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STATION VICKSBURG MS GEOTECHNICAL LAB W E NORTON
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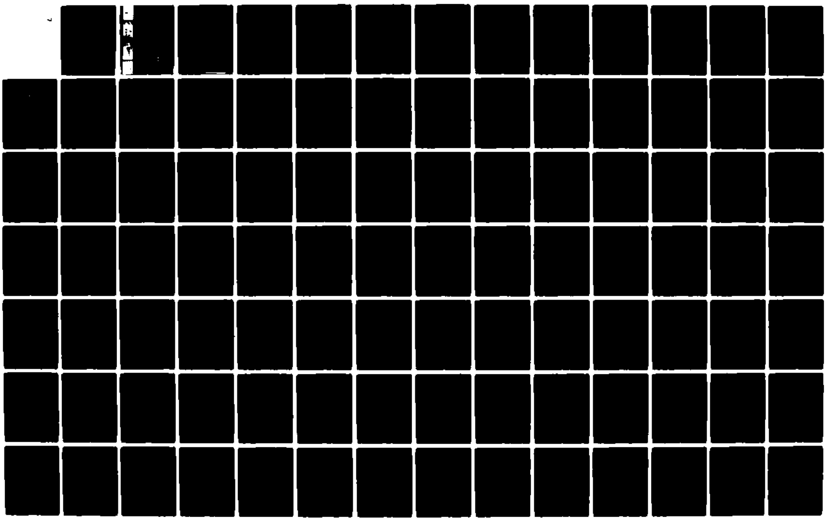

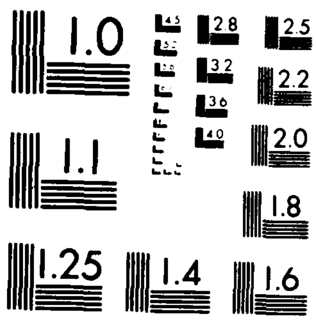

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TECHNICAL REPORT GL-83-15

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IN SITU DETERMINATION OF LIQUEFACTION POTENTIAL USING THE PQS PROBE

by

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Final Report

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report documents the field testing of a penetration device, the PQS probe, capable of simultaneously measuring penetration resistance, friction resistance, and pore pressure response. The probe is evaluated as a tool to measure liquefaction related soil characteristics in situ. Of special interest is the pore pressure response during penetration and whether it is diagnostic of contractive or dilative behavior of cohesionless soils and thus their liquefaction potential. In addition, a procedure using the penetration resistance, q , to assess liquefaction potential is presented.

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Evaluation of the pore pressure data recorded during penetration led to the following conclusions:

a. Positive pore pressures in situ are likely to occur in nonliquefiable sands even though they tend to dilate in shear. The PQS probe could not distinguish between liquefiable and nonliquefiable soils on the basis of positive or negative pore pressure response. The original hypothesis failed.

b. Negative pore pressures were observed in two situations not related to contractive or dilative behavior, one during the penetration of partially saturated soils above the water table and the other during temporary halts to add additional push rods.

c. The excess pore pressure ratio, u/q, and the friction ratio, f_s/q, behaved similarly in cohesive deposits and appear to be a reliable index to nonliquefiable material. However, since the PQS probe does not produce a sample for evaluation, the test is not conclusive.

A comparison of the PQS field test results with SPT data indicated that q is a reasonable measure of N, and it can be used to evaluate the liquefaction potential of a soil by using the simplified procedure. Three sites were evaluated using the POS probe, and it was found that the procedure worked well when the simplified procedure was appropriate. Since the quantity q is used in the analysis, any cone penetrometer that produces that quantity can be used. The advantage of using the PQS probe is that it adds the capability of determining the elevation of the ground water table and provides an indicator of cohesive soils.

PREFACE

This work was performed during the period November 1978 to September 1983 by the U. S. Army Engineer Waterways Experiment Station (WES) for the Office, Chief of Engineers (OCE), U. S. Army, under CWIS Work Unit 31619, "Development of a Technique and/or Device to Evaluate the Liquefaction Potential of In-Situ Cohesionless Material," for which Mr. R. F. Davidson was the OCE Technical Monitor.

The Montz, Louisiana, field work was carried out by Messrs. J. P. Koester, S. S. Cooper, D. H. Douglas, D. E. Yule, and S. W. Guy, Earthquake Engineering and Geophysical Division (EEGD), Geotechnical Laboratory (GL), WES. The Imperial Valley field work was carried out by Dr. A. G. Franklin, EEGD, and Messrs. S. S. Cooper, J. P. Koester, D. H. Douglas, and S. W. Guy. The study was performed under the direct supervision of Dr. A. G. Franklin, Chief, EEGD, and under the general supervision of Dr. W. F. Marcuson III, Chief, GL. This report was written by MAJ W. E. Norton, EEGD.

COL Tilford C. Creel, CE, was Commander and Director of WES during the period of this study. Mr. Fred R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
degrees Fahrenheit	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
inches	2.54	centimetres
miles (U. S. statute)	1.609347	kilometres
pounds (force)	4.448222	newtons
pounds (force) per square inch	6894.757	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (force) per square foot	95.76052	kilopascals

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9)(F - 32). To obtain Kelvin (K) readings, use: K = (5/9)(F - 32) + 273.15.

IN SITU DETERMINATION OF LIQUEFACTION POTENTIAL USING THE PQS PROBE

PART I: INTRODUCTION

Purpose

1. A new CWIS work unit was initiated in FY 79 with the purpose of developing methods for the evaluation of the liquefaction potential of cohesionless soils through in-situ measurements. These measurements could be either empirical correlations between liquefaction potential and data from conventional tests or could be measurements of liquefaction-related soil characteristics from new devices. The study was to include field and laboratory evaluation of in-situ test devices or methods such as the Standard Penetration Test (SPT), the Dutch resistivity sonde, and the Wissa piezometer probe. This report documents the investigation of the piezometer probe as an in-situ testing device to determine the liquefaction potential of cohesionless soils.

Background

2. At the American Society of Civil Engineers (ASCE) 1975 Geotechnical Engineering Division Specialty Conference on In-Situ Measurement of Soil Properties, separate papers were submitted by Anwar Wissa (Wissa, Martin, and Garlanger, 1975) and Bengt-Arne Torstensson (1975) describing penetrating probes capable of measuring the dynamic pore pressures generated during penetration. Both probes had the same end area as the Dutch cone (10 cm^2). The theory advanced at that conference was that the penetration of a probe through soil generated a pore pressure field around the tip (Schertmann (1975), Torstensson (1975), and Wissa, Martin, and Garlanger (1975)). It was stated that in loose sands and weak clays the pore pressures developed would be positive due to the collapsing nature of the soil. In dense sands and stiff clays the pore pressures would be negative due to the dilatancy of the soil.

3. It was postulated that if a piezometer probe could determine whether a soil was collapsible or dilative, then the probe might be effective in

determining whether a deposit of cohesionless material was susceptible to liquefaction. Judging from the literature, this approach had not been attempted, and the U. S. Army Engineer Waterways Experiment Station (WES) commissioned a pilot study to review the feasibility.

Pilot Study

4. Dr. John H. Schmertmann (1978) conducted a pilot study to test the feasibility of using the piezometer probe to determine liquefaction potential. He did it in two phases:

   a. Field tests using both the Wissa probe and a special piezometer probe constructed by the University of Florida.

   b. Lab tests using the Wissa probe.

5. Phase I of the study successfully demonstrated the ability of the piezometer probe to measure dynamic pore pressures which could be correlated with tip resistances obtained from Dutch cone data. In this study both positive and negative pore pressures were observed in the field investigation, but an interpretation could not be made becasue of the confusing effects of permeability and dilation on pore pressure. The positive pore pressures were observed in a loose sand dredge tailings deposit, and the negative pore pressures were observed in an underlying undisturbed material thought to be more silty (but never confirmed). Because of the combined effects of (a) "elastic" compression under load, (b) tendency for volume change in shear, (c) permeability, and (d) drainage path and pore pressure generation, two materials could give the same response although one was more resistive to liquefaction than the other.

