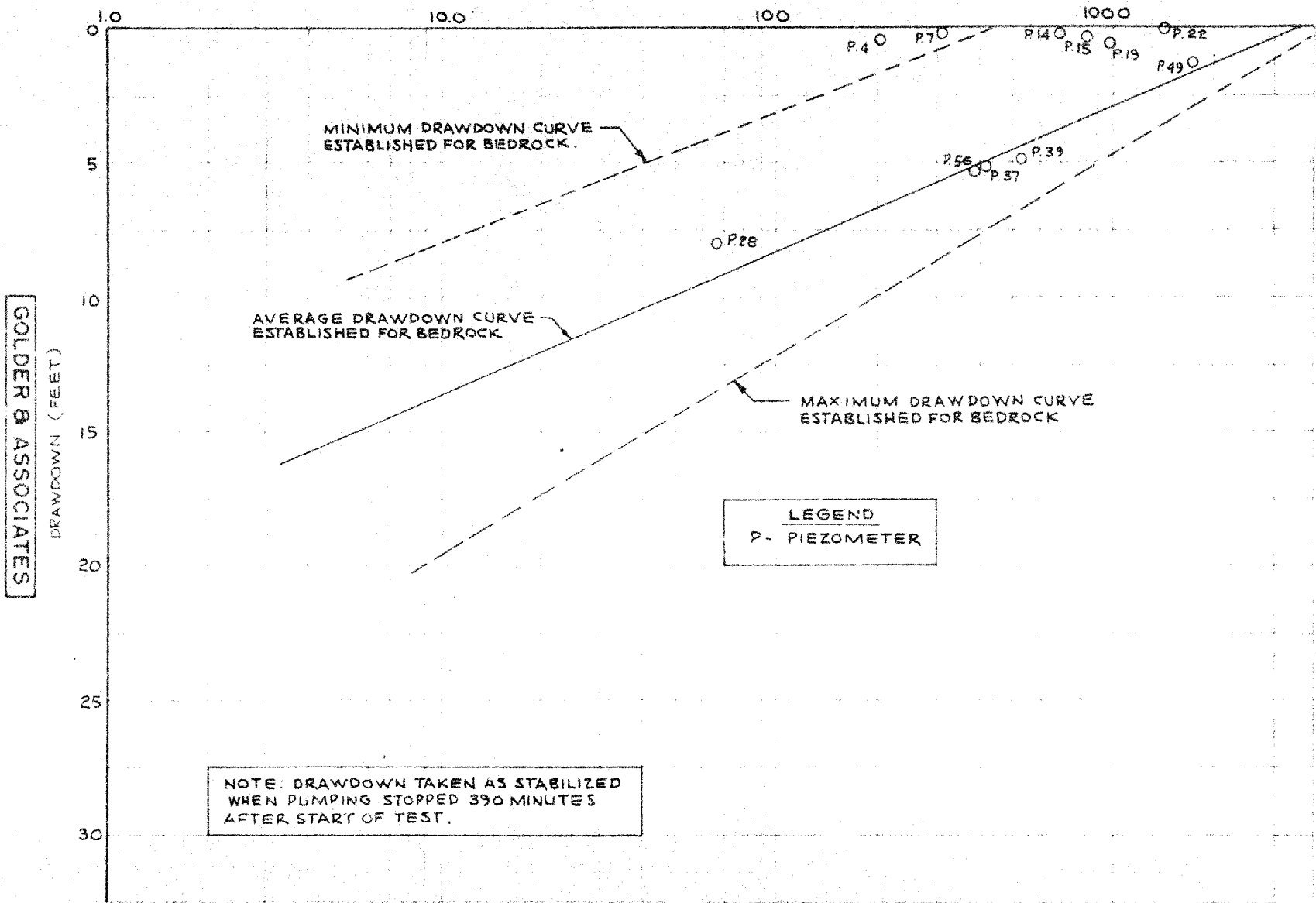
DRAWDOWN IN BEDROCK
PUMPING RATE $Q = 101$ I.G.F.M. (SECOND PUMPING)

FIGURE 2-7

GOLDER & ASSOCIATES



RADIAL DISTANCE FROM PUMPED WELL (FEET)

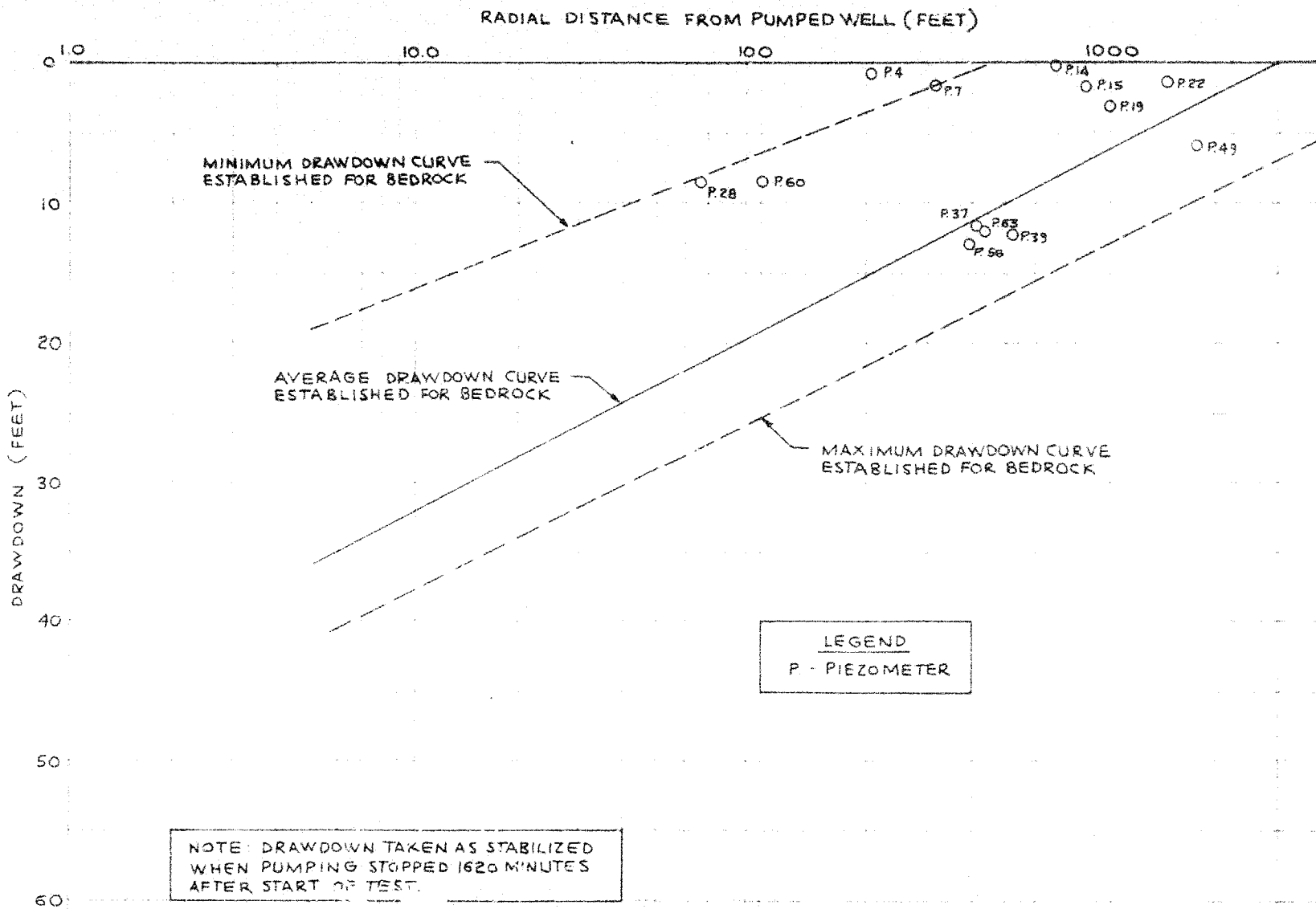


GOLDER & ASSOCIATES

DRAWDOWN IN TILL
PUMPING RATE Q = 51 T.G.P.M.

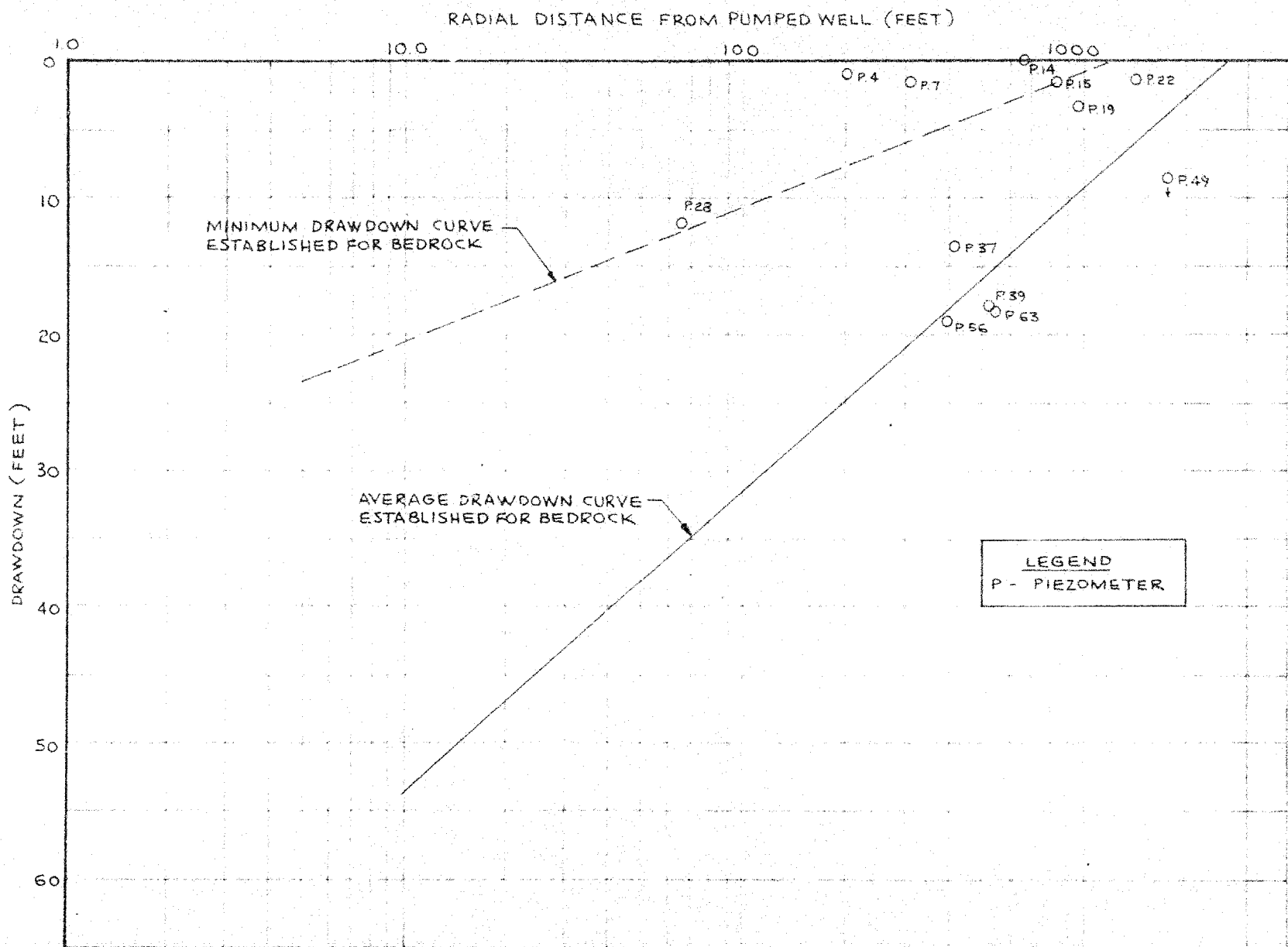
FIGURE 2-9

GOLDER & ASSOCIATES

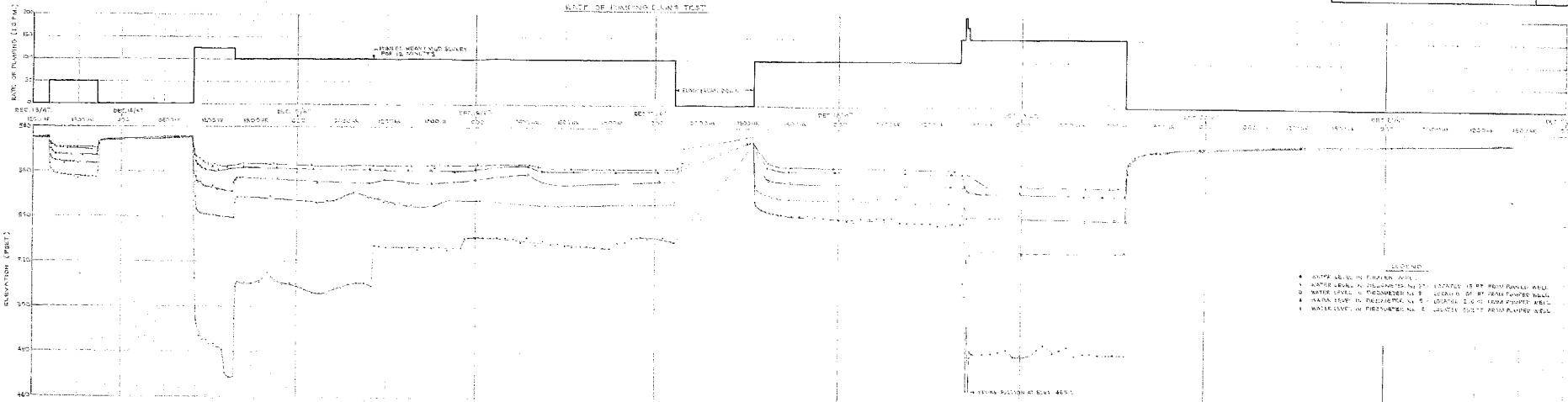


DRAWDOWN IN TILL
PUMPING RATE $Q = 101$ G.P.M. (SECOND PUMPING)