6. Phase II of the study consisted of continued evaluation of the University of Florida and Wissa probes in the laboratory. Five soundings were made in the University of Florida test chamber on Reid Bedford Model Sand (RBMS). This is a sand that contains less than 1 percent passing the No. 200 sieve. Both loose and dense samples were constructed and tested, and very small pore pressures were observed in all of the tests. The lab data could not provide enough information to determine whether liquefaction potential could be evaluated from the dynamic pore pressures, but it did point out the following limitations of working in the lab:

a. Chamber boundary conditions seriously affect pore pressure development.

b. Saturation of the samples was computed to be 97 percent, and lack of complete saturation degrades pore pressure response.

c. The permeability of the test sand was considerably different than the sands in the field.

It was concluded that the laboratory tests did not help in the interpretation of the field data and did not alter the earlier conclusion of the field tests.

7. The potential value of the piezometer probe was recognized, and the enhancement of the Fugro-type friction cone with pore pressure capability was envisioned as offering an instrument of great capability. WES constructed such a device, called the PQS probe (Cooper and Franklin, 1982), and developed the equipment and techniques to operate it in the laboratory and the field.

8. The evaluation of the piezometer probe was felt to be a field problem because of the numerous uncertainties involved. Professor Schmertmann had demonstrated that the inability to set boundary conditions, determine the effects of dilation and permeability, and ensure saturation had hampered the interpretation of lab data. In-situ testing would eliminate the boundary condition and saturation problems since natural boundary conditions would exist and soils in nature are generally saturated if below the water table. The volume change and permeability characteristics are something that can usually be determined by laboratory testing. Thus, empirical correlation appeared to offer the best means of evaluating the potential of the PQS probe.

9. For field evaluation three sites were chosen. The first site at Montz, La., was chosen because the WES was conducting an extensive site investigation of a major flow slide in the Mississippi River, and a full range of tests were being conducted. The flow slide was caused by an unknown mechanism at first thought to be liquefaction by the nature of the failure. The other two sites were in the Imperial Valley in California at sites where extensive liquefaction had occurred during the 1979 earthquake. At these sites a detailed subsurface investigation was being conducted by the U. S. Geological Survey (USGS).

10. This report documents the field trials conducted by WES and evaluates the PQS probe in terms of its strengths and weaknesses. In addition, a method to use the probe for rapid evaluation of liquefaction potential in situ is presented.

PART II: FIELD TESTING

11. The push equipment and data acquisition equipment are described by Cooper and Franklin (1982). The push rig was limited to 14,000-lb* capacity, which limited the depth of investigation. The data were recorded on a three channel strip chart recorder, and the strip charts for each hole were later digitized by tracing the curves manually on a graphics tablet. For general use this system could be automated. For research, the complete record was desirable, and the strip chart was adequate; however, use of the strip chart did impede the evaluation of data in the field.

12. The data obtained during the test consists of the analog readings from three separate load cells. The three readings are defined as follows

- a. q is the penetration resistance of the tip measured, tsf.
- b. P is the total pore pressure, psi.
- c. f_s is the frictional resistance per unit surface area of t friction sleeve, tsf.

The values of q and f_s are equivalent to, and interchangeable with, values obtained by a conventional electric friction cone.

13. From the data obtained, two ratios are computed:

- a. f_r (friction ratio) $= \frac{f_s}{q}$, dimensionless.
- b. $\frac{u}{q}$ (pore pressure ratio) $= \frac{P - \text{Hydrostatic Pressure}}{q}$, dimensionless.

14. The presentation of the data is done with charts of q, p, f_r, and u/q versus depth. The computer program that does the plotting automatically scales each record to obtain maximum definition.

Montz, La., Field Tests

15. The Montz, La., site was selected for field evaluation because it was the site of a massive flow during the high water on the Mississippi River in 1973. It was originally thought that the mechanism that triggered the slide was liquefaction, and the WES had been commissioned by the Lower Mississippi Valley Division, U. S. Army Corps of Engineers, to study the phenomenon. The

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

work since 1976 had been directed at applying current state-of-the-art techniques to investigate the in-situ characteristics of the susceptible deposits. Thus, an excellent opportunity to field test the PQS probe was presented. The location of the Montz site is shown in Figure 1.

16. The investigation at Montz was detailed and employed a full range of tests, including the SPT, continuous undisturbed samples for laboratory testing, cone penetration tests (CPT), resistivity cone tests (RC), piezometer probe measurements (W), and nuclear density measurements. A detailed site layout is shown in Figure 2. The results of this investigation were reported by Torrey and Peterson (1981), and the site description and test data that follow are drawn from that report.

17. Figure 3 is a profile of the site. This cross section is an idealization of the soil profile that exists where the PQS probe soundings were made. On the site layout in Figure 2, the PQS holes are identified by the designation F-3 and F-4 and were close to boring SPT-2. The complete logs of the two PQS holes are included in this report in Appendix A.

18. Figure 4 shows the results of SPT hole No. 1. The N values have been adjusted to an effective overburden pressure of 1 tsf by the relationship $N_1 = N \times C_n$, where C_n is a function of the effective overburden pressure at the depth where the penetration tests were conducted (Seed and Idriss, 1981). In addition, the D_{90}, D_{50}, and percent passing the No. 200 sieve are indicated. A summary of the grain sizes for all tests can be seen in Figure 5. The sand is a poorly graded fine sand that gradually gets coarser with depth. Numerous layers of silty sand to silt lenses, probably discontinuous, are located throughout. These lenses may vary in thickness up to a few tenths of a foot and are identified by low blow counts, low penetration resistance, or in X-rays taken of the undisturbed samples.

19. Continuous 3-in.-diam thin-walled Shelby tube samples to a depth of 120 ft were obtained using the Osterberg sampler. These samples were allowed to drain in an upright position, were frozen in the same position, and were then transported to the WES and stored in an environmental room at 20°F. Each sample was X-rayed in order to select the best specimens for laboratory testing. Twenty-two specimens were selected and tested. The test performed was the stress controlled, anisotropically consolidated undrained triaxial shear test with pore pressure measurement and is described in Appendix 10 of EM 1110-2-1906 (U. S. Army, Office, Chief of Engineers, 1970). The test is commonly