FIGURE 2-10



RATE OF PUMPING DURING TEST



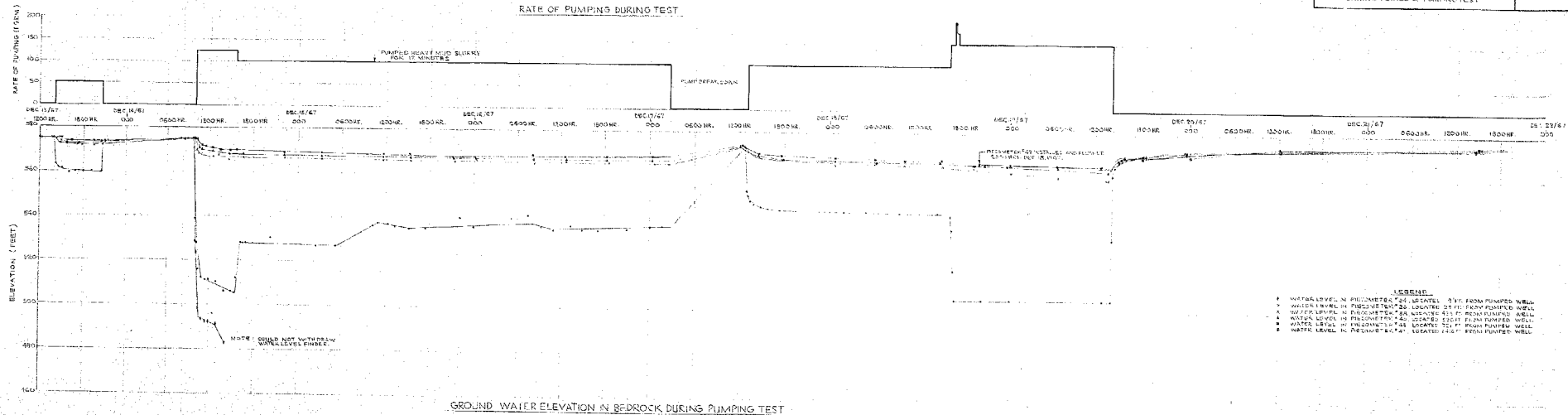
GROUND WATER ELEVATION IN BEDROCK DURING PUMPING TEST

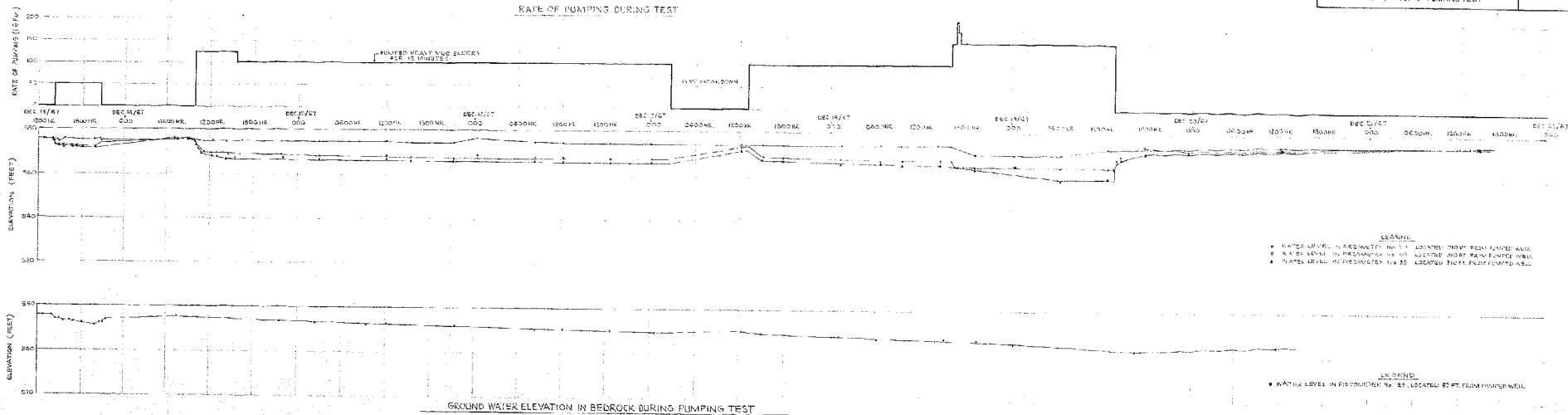
- LEGEND
- A WATER LEVEL IN PUMPING WELL
 - B WATER LEVEL IN OBSERVATION POINT LOCATED 15 FT FROM PUMPING WELL
 - C WATER LEVEL IN OBSERVATION POINT LOCATED 50 FT FROM PUMPING WELL
 - D WATER LEVEL IN OBSERVATION POINT LOCATED 100 FT FROM PUMPING WELL
 - E WATER LEVEL IN OBSERVATION POINT LOCATED 200 FT FROM PUMPING WELL

Drawn: FEB. 20, 1968.

GOLDER & ASSOCIATES

Made by: J. J. Golder
Chk'd: J. J. Golder
App'd: J. J. Golder



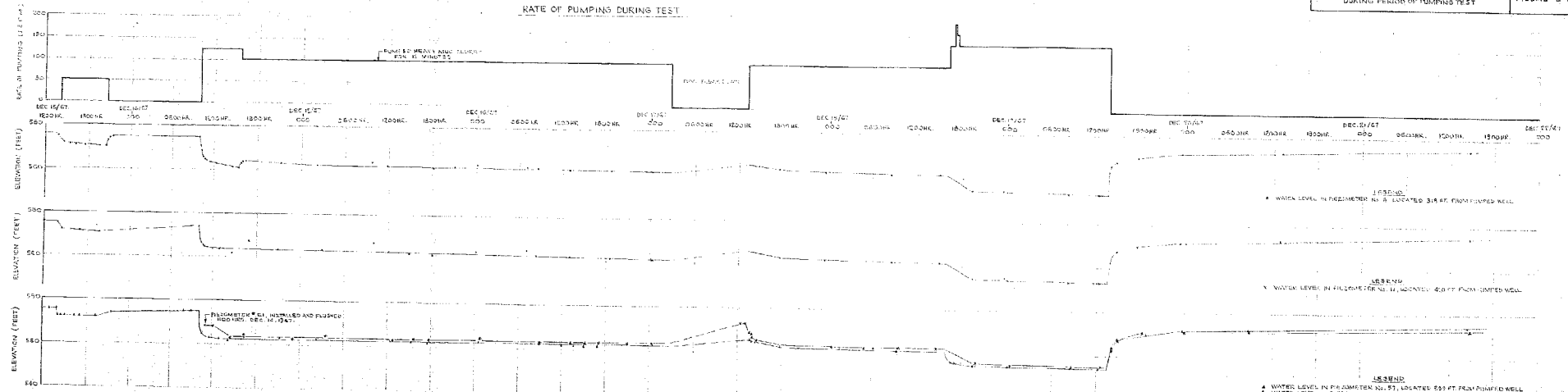


Drawn: FEB. 28, 1968.