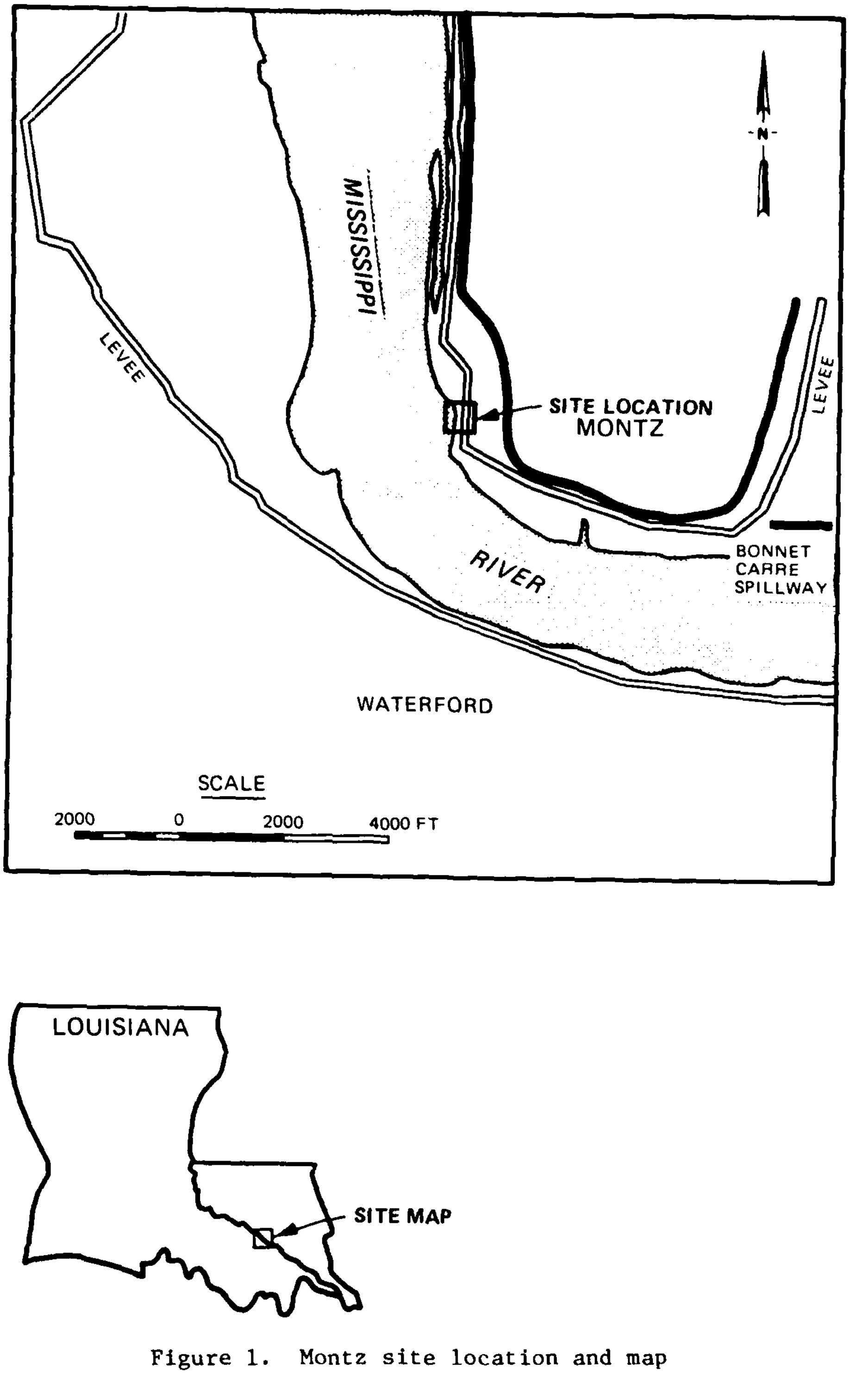


Figure 1. Montz site location and map

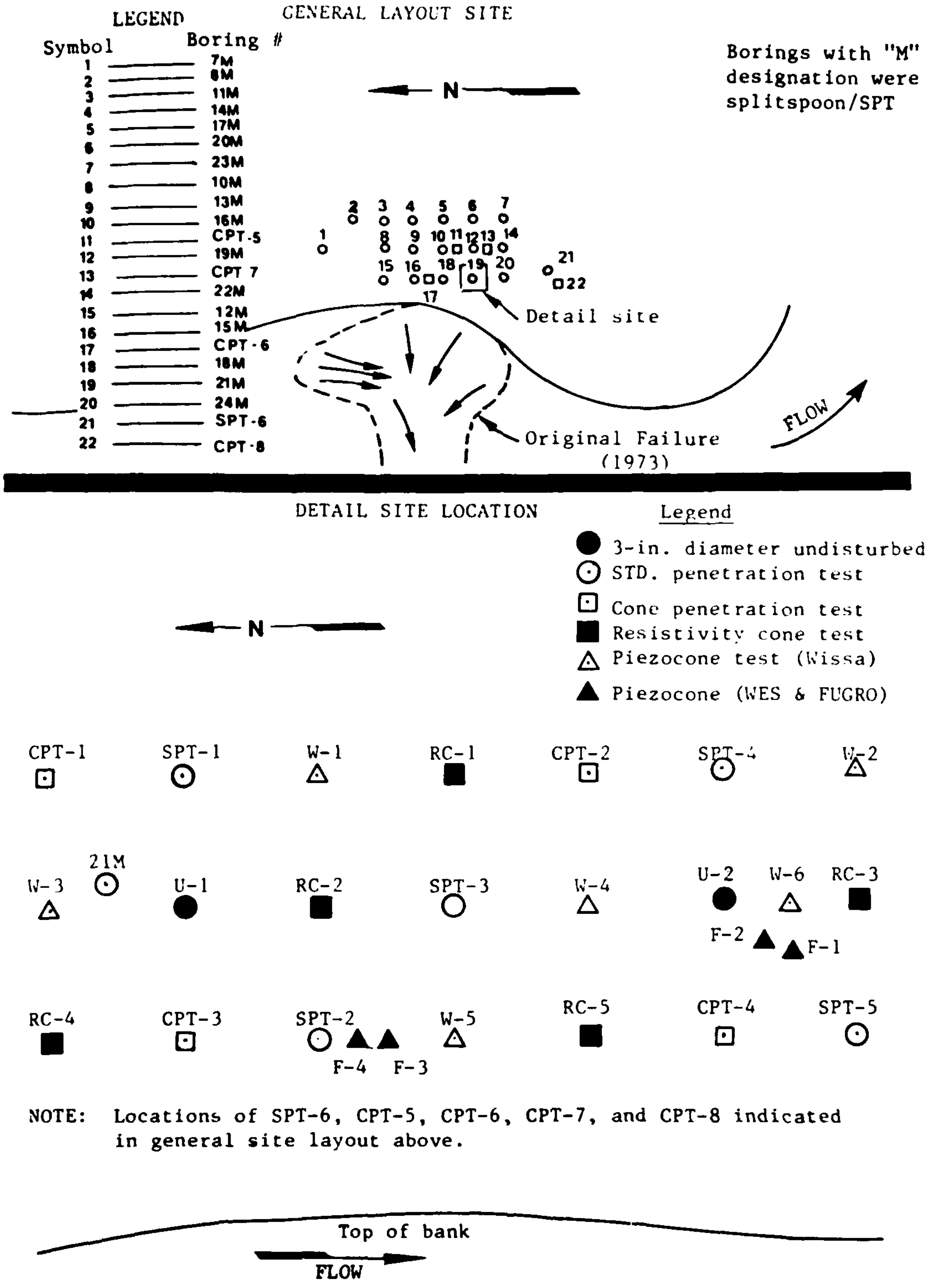


Figure 2. General and detail sites, Montz, La.
(after Torrey and Peterson, 1981)