GOLDER & ASSOCIATES

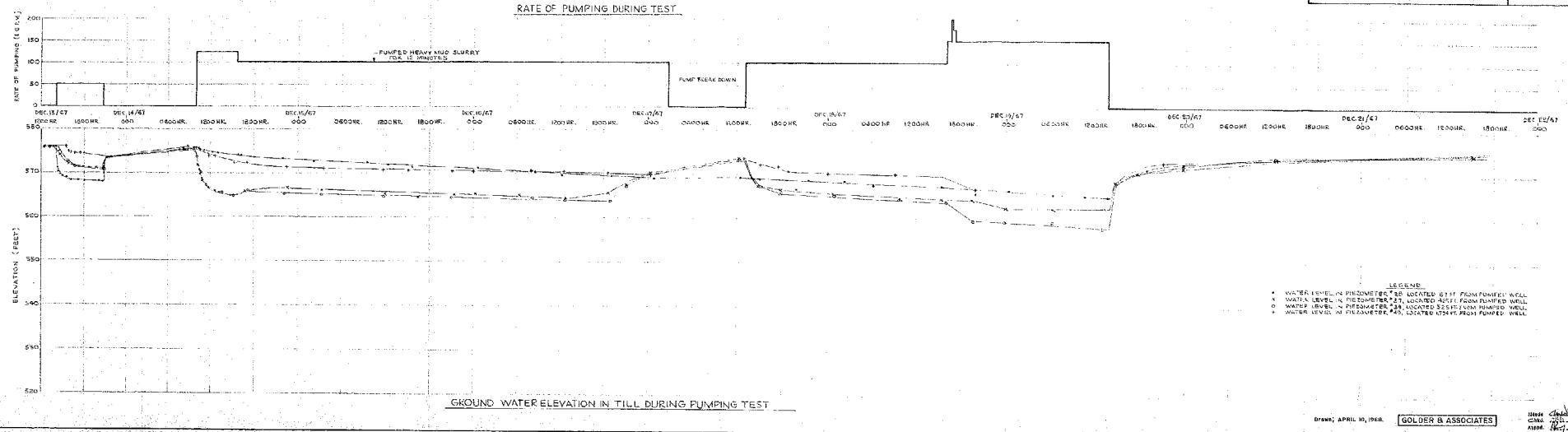
 Made
 Chd.
 2/28/68

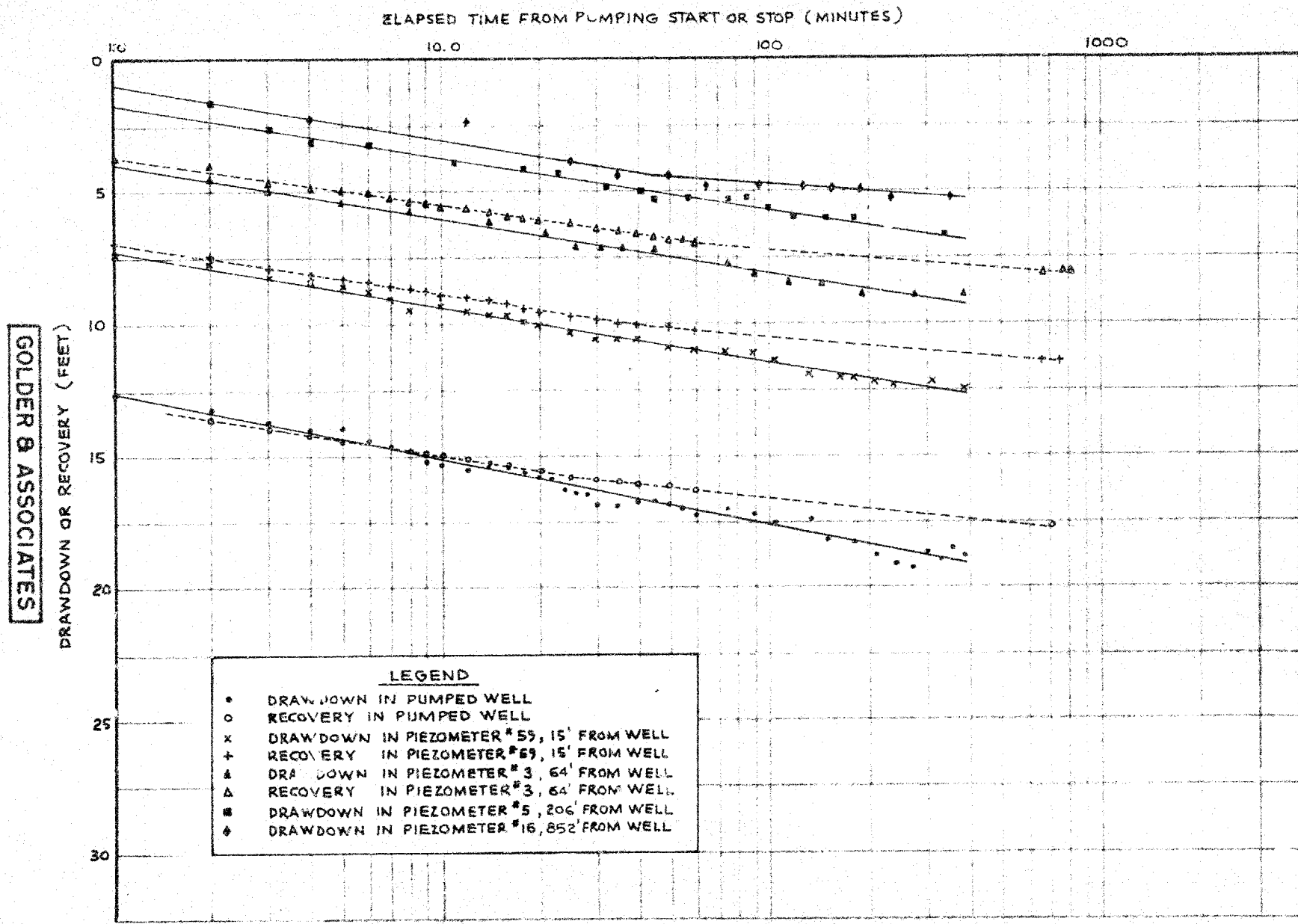
RATE OF PUMPING DURING TEST

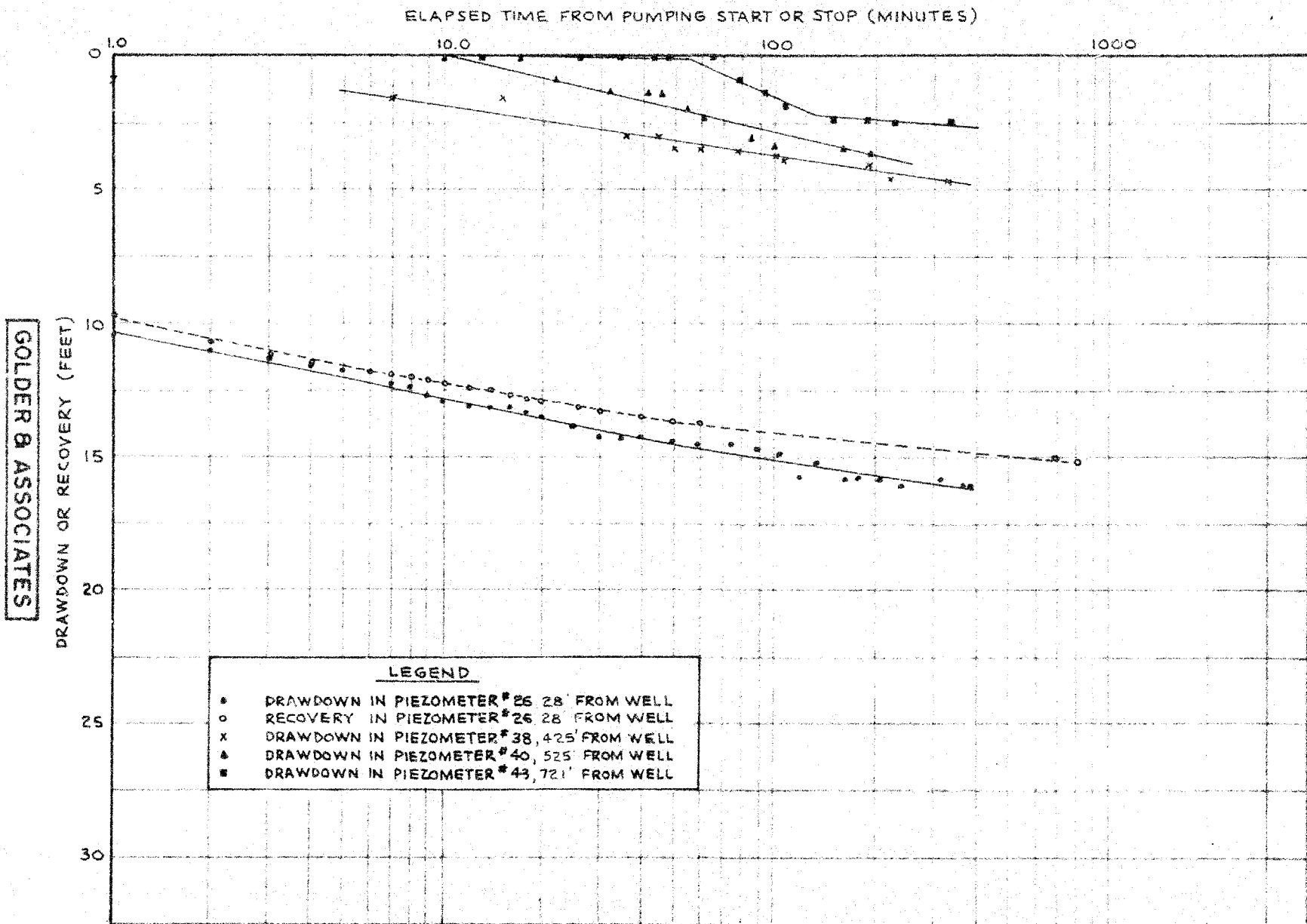


GROUND WATER ELEVATION IN BEDROCK DURING PUMPING TEST

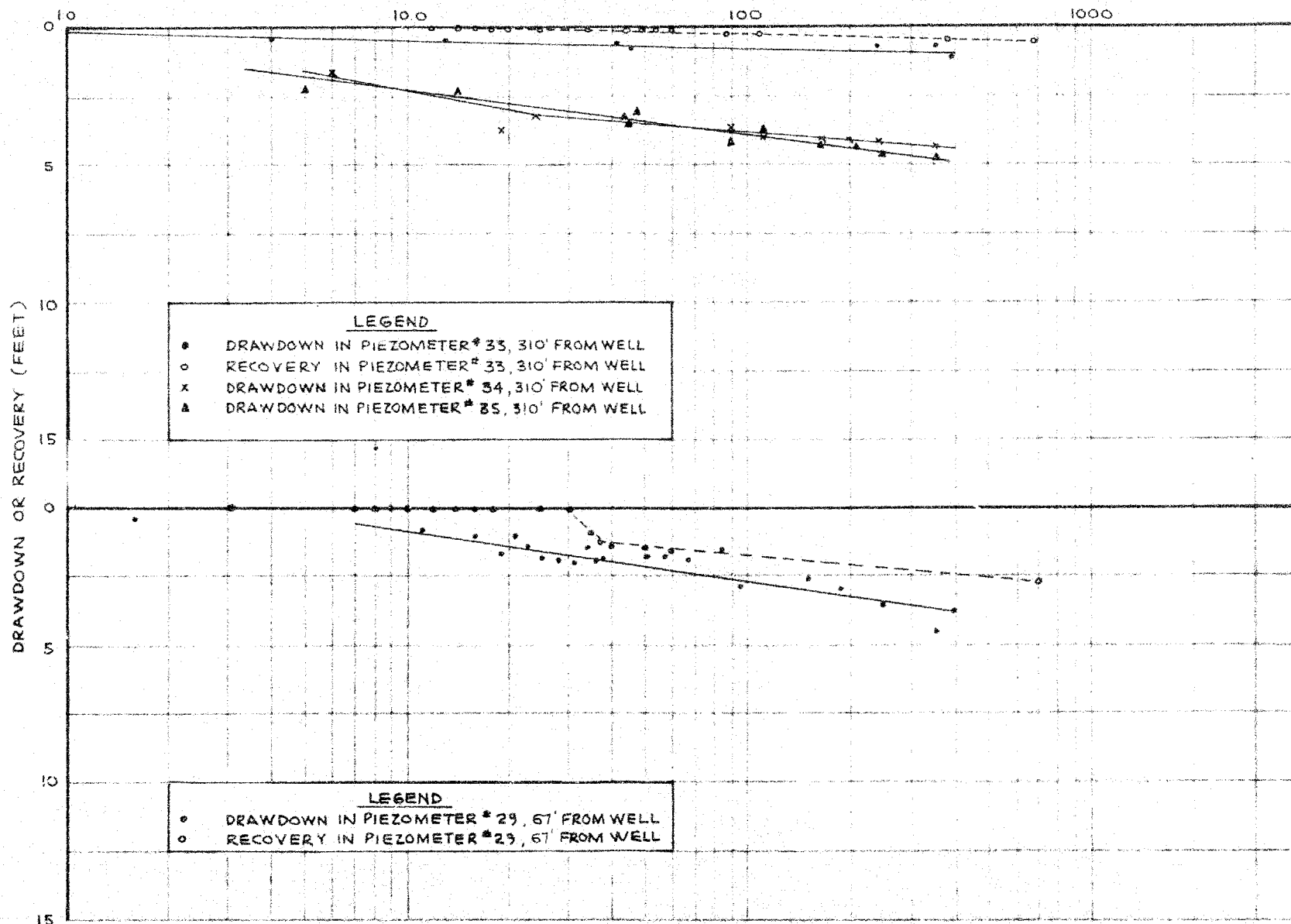
- LEGEND
- ▲ WATER LEVEL IN PIEZOMETER NO. 5, LOCATED 315 FT. FROM PUMPED WELL
 - ▲ WATER LEVEL IN PIEZOMETER NO. 41, LOCATED 400 FT. FROM PUMPED WELL





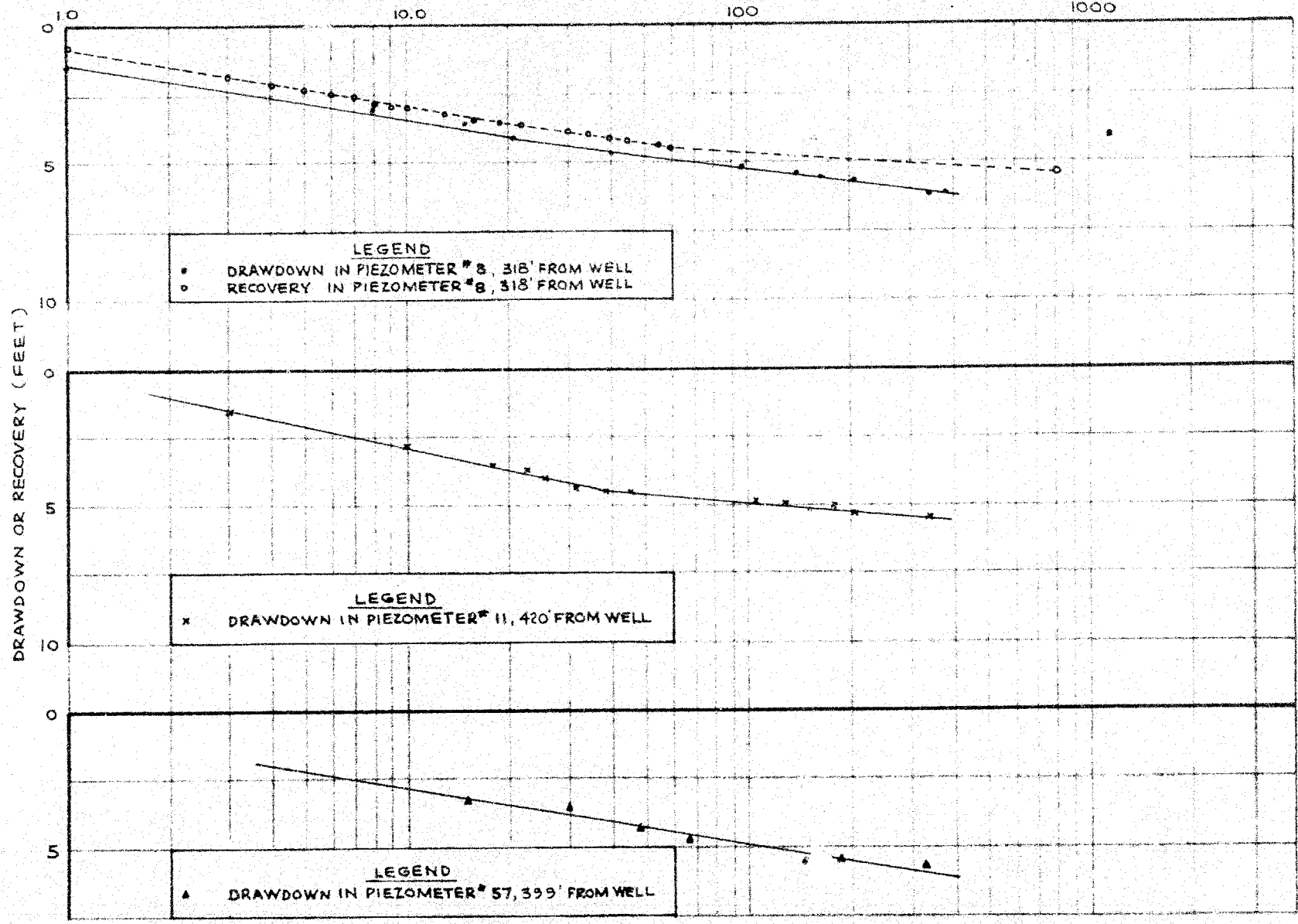


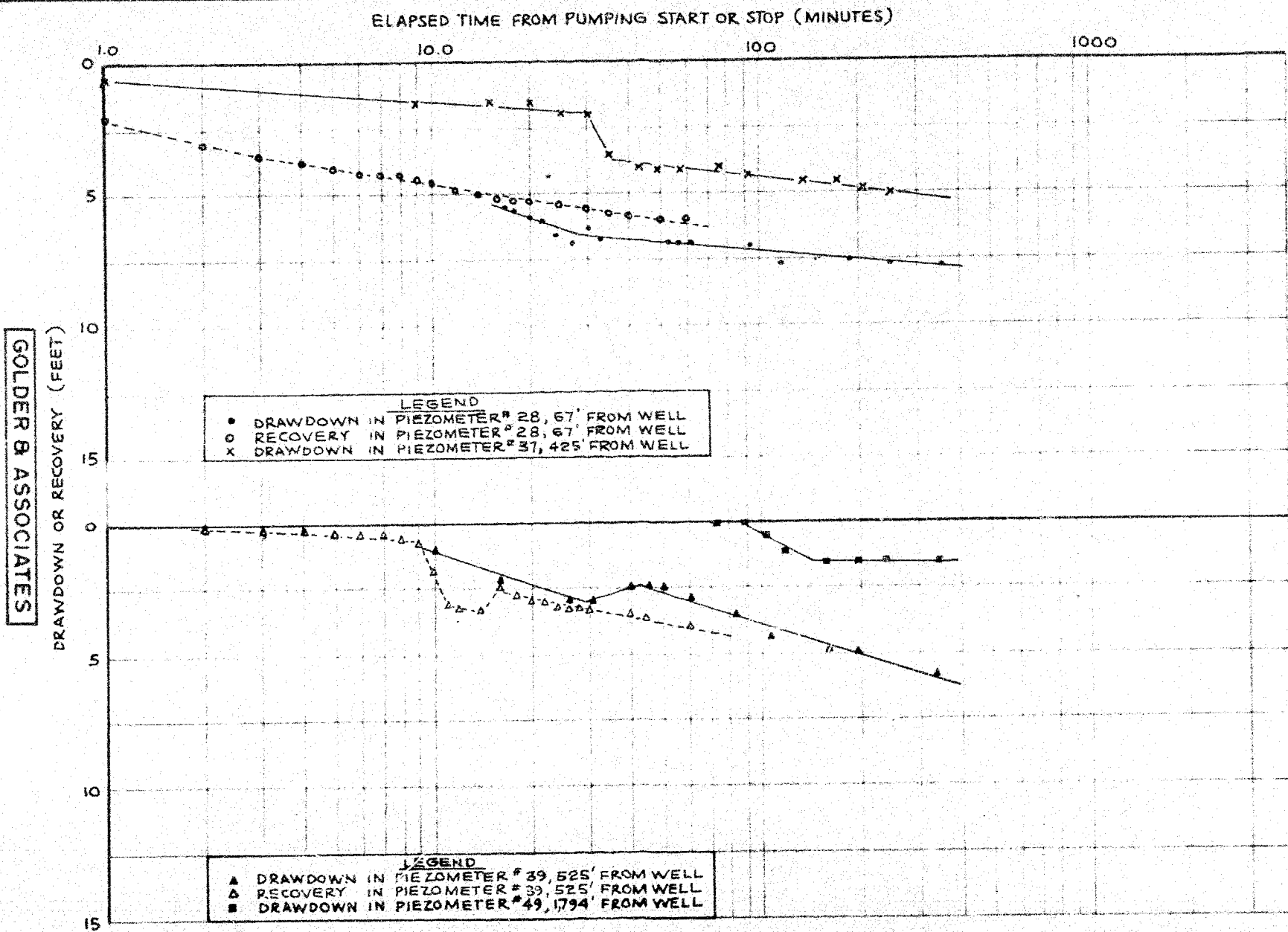
ELAPSED TIME FROM PUMPING START OR STOP (MINUTES)



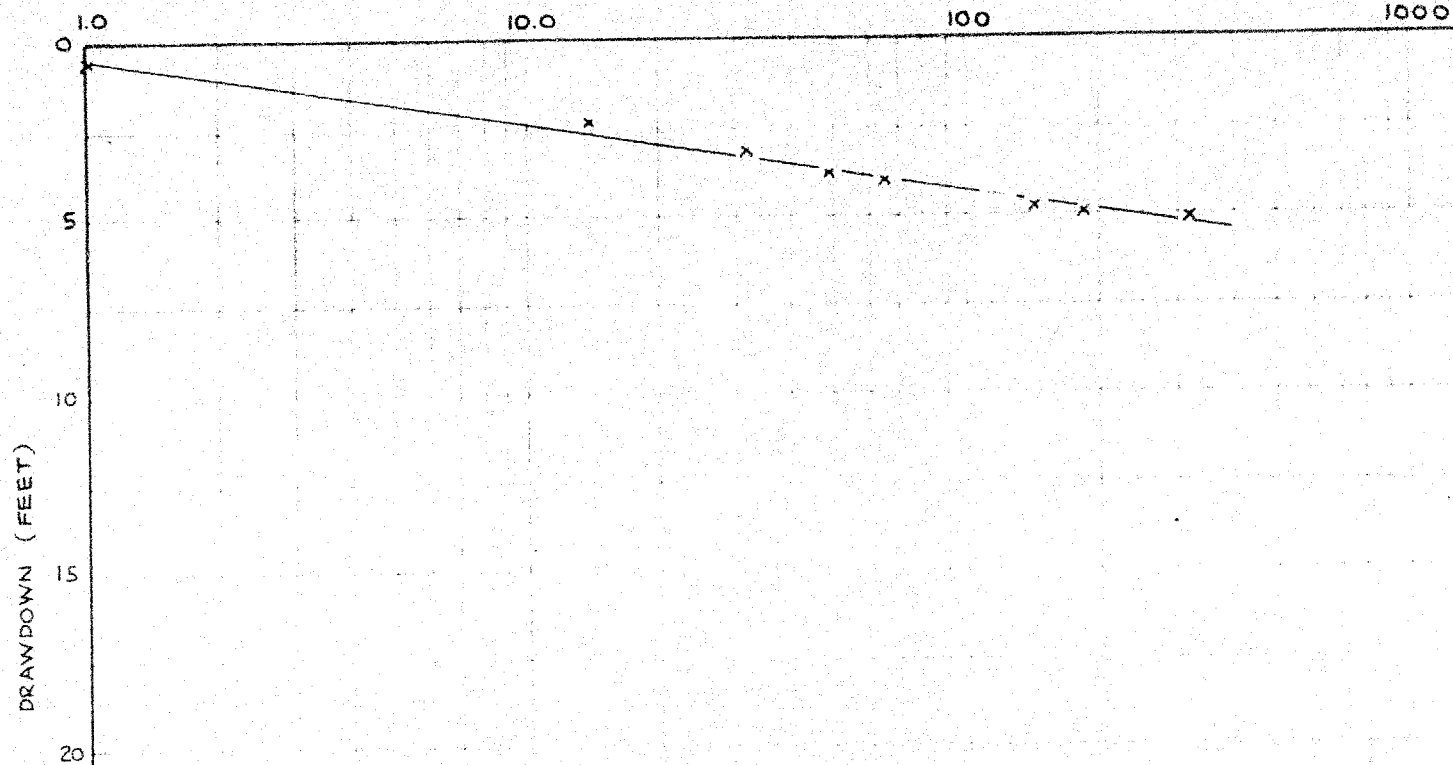
ELAPSED TIME FROM PUMPING START OR STOP (MINUTES)

GOLDER & ASSOCIATES



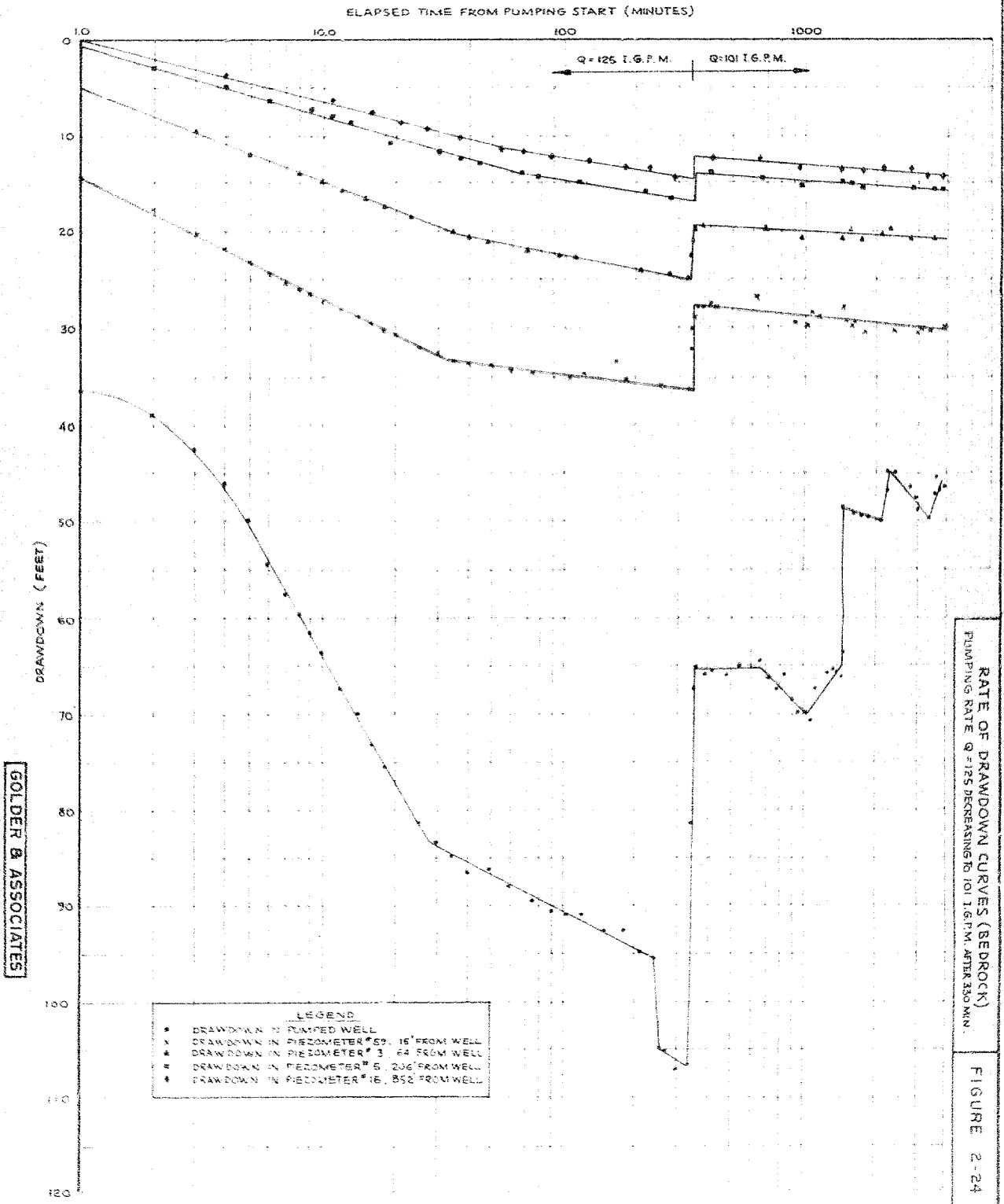


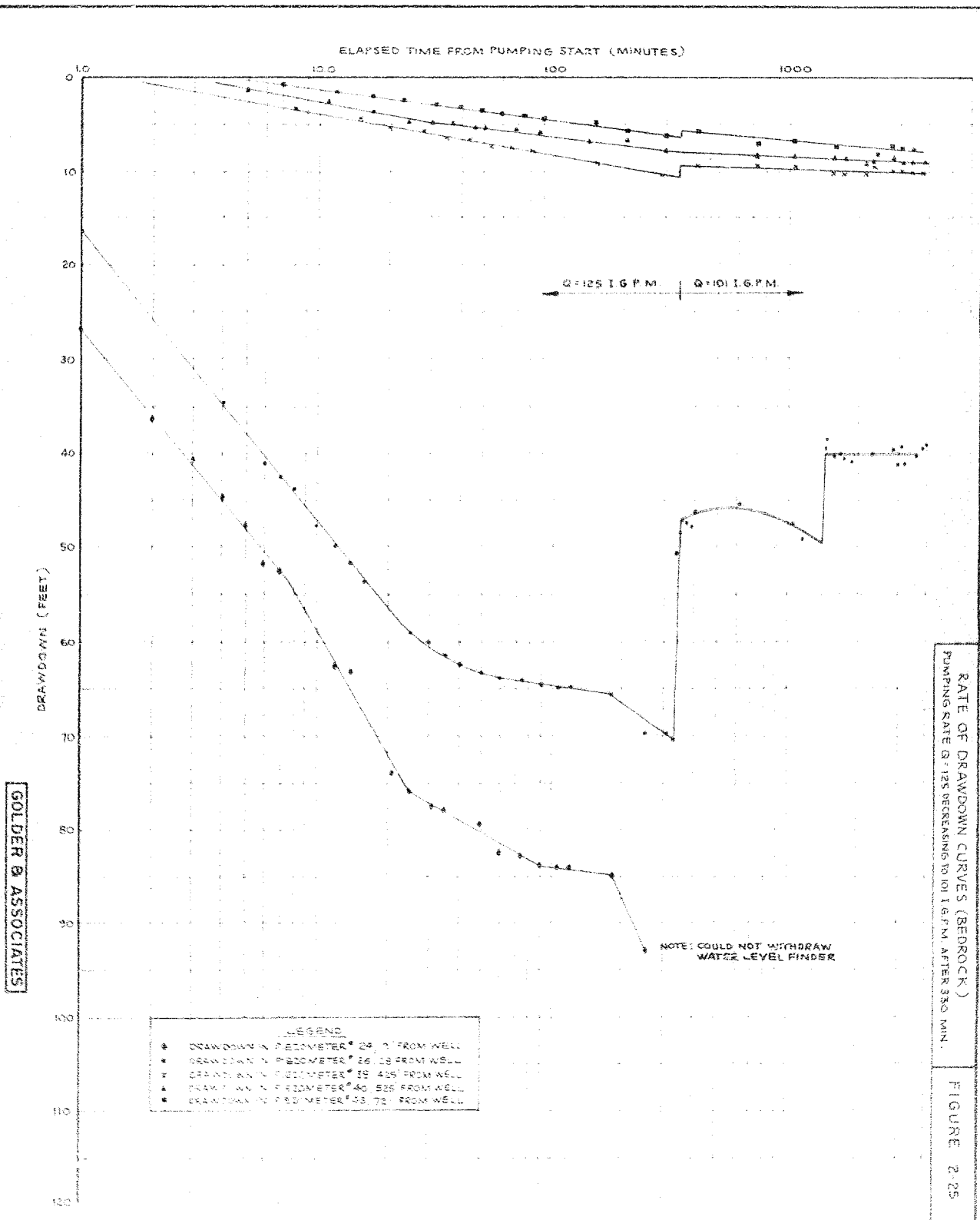
ELAPSED TIME FROM PUMPING START (MINUTES)