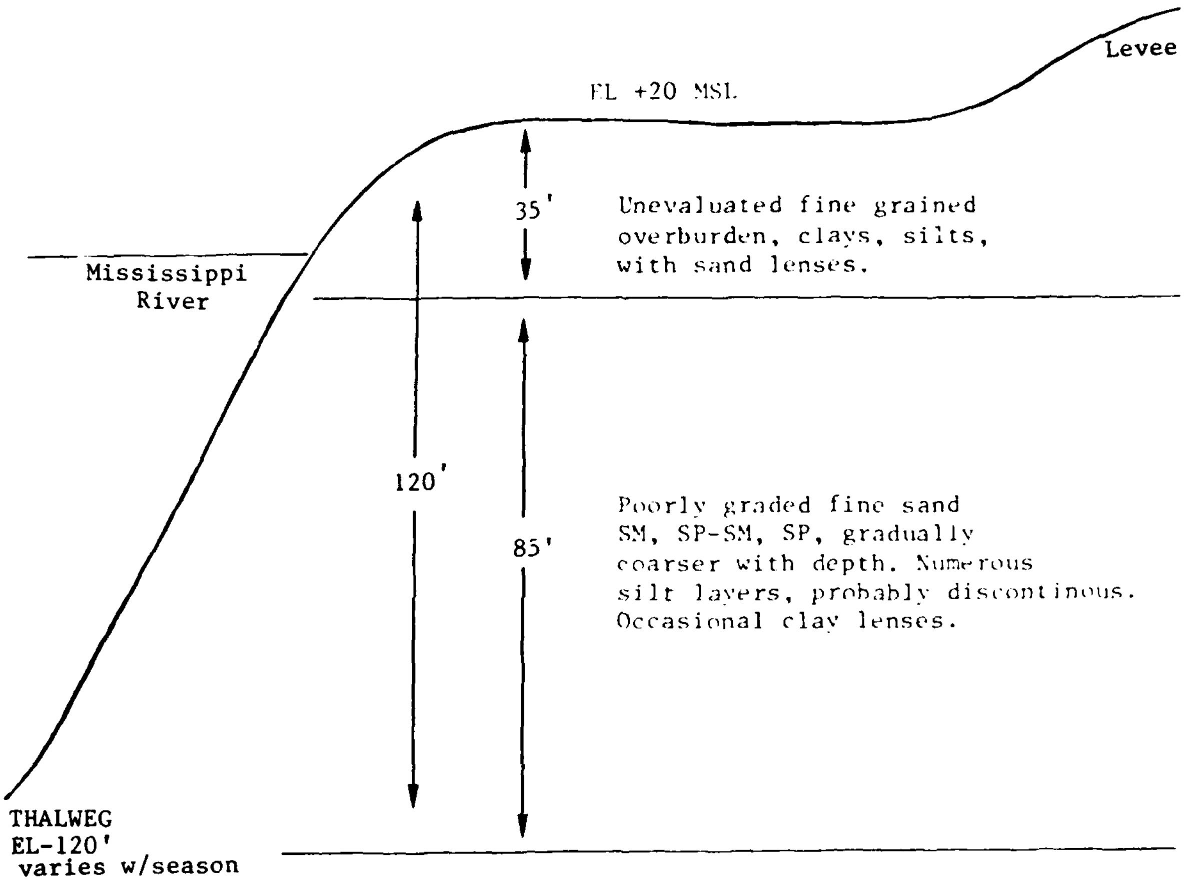
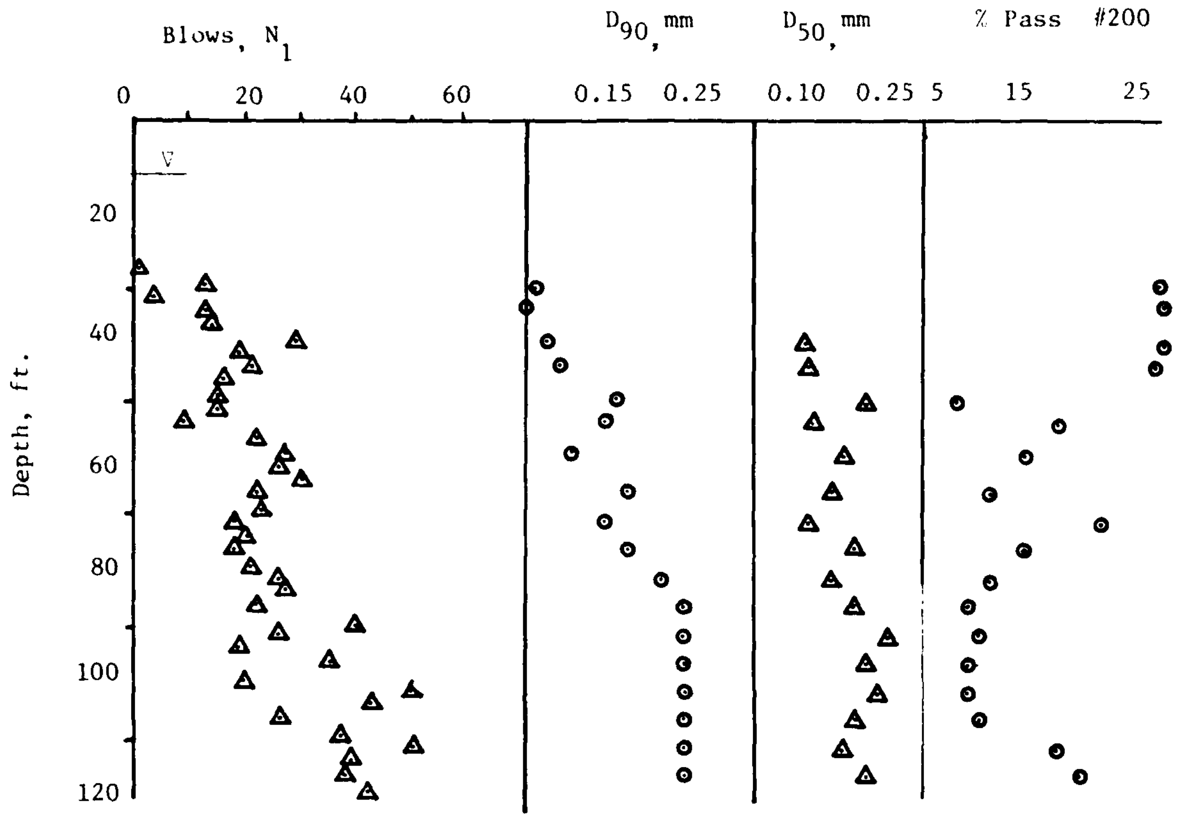


Figure 3. Montz site profile



$N_1 = N \cdot C_N$

$C_N = 0.77 \text{ Log } (20/\sigma_v')$ (Torrey and Peterson, 1981)

Figure 4. Montz SPT-2 (after Torrey and Peterson, 1981)

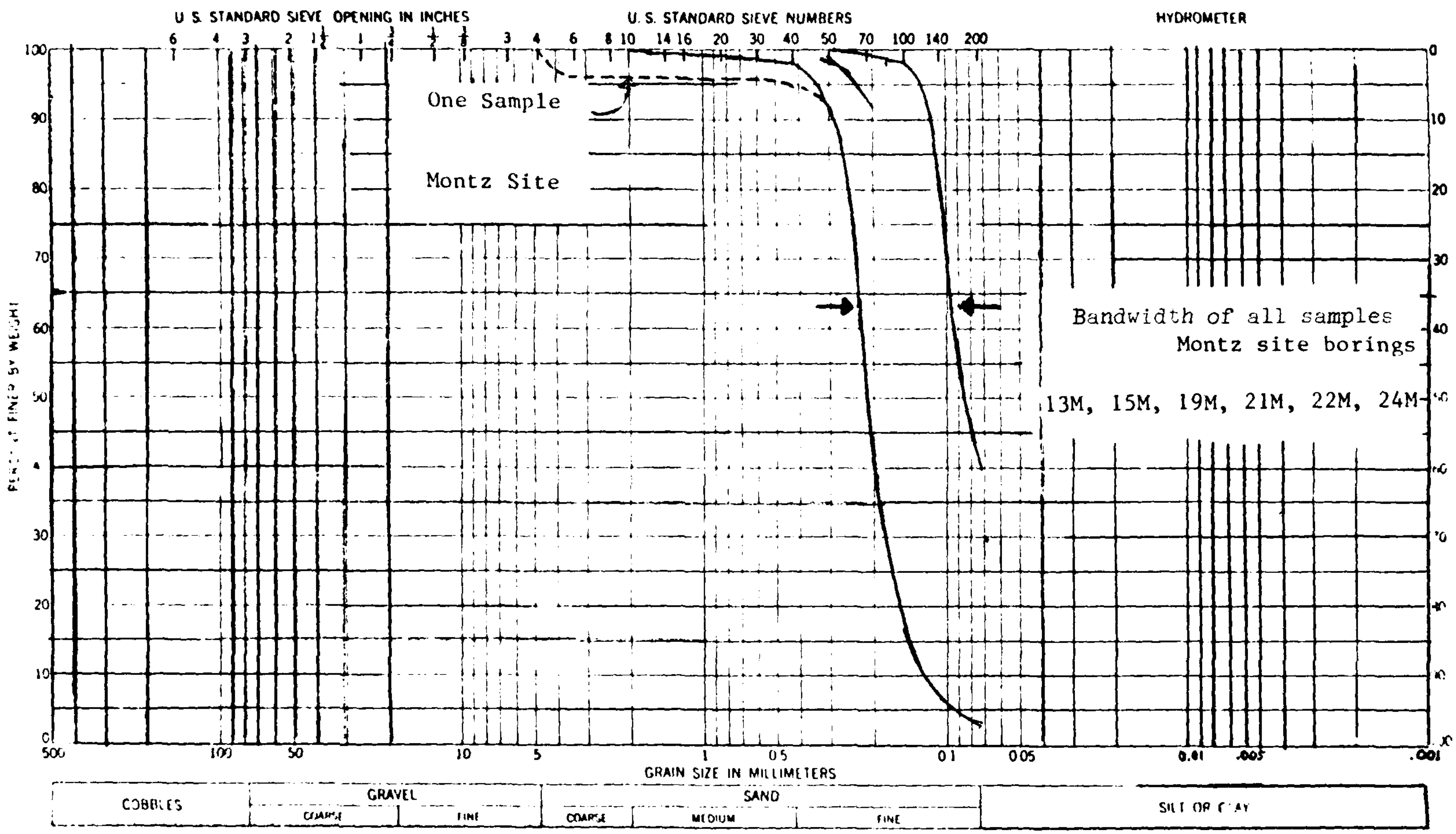


Figure 5. Range of Zone A sand gradation, Montz site

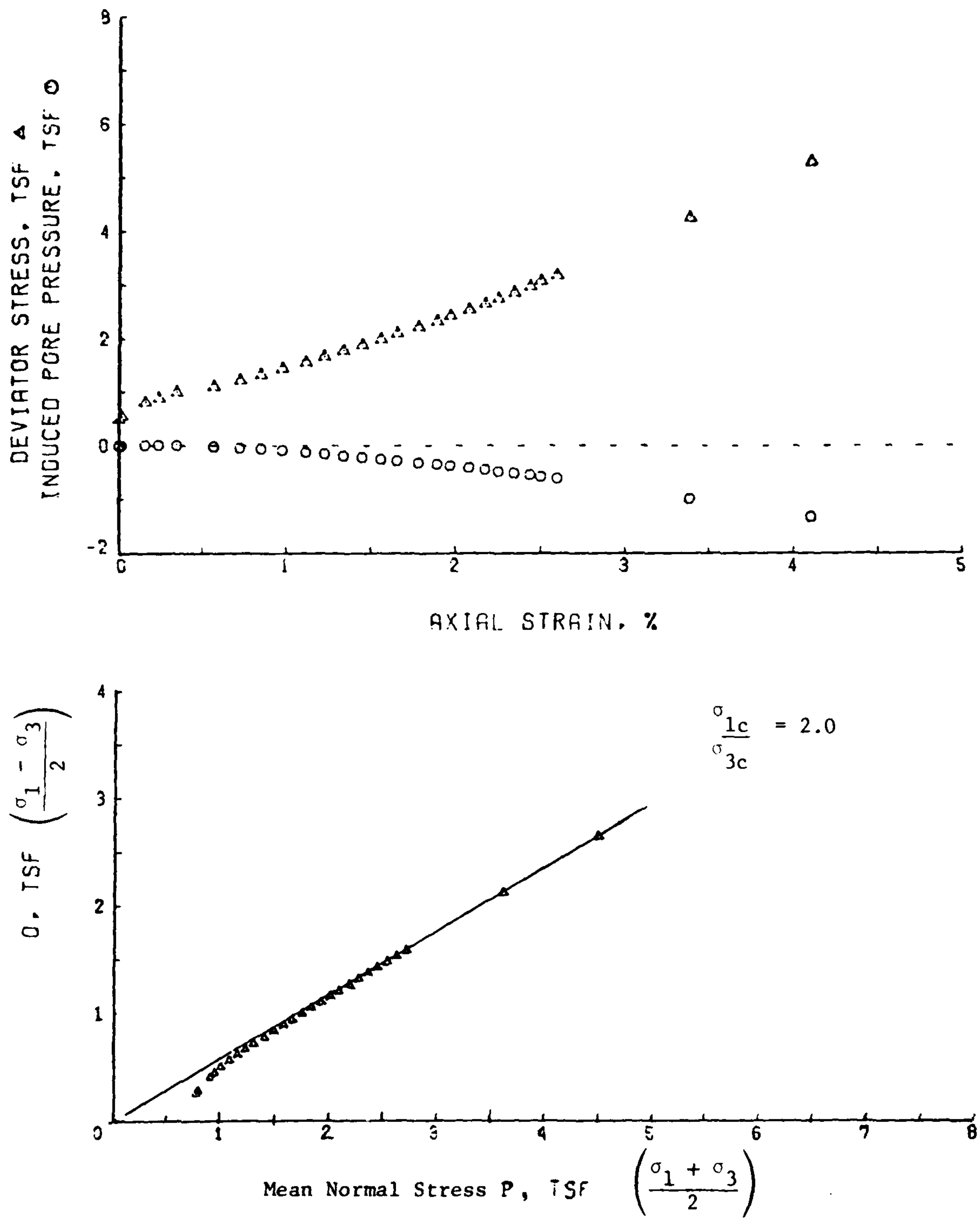
referred to as the $\bar{R}$ (pronounced R-bar) test. The results of the test indicated that the effective angle of internal friction, ϕ' , was between 33 and 36 deg and that all of the specimens tended to dilate under applied confining pressures approximating those of the depth the samples were taken, a very important point. A sample test result is presented in Figure 6.

20. The results of all of the tests on undisturbed samples indicated that the sands ranged in relative density from a minimum of 40 percent to more than 60 percent. This corresponds to a range of void ratios from 0.87 to 0.70. Figure 7 presents the test results that show that the critical void ratio in the composite Montz sands are all well above what exists in situ. The fact that the sands were all dilative during triaxial testing and that their in-situ void ratio is less than the critical void ratio lead to the conclusion that classical liquefaction was not the causative mechanism of the flow slide. Other phenomena are under consideration and the flow slide study continues; what is important to this study is that the sands are dilatant, and if the hypothesis is correct that dilative sands produce negative pore pressures during penetration, then the PQS probe should experience negative pore pressures in the sand deposits at Montz.

21. The two holes pushed at the Montz site with the PQS probe in general show good agreement with what was expected from the developed profile as far as q and f_r are concerned. The following observations concerning the pore pressure response are made:

- a. Positive pore pressures are built up and maintained in the overburden above the pointbar deposit evaluated in the flow slide study. The water table was observed to be near the surface and thus excess pore pressures developed very quickly.
- b. The sands begin to appear at about 20 ft. Where sands occur, pore pressure development becomes very small. Positive pore pressure magnitude variation occurs consistent with the interbedding of materials.
- c. The sands at 40 ft are dilative according to the lab tests (Figure 6), yet the pore pressure development is positive. In fact, the sands throughout are dilative according to laboratory testing but still produce positive pore pressures during penetration.

22. The conclusion reached from this field investigation was that the penetration phenomena are more complex than originally thought and not solely a matter of contractive or dilative materials. Apparently, cone penetrometer testing produced positive pore pressures in many types of materials. However,



MONTZ U2-14-1 DEPTH - 40.5 FT DRY DENSITY - 92.5 PCF

Figure 6. Sample results of the stress-controlled consolidated undrained tests (after Torrey and Peterson, 1981)