LEGEND

x DRAWDOWN IN PIEZOMETER # 56, 399' FROM WELL

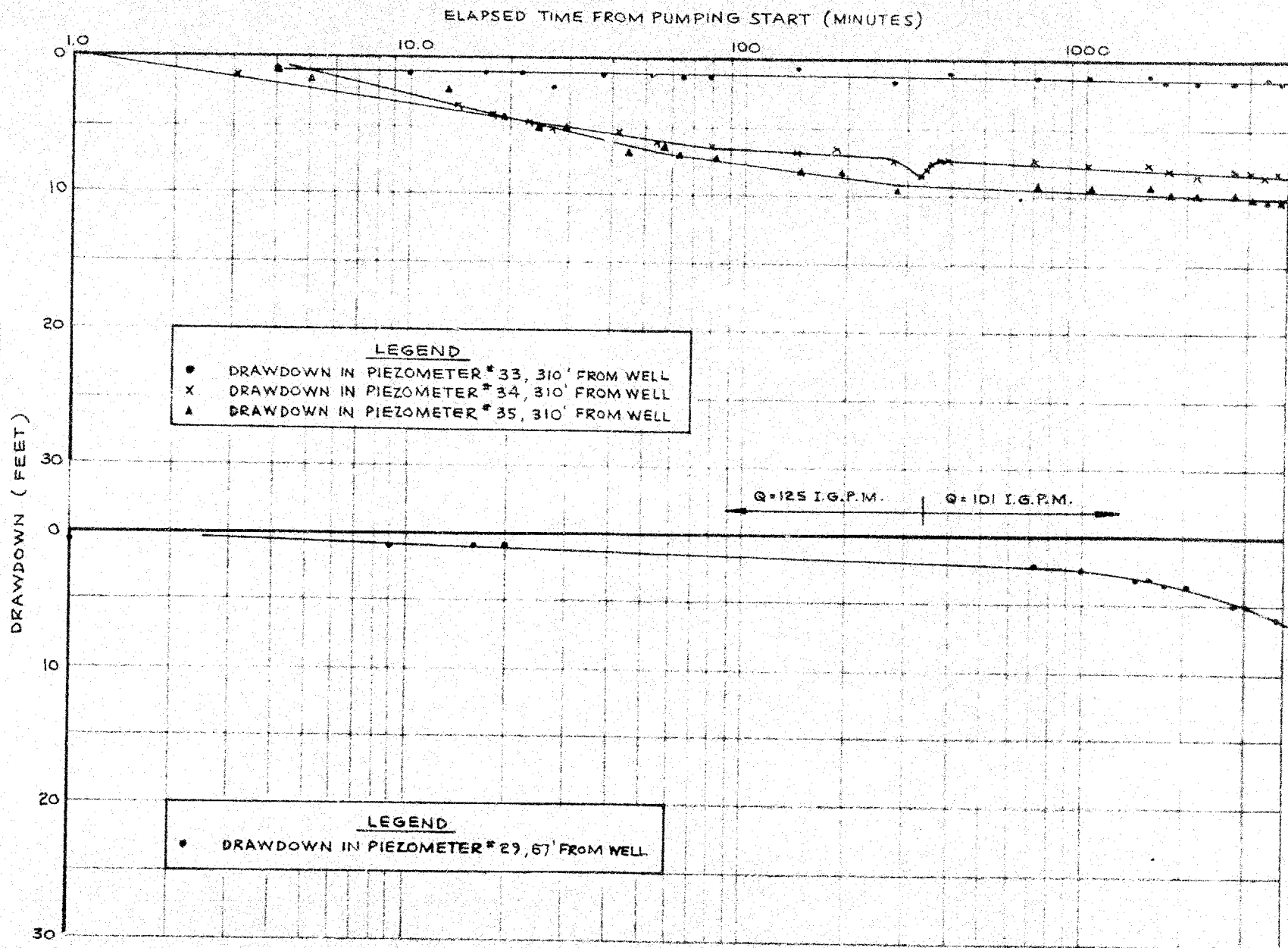




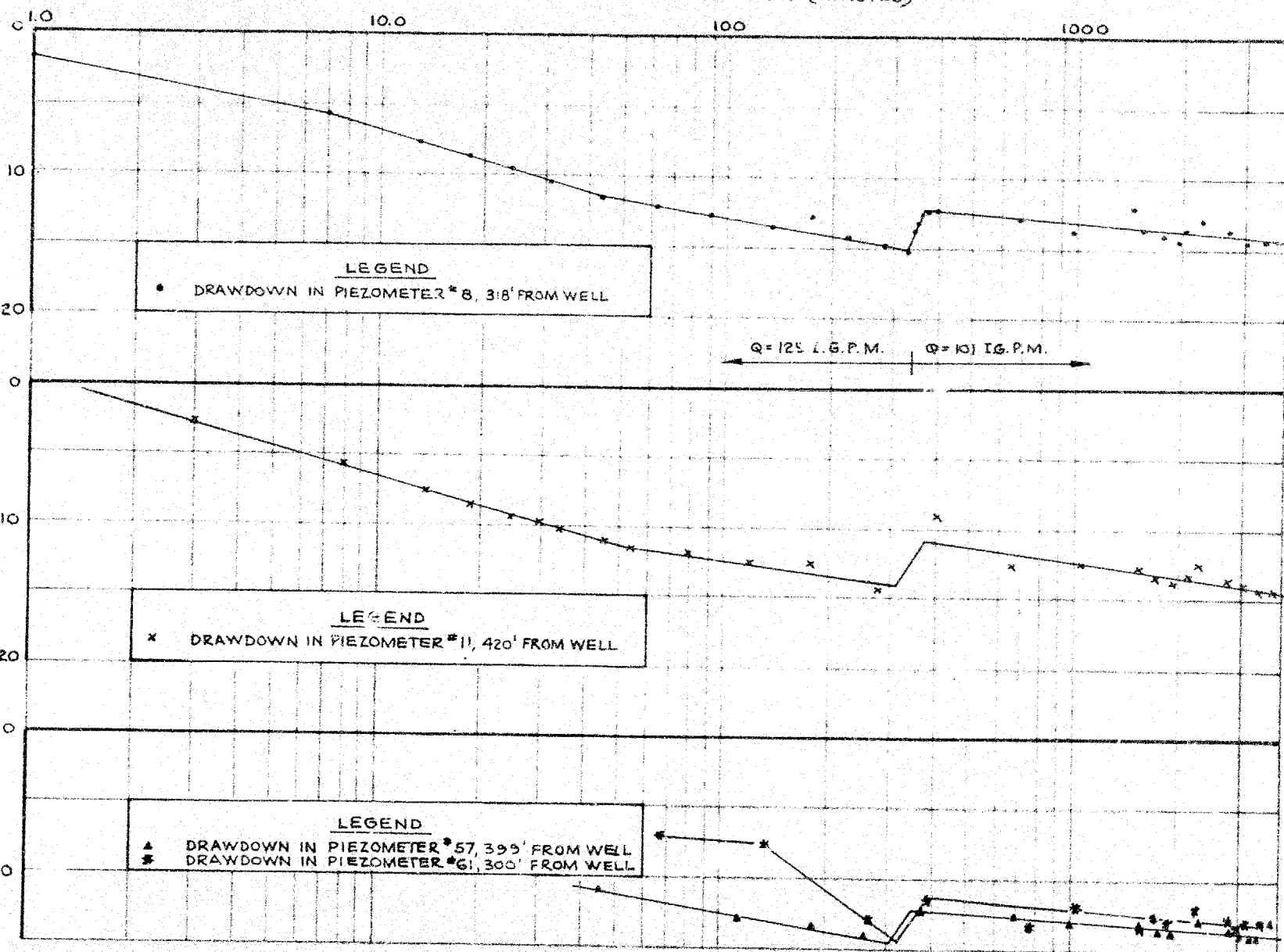
RATE OF DRAWDOWN CURVES (BEDROCK)
PUMPING RATE IS 125 DECREASING TO 101 I.G.P.M. AFTER 330 MIN.

FIGURE 2-25

GOLDER & ASSOCIATES



ELAPSED TIME FROM PUMPING START (MINUTES)



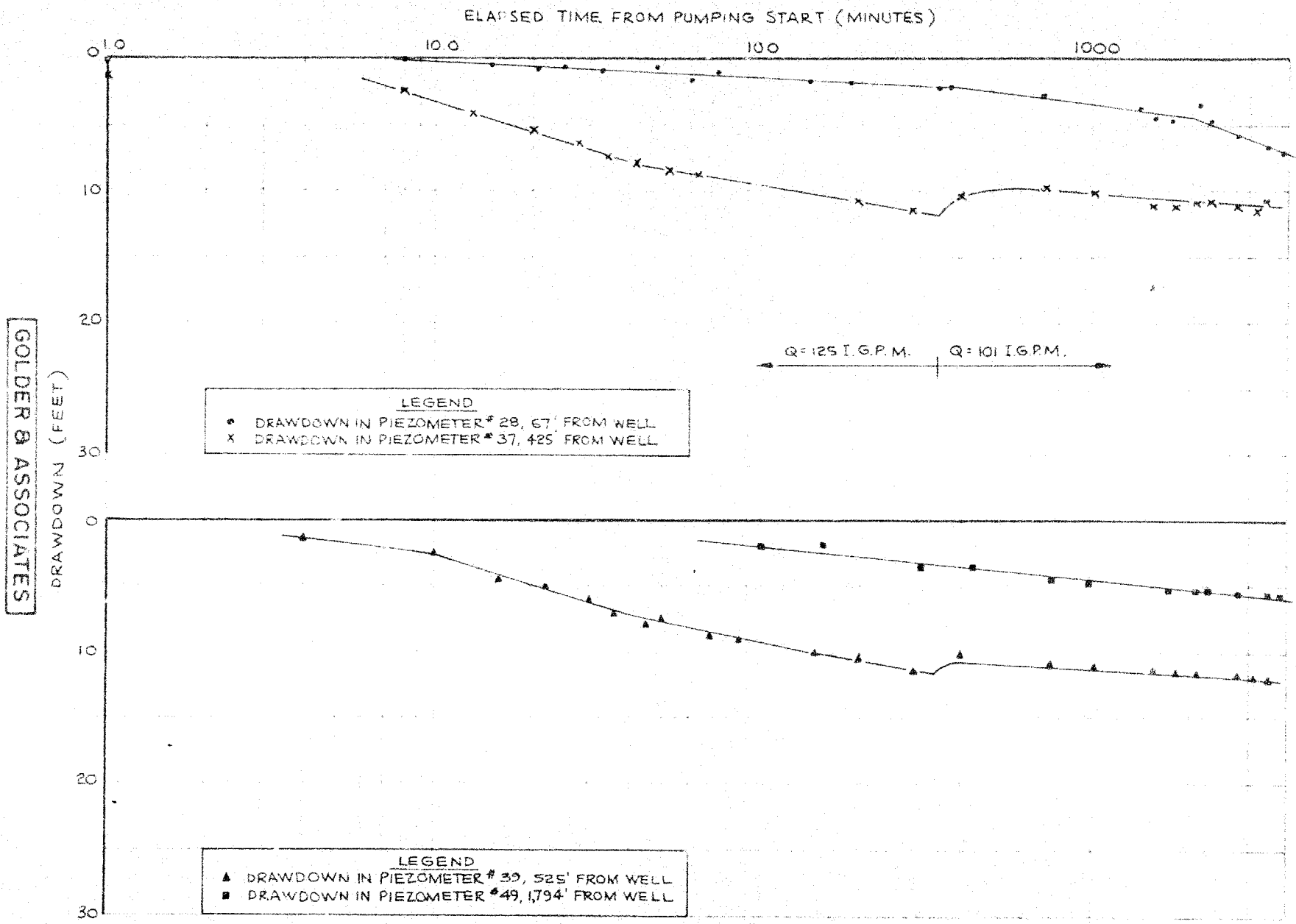
GOLDER & ASSOCIATES

 RATE OF DRAWDOWN CURVES (BEDROCK)
 PUMPING RATE Q = 125 DECREASING TO 101 L.G.P.M. AFTER 350 MIN.

FIGURE 2-27

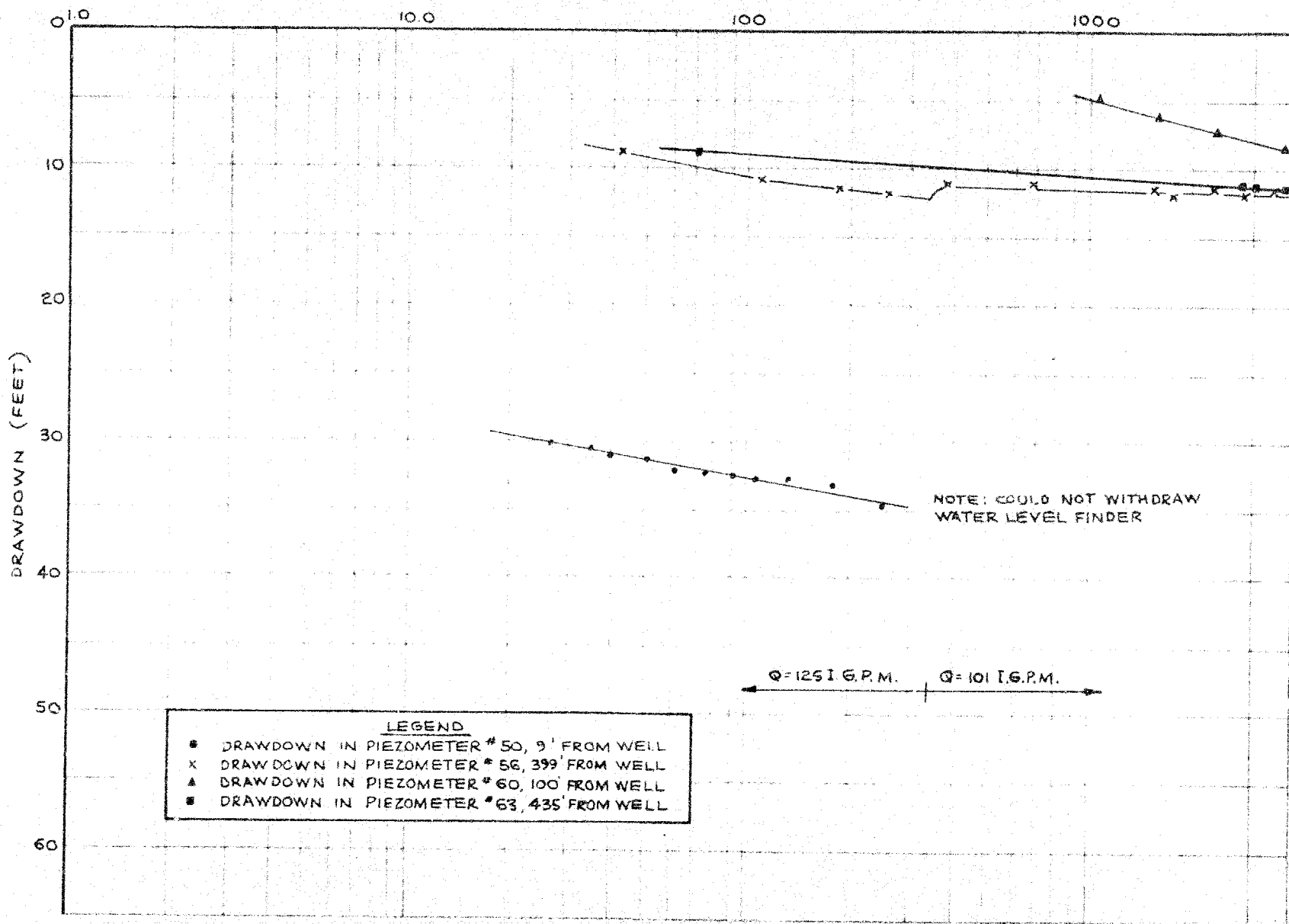
RATE OF DRAWDOWN CURVES (TILL)
 PUMPING RATE Q = 125 DECREASING TO 101 I.G.P.M. AFTER 350 MIN.

FIGURE 2-28



ELAPSED TIME FROM PUMPING START (MINUTES)