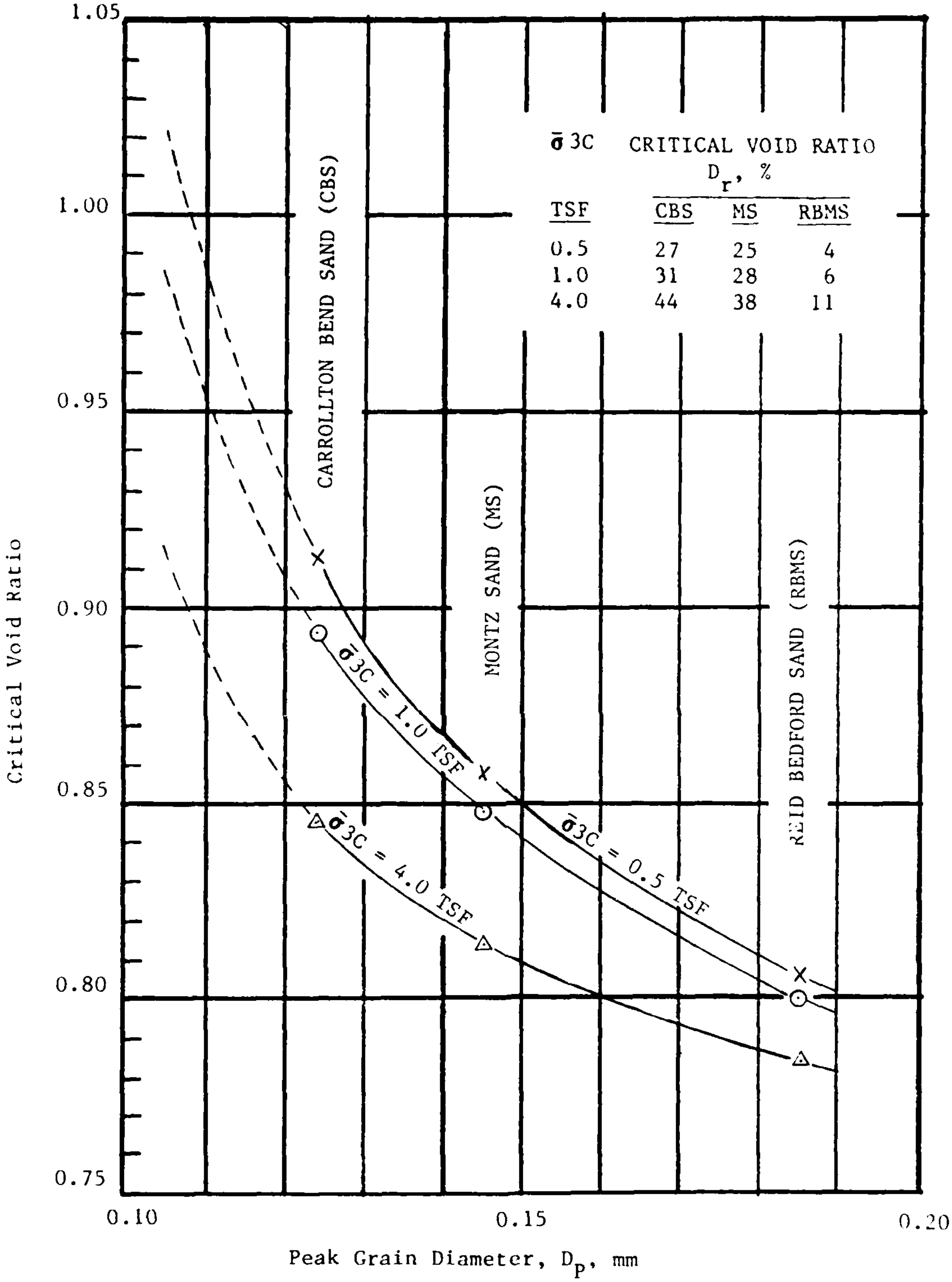


Figure 7. Critical void ratio versus peak grain diameter (after Torrey and Peterson, 1981)

whether the PQS probe could determine liquefaction susceptibility in situ was not resolved because it was determined that liquefaction had not occurred at Montz.

Imperial Valley Field Tests

23. On 15 October 1979 the ground surface ruptured suddenly along a 19.3-mile segment of the Imperial Valley fault with additional movement along approximately 8 miles of the Brawley fault. The movement caused an earthquake of magnitude 6.6 on the Richter scale. The locations of the Imperial Valley fault and Brawley fault are given in Figure 8.

24. The Imperial Valley is a region of frequent seismic activity and has been under intense study by the USGS and other institutions for a number of years. A comprehensive accelerograph network capable of recording strong motions in both the earth and in structures had been installed. The 15 October earthquake produced a large amount of data on earth shaking that is still being analyzed. It also produced liquefaction at numerous sites in the valley.

25. T. L. Youd of the USGS in Menlo Park, Calif., chose two sites where liquefaction had been particularly pronounced and conducted extensive subsurface investigations to develop detailed soil profiles. The two sites were the River Park site in Brawley, Calif., and the Heber Road site southeast of El Centro, Calif. The site descriptions that follow and the data on the geologic conditions are drawn from Bennett et al. (1979), and Youd and Bennett (1981).

Heber Road tests

26. In the geologic past a lacustrine environment existed at the Heber Road site. The lake level rose and fell and lacustrine clays were interbedded with sands deposited by meandering channels. Superimposed on the alternating lacustrine and fluvial deposits was a sequence of deltaic sands deposited at the mouth of a stream entering the lake. One such delta formed at the mouth of an ancient stream 1 to 2 km west of the present course of the Alamo River. An old channel marks the former course of the stream and Heber Road crosses the old channel (Youd and Bennett, 1981). Liquefaction of loose sand in the old channel caused sand boils, lateral spreading, and ground cracking at the crossing, and shifted Heber Road 1.2 m southward.

27. Youd developed a detailed cross section of the site using SPT, CPT, disturbed continuous sampling, and thin-walled tube samples. Youd's definitions

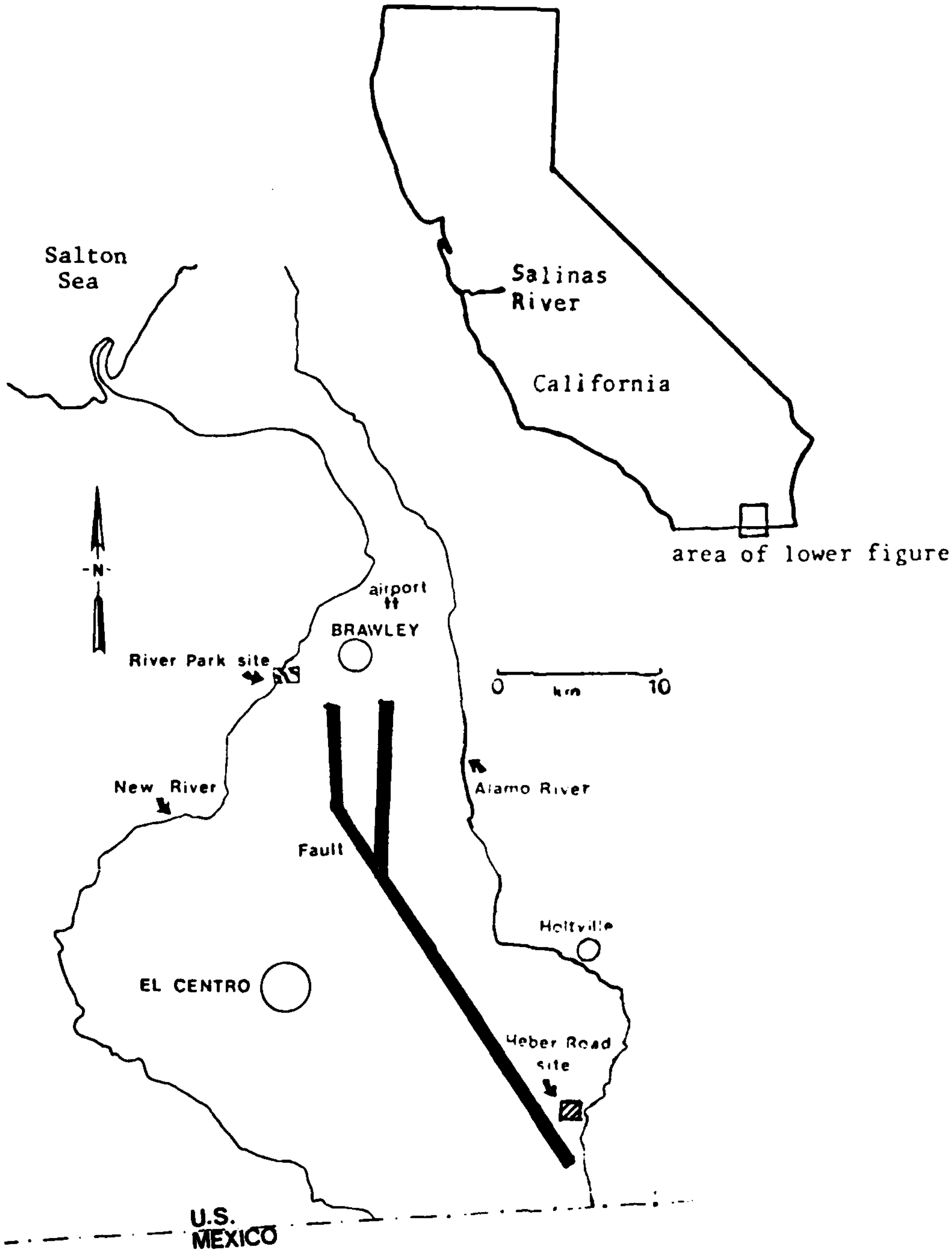


Figure 8. Southern Imperial Valley with Heber Road and River Park sites (after Youd and Bennett, 1981)

of stratigraphic units in the geologic profile will be followed here for convenience. Figure 9 shows the location of the PQS holes in reference to the USGS holes which lie along the same centerline. Figure 10 shows the profile of the sediments at Heber Road with the location of the WES PQS holes indicated. Figure 11 gives the generalized characteristics of the sediments.

28. The three units of concern are units A_1, A_2, and A_3, since units B and D are nonliquefiable clays and unit C could not be penetrated by the 14,000-lb push capacity of the PQS push rig used. Unit A is medium dense to dense pointbar deposit consisting of thinly bedded fine sand. Unit A_2 is a channel fill consisting of a very loose fine sand, and unit A_3 is a medium dense fine sand thought to be an overbank deposit. Youd and Bennett (1981) performed a simplified liquefaction analysis and found that unit A_1 was not susceptible to liquefaction, that unit A_2 was, and that unit A_3 was marginal depending on the corrections applied to the blow count. Surface evidence supported this analysis as there was no evidence of liquefaction of unit A_1, while there was ample evidence that liquefaction had occurred in unit A_2. The only observed evidence of liquefaction in unit A_3 was over a buried pipeline where the soil had been disturbed.

29. The penetration logs of each of the PQS holes are shown in Appendix B. In general, the logs agree well with the USGS profile. Since A_1 and A_3 probably did not liquefy and unit A_2 did, an evaluation of the PQS probe's ability to distinguish between the two cases could be made.

30. Unit A_1 was penetrated by PQS holes 1, 4, 5, 6, and 10. Refusal occurred rapidly in holes 1 and 10 due to the limited push capacity of the push rig. At refusal, both holes were showing positive pore pressures. Holes 4, 5, and 6 penetrated unit A_1 and continued to refusal in unit C. In all three holes small positive pore pressures were seen in the unit.

31. Unit A_2 was penetrated by holes 2, 2a, 3, 4, 5, 6, and 7. In general, holes 4, 5, 6, and 7 show the generation of small positive pore pressures. In holes 2, 2a, and 3, negative pore pressures were developed in the random fill above unit A_2. These negative pore pressure were generated in a partially saturated loose material above the water table, and once the probe passed the water table positive pore pressures developed.

32. Unit A_3 was penetrated by holes 8 and 9. Both holes showed positive pore pressure development in the unit.

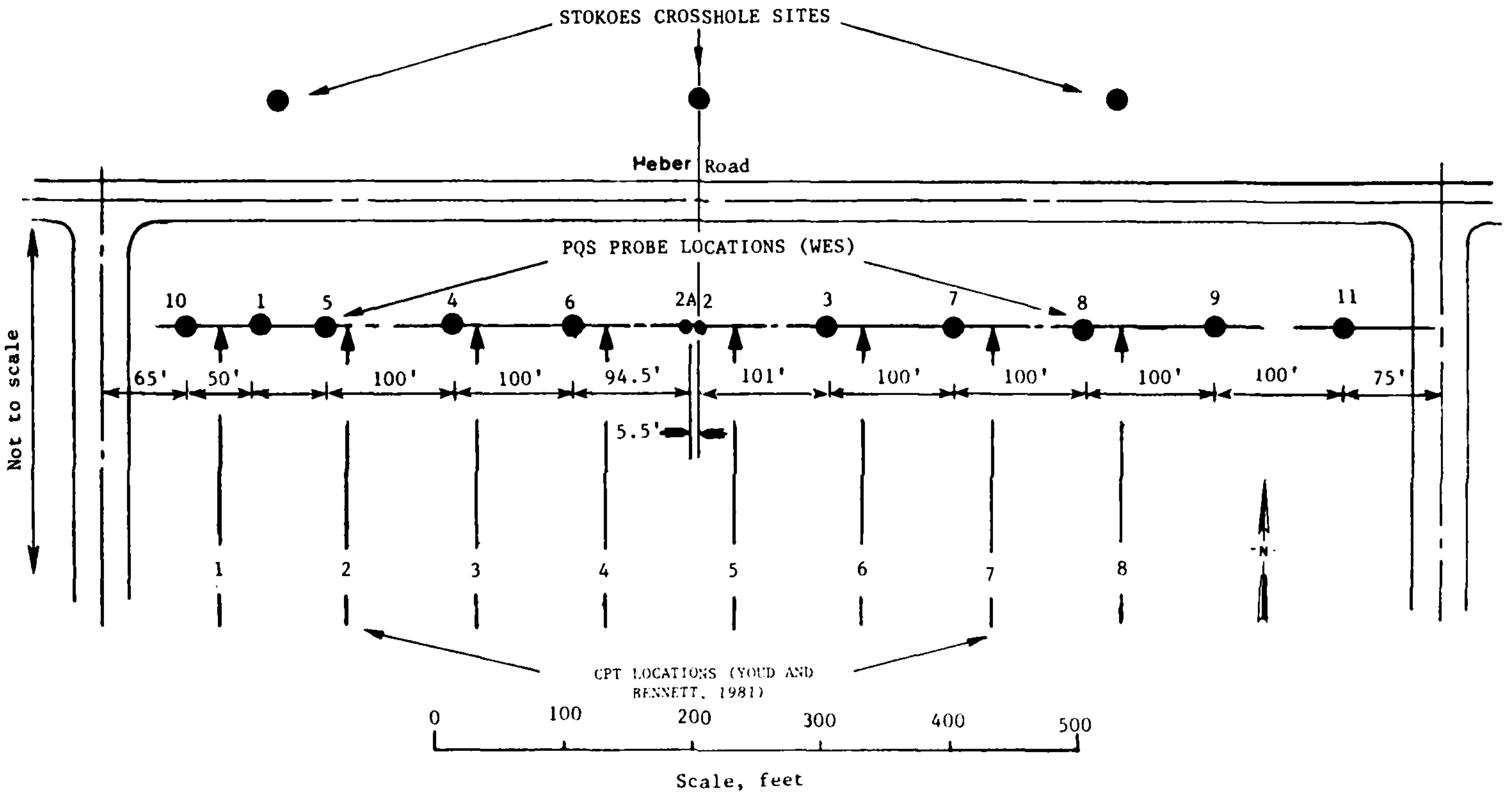


Figure 9. PQS probe hole locations

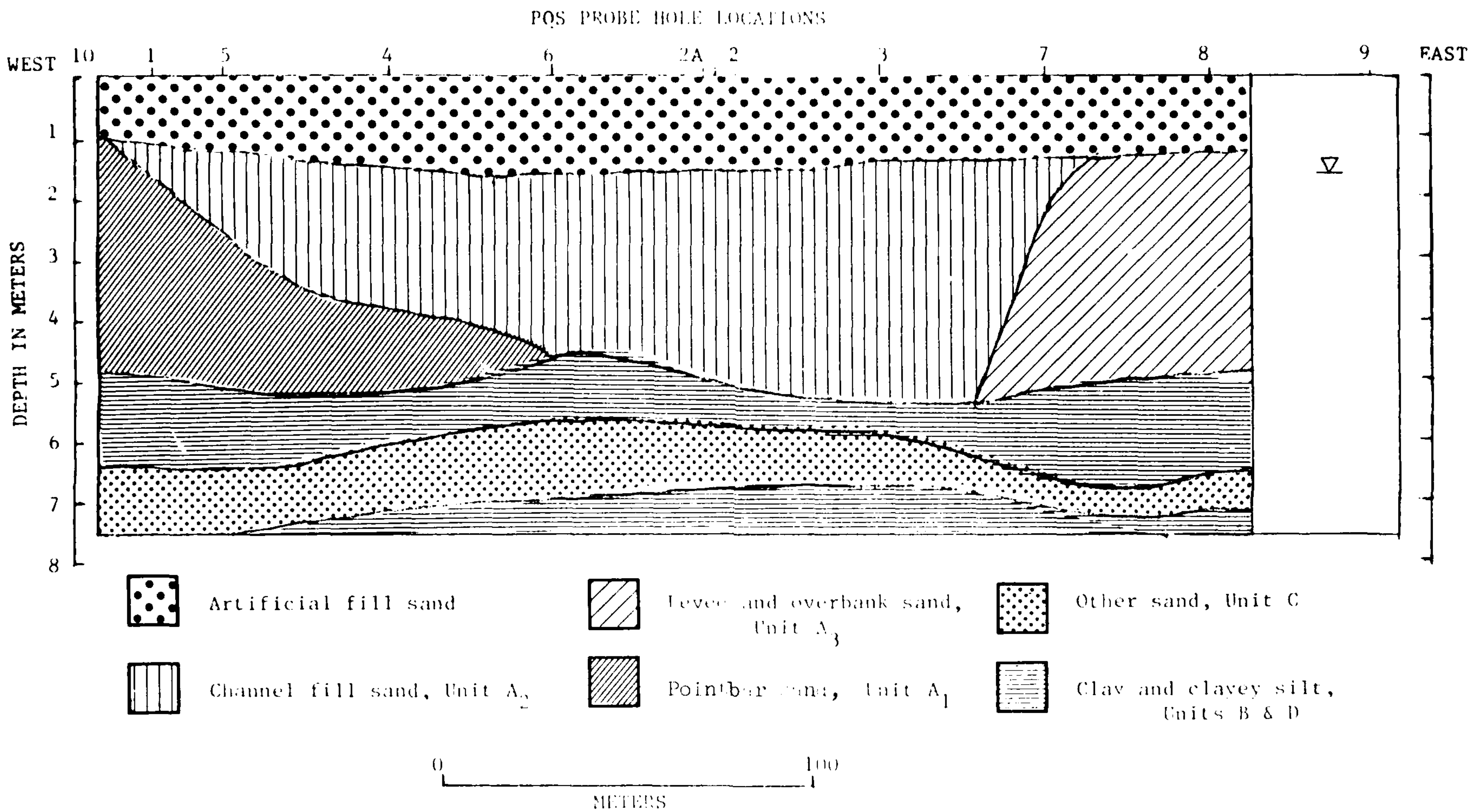


Figure 10. Profile of sediments at Heber Road sites. Units A and A_3 represent dense and medium sand deposits, respecti ely. Unit A_2 is the channel fill sand portions of which liquefied. The locations of the PQS probe holes are shown at the top of the profile (after Youd and Bennett, 1981)