GOLDER & ASSOCIATES

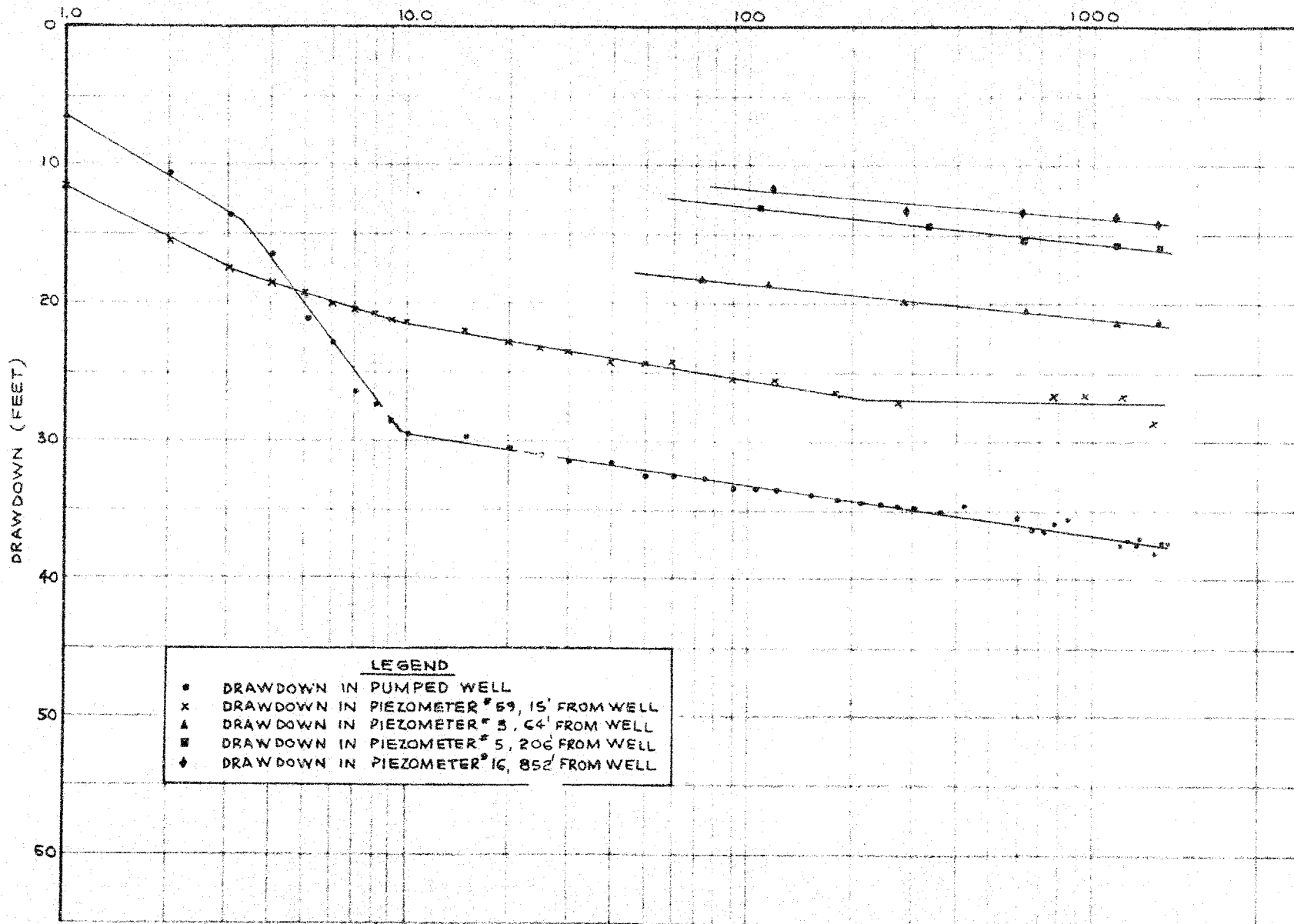


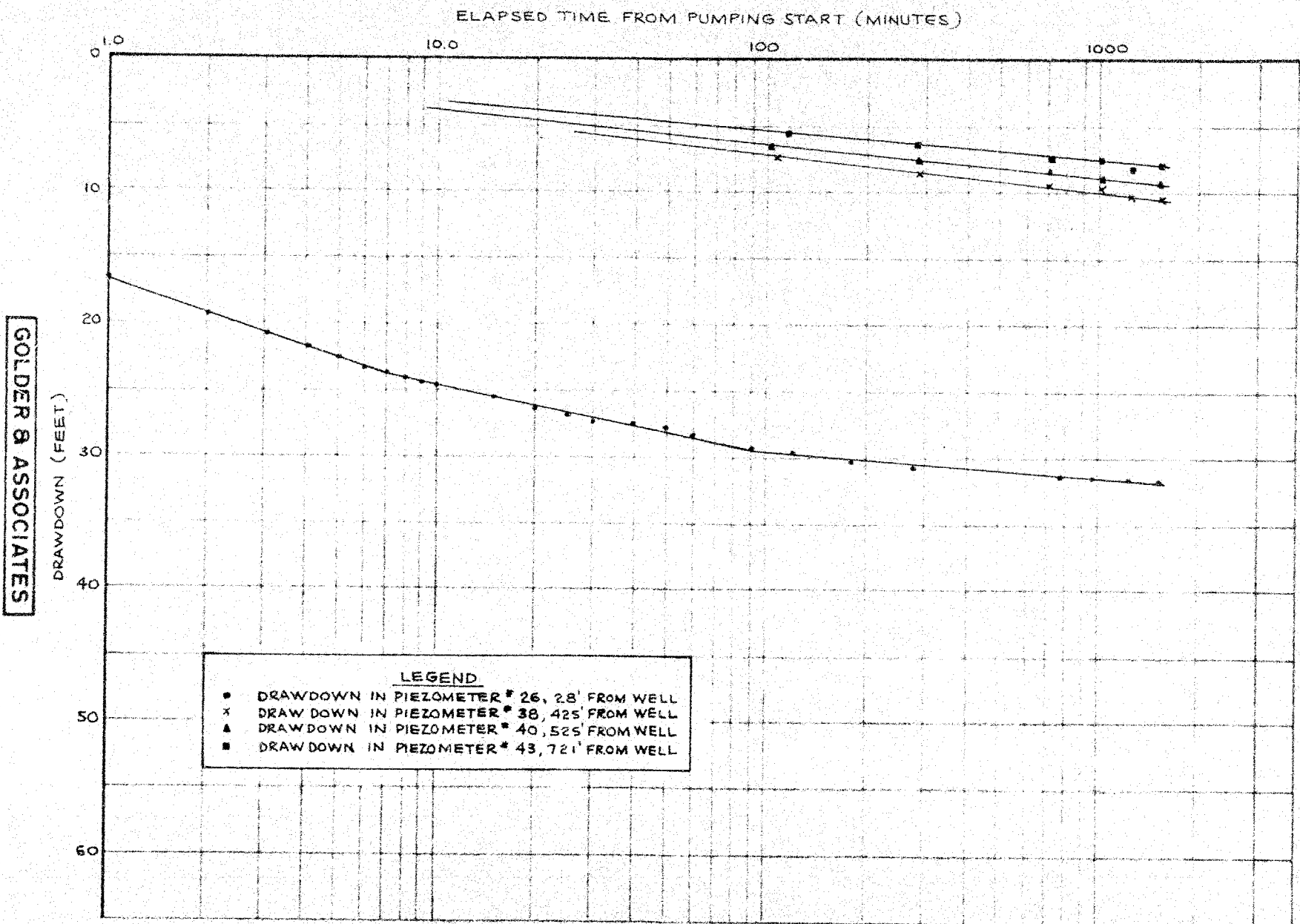
ELAPSED TIME FROM PUMPING START (MINUTES)

10.0

100

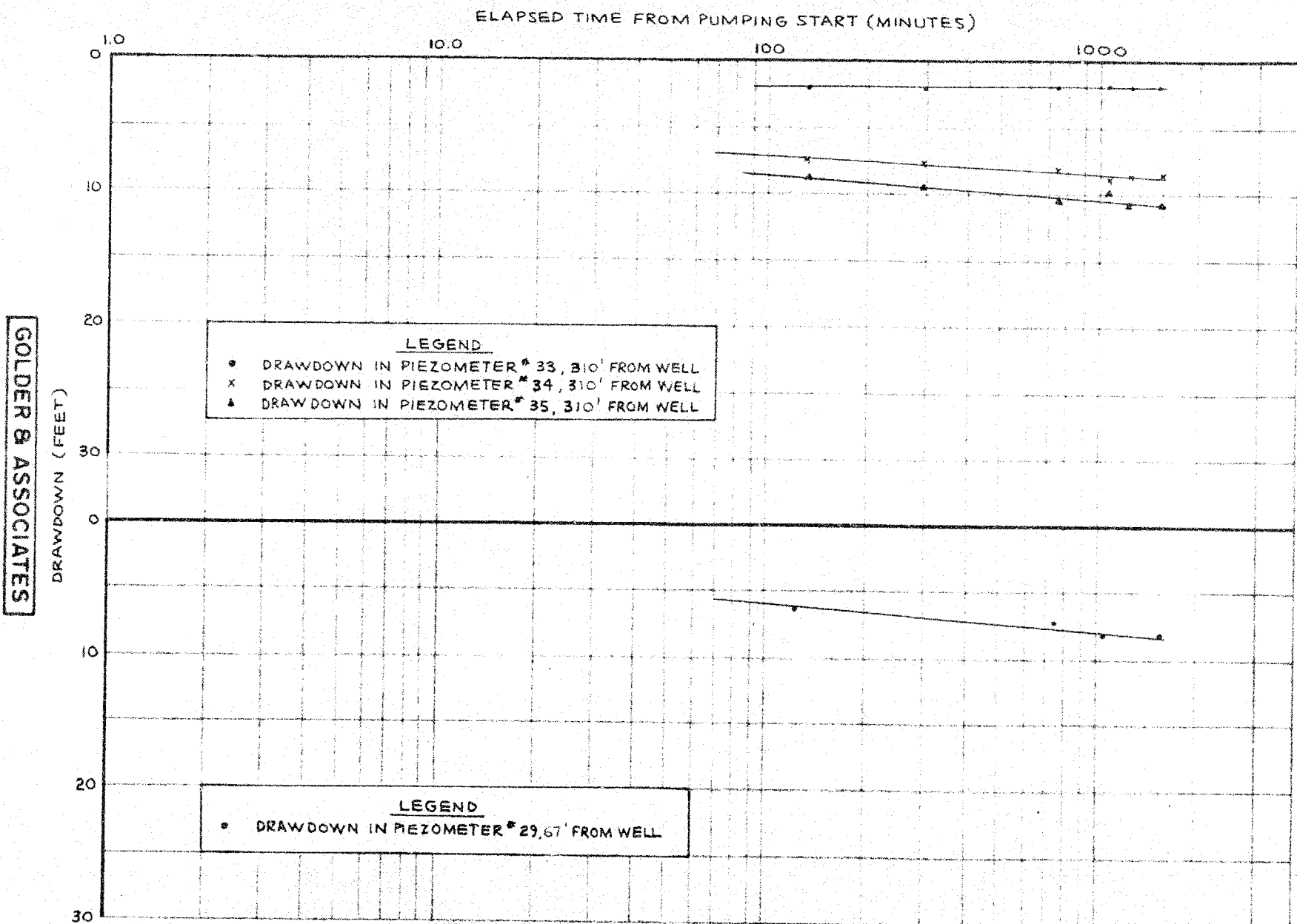
1000





RATE OF DRAWDOWN CURVES (BEDROCK)
PUMPING RATE $Q = 101$ I.G.P.M. (SECOND PUMPING)

FIGURE 2-32



ELAPSED TIME FROM PUMPING START (MINUTES)

10.0

100

1000

LEGEND

- DRAWDOWN IN PIEZOMETER #8, 318' FROM WELL

LEGEND

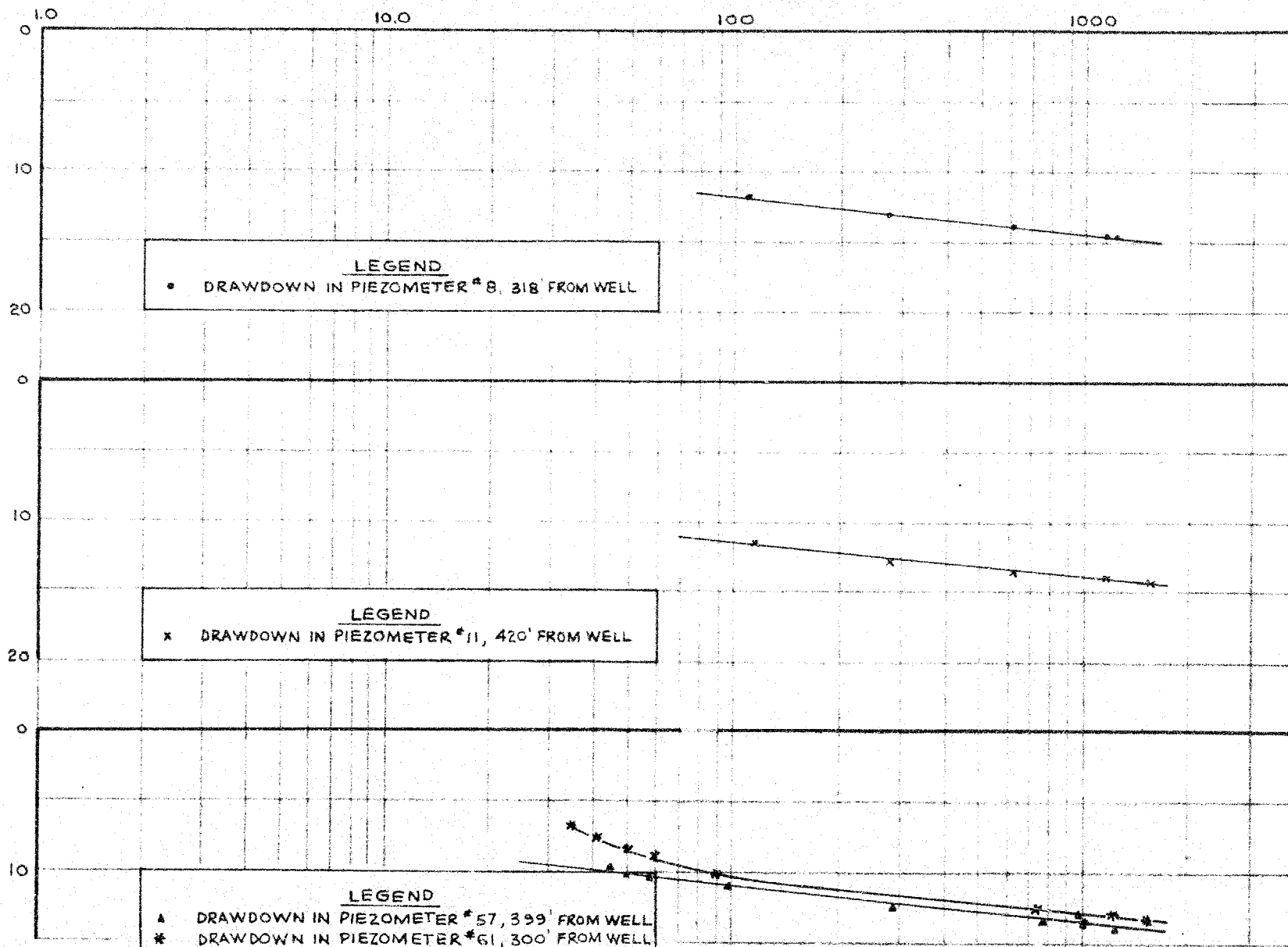
- x DRAWDOWN IN PIEZOMETER #11, 420' FROM WELL

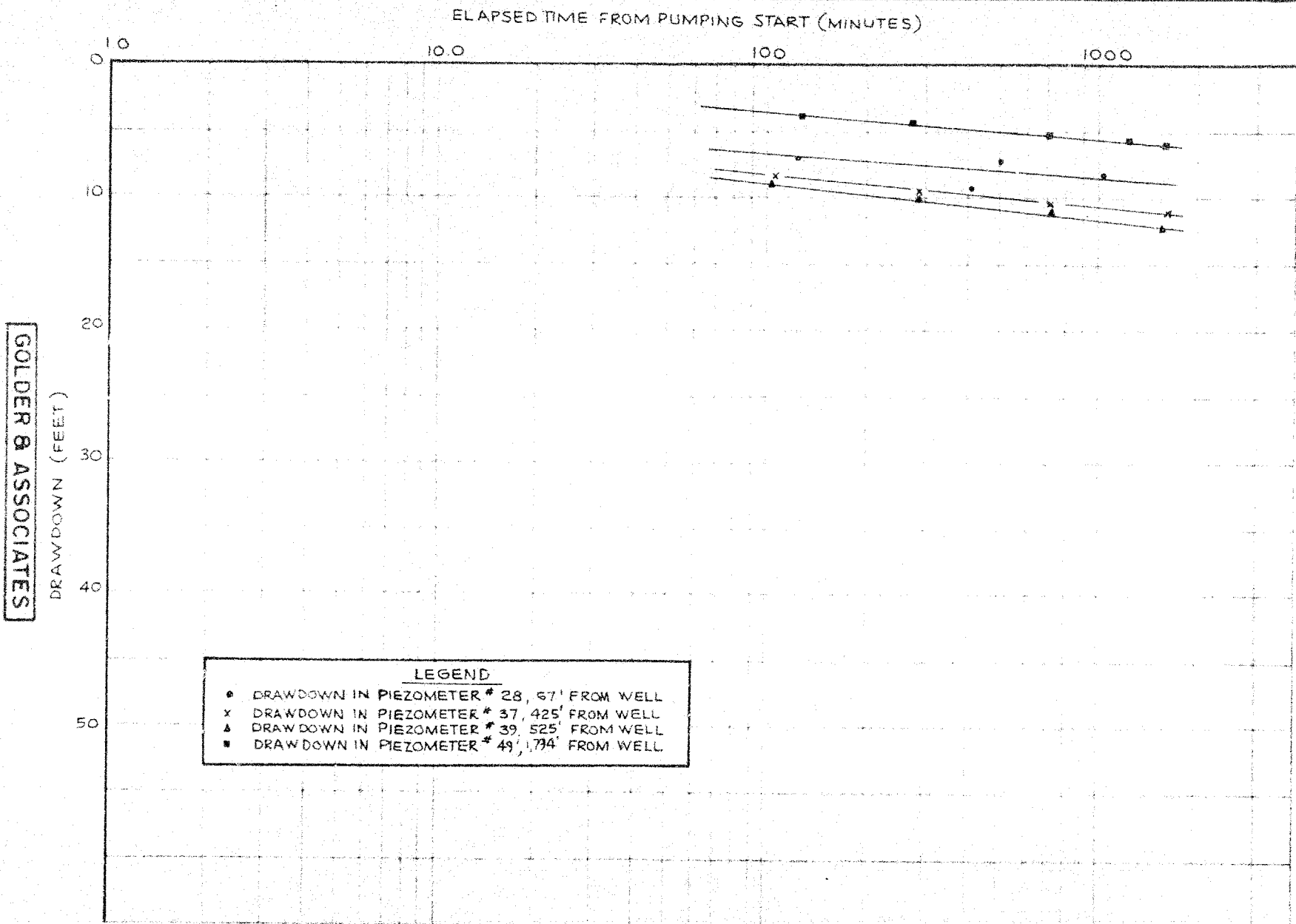
LEGEND

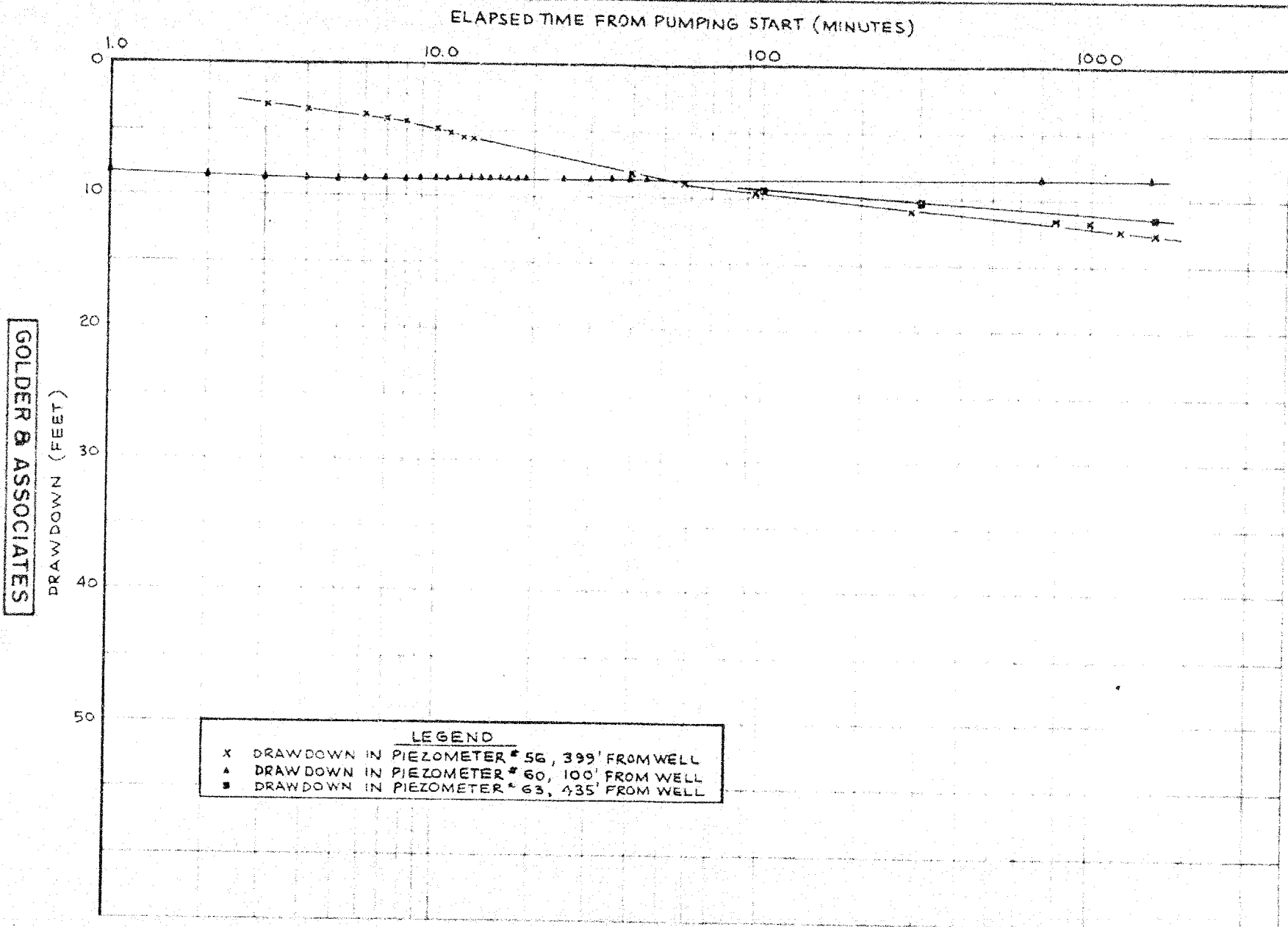
- ▲ DRAWDOWN IN PIEZOMETER #57, 399' FROM WELL
- * DRAWDOWN IN PIEZOMETER #61, 300' FROM WELL

GOLDER & ASSOCIATES

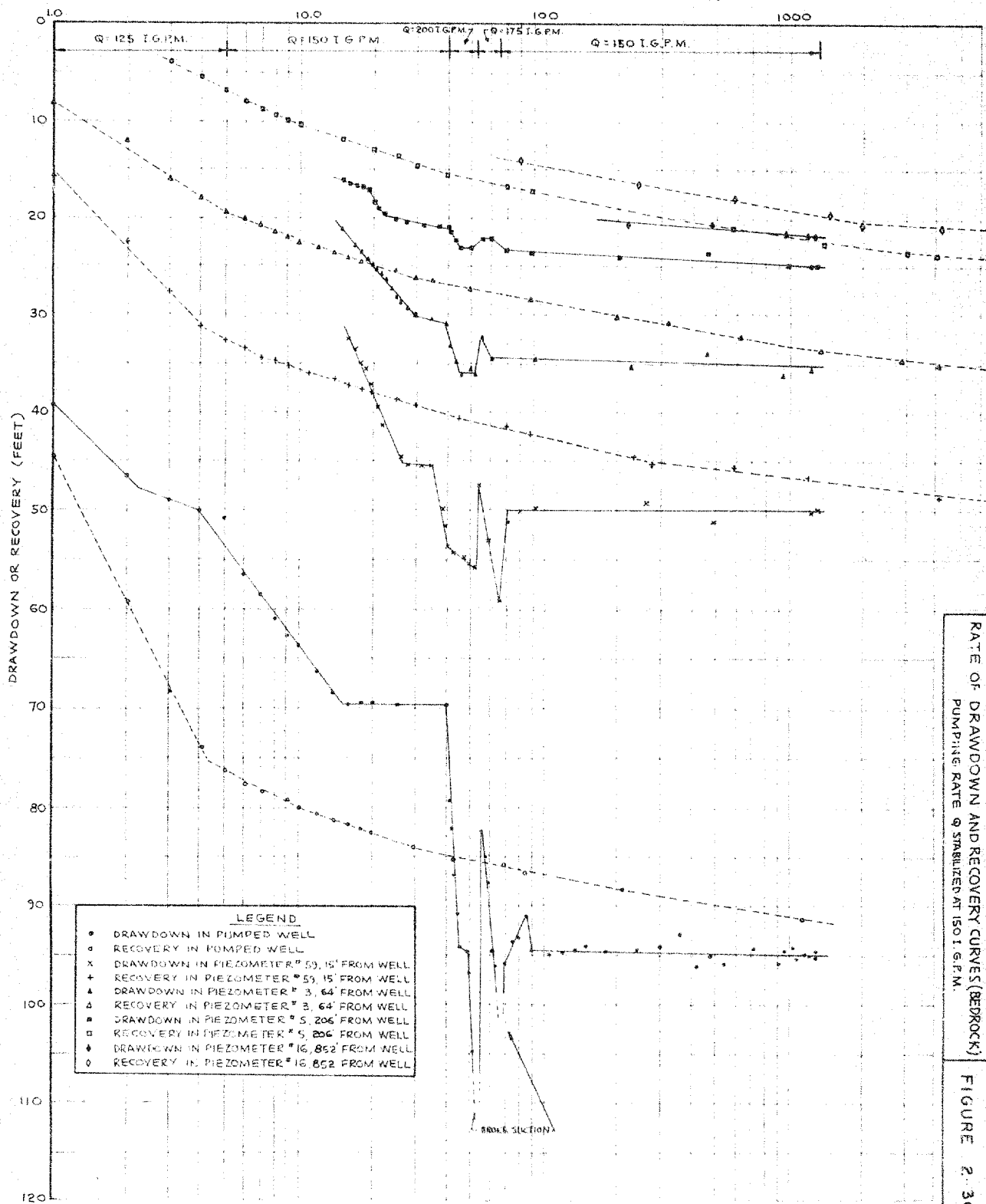
DRAWDOWN (FEET)







ELAPSED TIME FROM PUMPING START OR STOP (MINUTES)

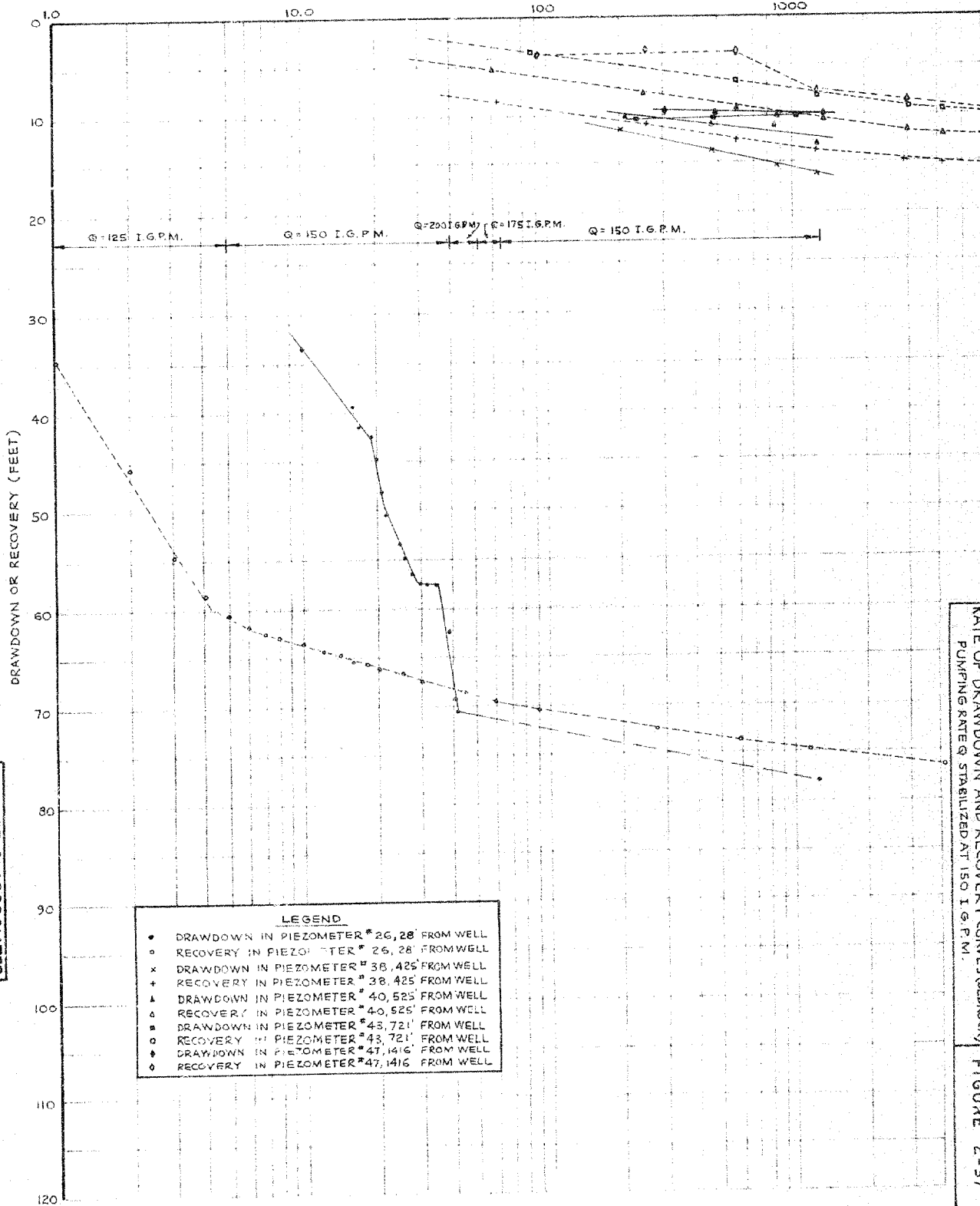


GOLDER & ASSOCIATES

RATE OF DRAWDOWN AND RECOVERY CURVES (BEDROCK)
PUMPING RATE Q STABILIZED AT 150 I.G.P.M.

FIGURE 2-36

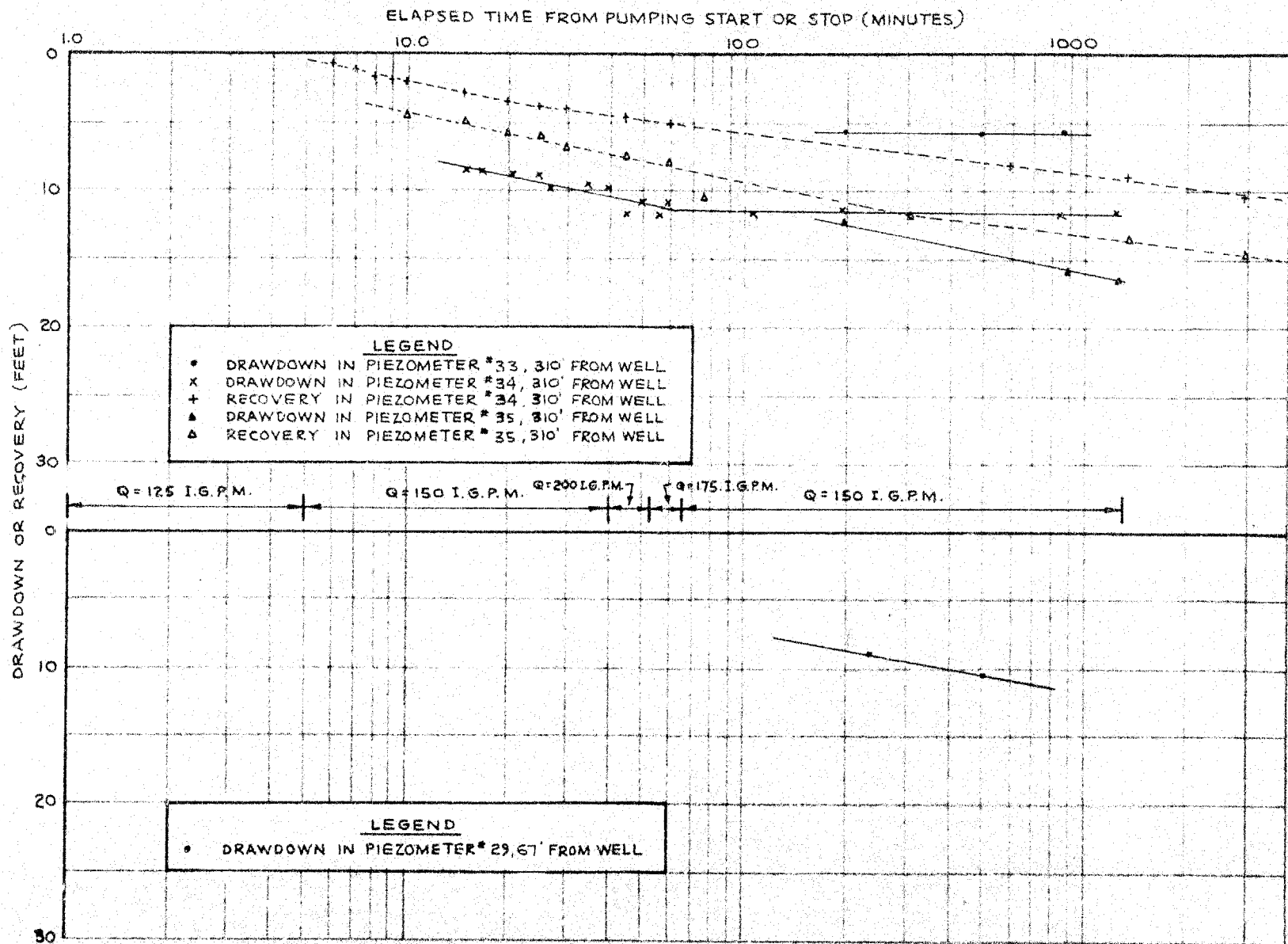
ELAPSED TIME FROM PUMPING START OR STOP (MINUTES)



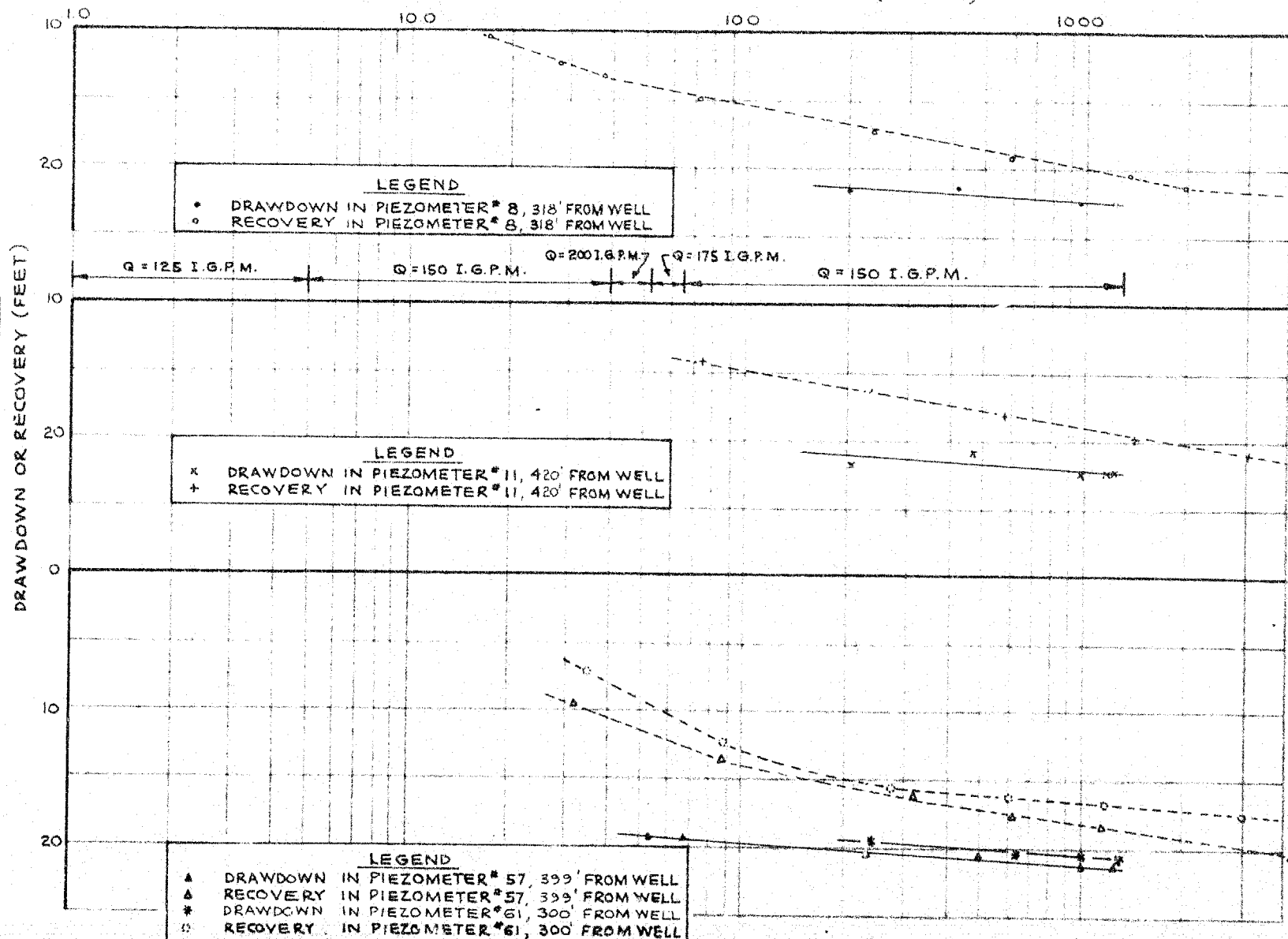
GOLDER & ASSOCIATES

RATE OF DRAWDOWN AND RECOVERY CURVES (BEDROCK)
PUMPING RATE STABILIZED AT 150 I.G.P.M.

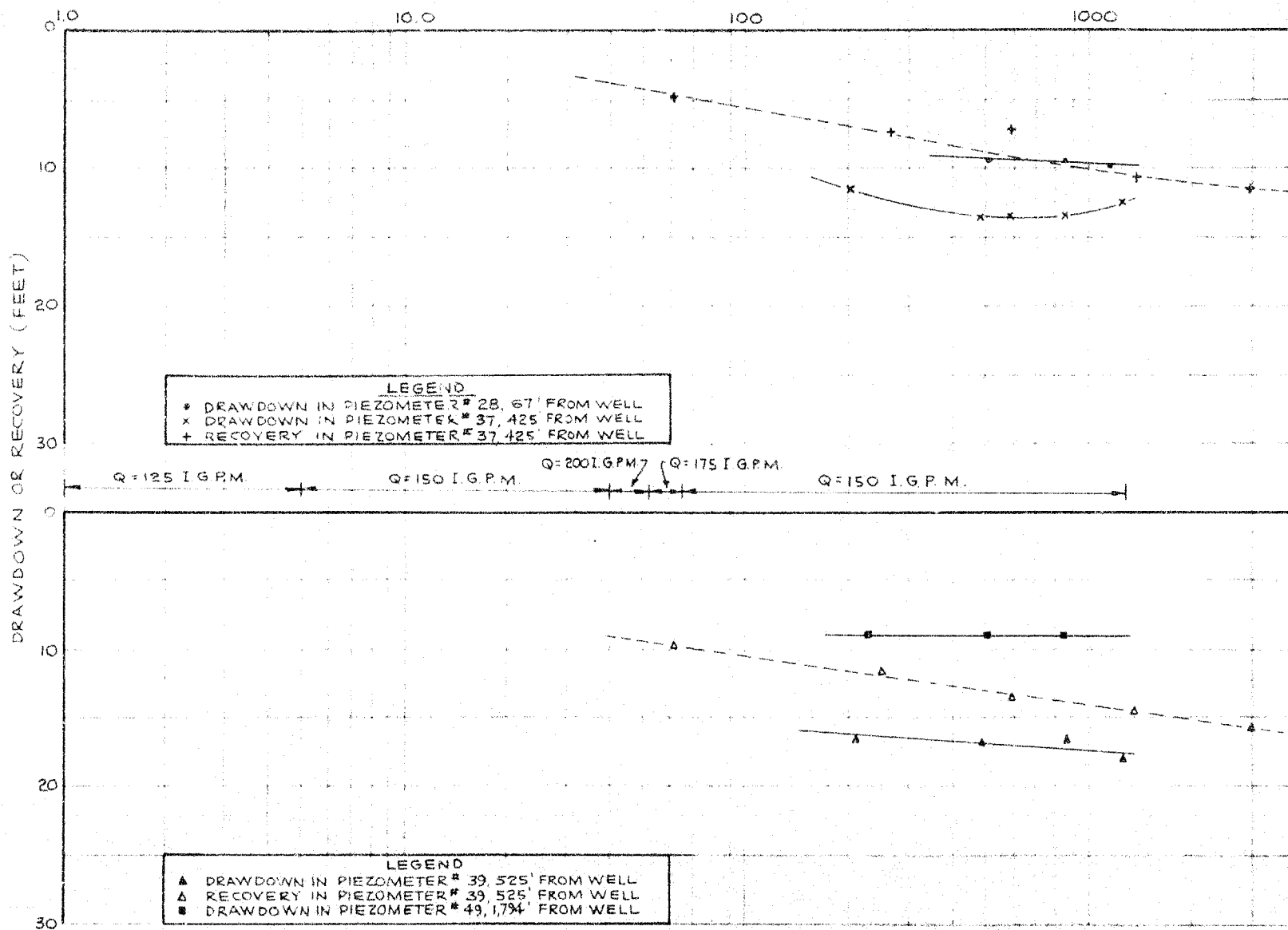
FIGURE 2-37



ELAPSED TIME FROM PUMPING START OR STOP (MINUTES)

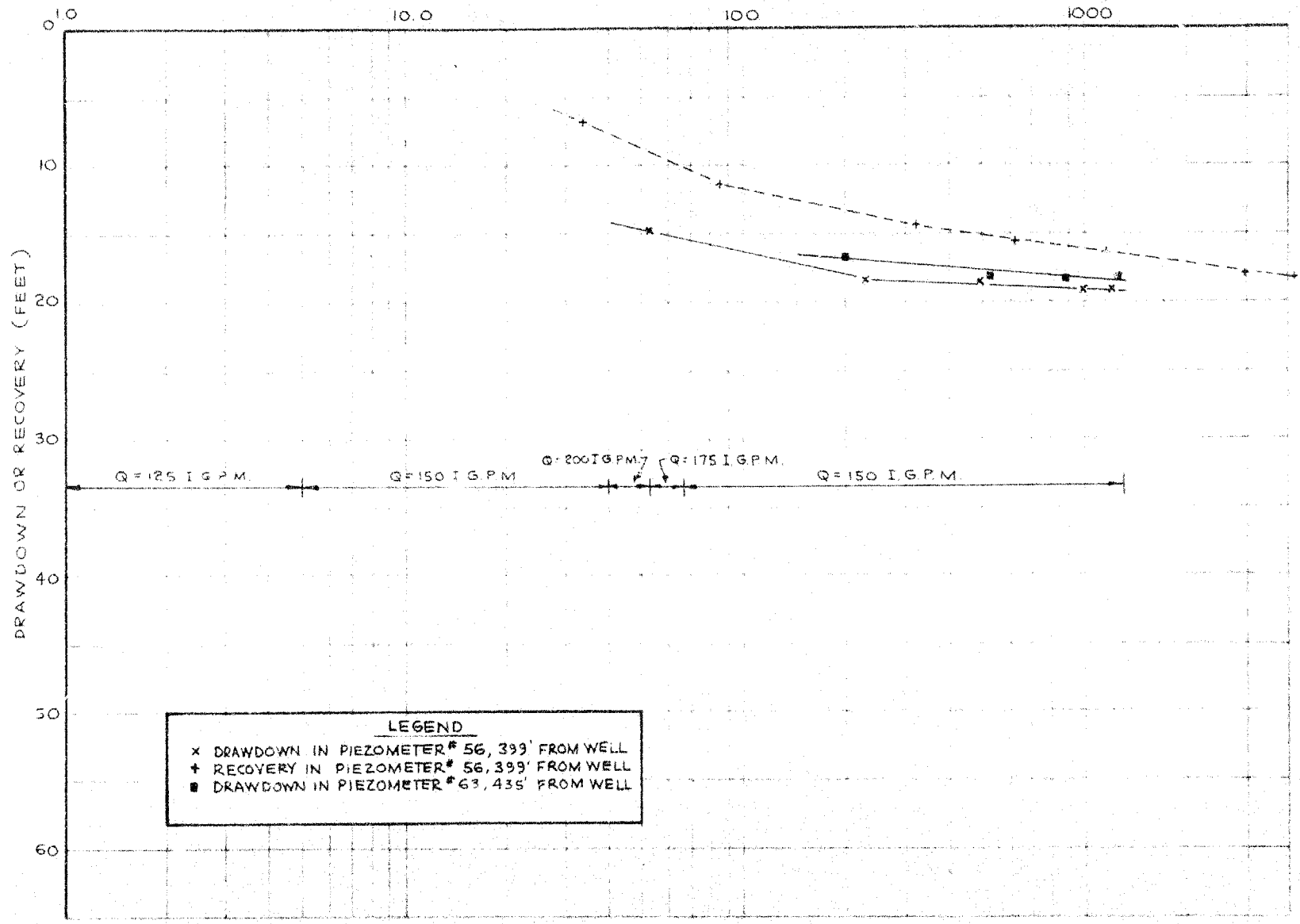


ELAPSED TIME FROM PUMPING START OR STOP (MINUTES)



ELAPSED TIME FROM PUMPING START OR STOP (MINUTES)

GOLDER & ASSOCIATES



RATE OF DRAWDOWN AND RECOVERY CURVES (TILL)
PUMPING RATE Q STABILIZED AT 150 I G P M

FIGURE 2-41