UNITS	$\bar{N}$	$\bar{q}_c$	$\bar{R}_f$	$\bar{D}_r$	approximate depth, in meters
Fill	5	22	3.0	52%	0
A_1- top * fine sand	12	75	3.40	80%	1.5
A_1- bottom * fine sand	31	160	2.87	119%	1.8
A_2- fine sand	4	20	2.76	23%	5
A_3- fine sand	11	49	2.46	69%	5
B- clay	8	22	3.36	---	5
C- fine sand	23	169	2.56	105%	6
D- clay	13	27	4.00	---	7
					9

$\bar{N}$ = *Average N, in blows per foot*
q : Average cone resistance, in kg/cm^2
f_r= Average ratio, in percent
$\bar{D}_r$= Average relative density, in per cent

* The top part of unit A_1 is characterized by ripple bedding and medium dense sand. The bottom part of the unit is characterized by horizontal bedding and dense sand.

Figure 11. Generalized characteristics of sediment at Heber Road site. Data include average blows per foot (N), point resistance/friction ratio, and relative density (D_r)(after Bennett et al., 1979)

33. It is apparent that one cannot discern differences among sand units A_1, A_2, and A_3, based on the maximum dynamic pore pressure response or the pore pressure ratio as the sand deposits all behave about the same in this respect. However, pore pressure development is very distinctive in the lacustrine clay (unit B) and the partially saturated soil above unit A_2.

<u>River Park tests</u>

34. The River Park site lies in the floodplain of the New River and is the result of that environment. An investigation similar to the investigation at Heber Road was conducted and reported by the USGS (Youd and Bennett, 1981), and the site was selected by the WES for field testing the PQS probe. The layout of the PQS holes in relation to the centerline of the USGS holes is shown in Figure 12. The profile developed by Youd is shown in Figure 13 with the PQS holes properly located.

35. In Figure 14 unit A is very loose silty sand. Unit B is a silty clay typical of a backswamp deposit. Unit C is a medium dense fine sand typical of a pointbar deposit; however, the top of unit C was found to be considerably less dense than the bottom. Evidence of liquefaction showed on the surface with two distinct grain size distributions present in sand boils, one similar to unit A and one similar to unit C. Figure 14 shows the generalized characteristics of the units. A simplified analysis performed by Youd and Bennett indicated that all of unit A was susceptible to liquefaction and that only the upper few feet of unit C was.

36. The complete logs of the PQS holes at the River Park site are in Appendix C. In general, the PQS holes agree very well with the profile shown in Figure 13, except that the softness in the upper part of unit C reported by Youd is not indicated in the PQS logs. Local variation probably accounts for this since the PQS holes were offset 50 ft from the line at USGS holes.

37. In holes 1, 2, 3, 4, 5, and 6 it can be seen that the unsaturated sand and silt above the water table developed negative pore pressures. In holes 2, 3, 4, and 5 this tended to degrade the responsiveness of the probe. Generally the effect was not so serious as to destroy the usefulness of the sounding, but at River Park it was worse than at Montz or Heber Road. Extremely low tip resistances are seen in the lower half of unit A and throughout unit B indicating that both layers are very soft. The pore pressure response in unit A and unit C are very similar and one unit is not distinguishable from the other. Unit B does develop significant pore pressure response and is easily

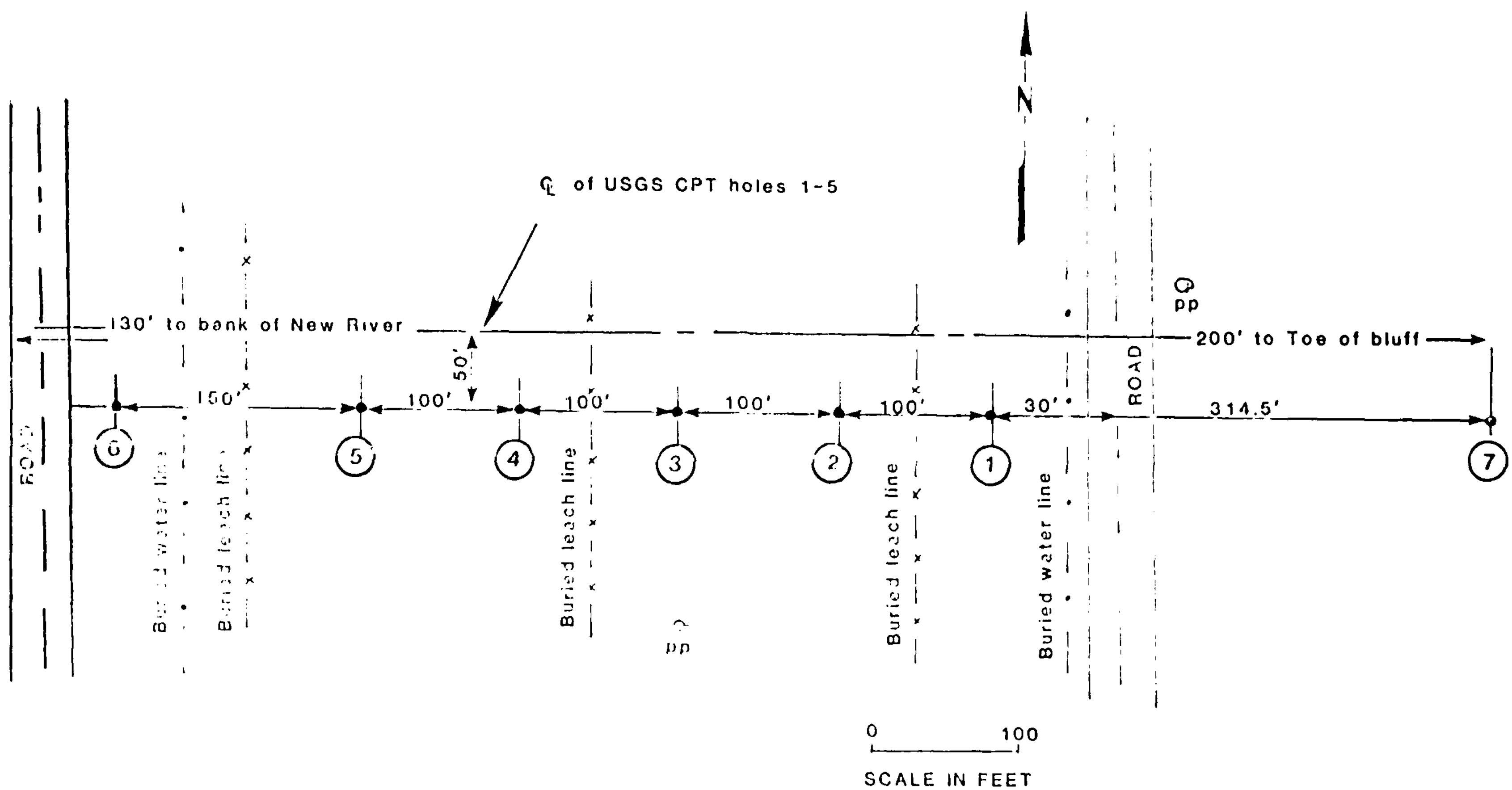


Figure 12. Site layout of PQS probe sounding at River Park

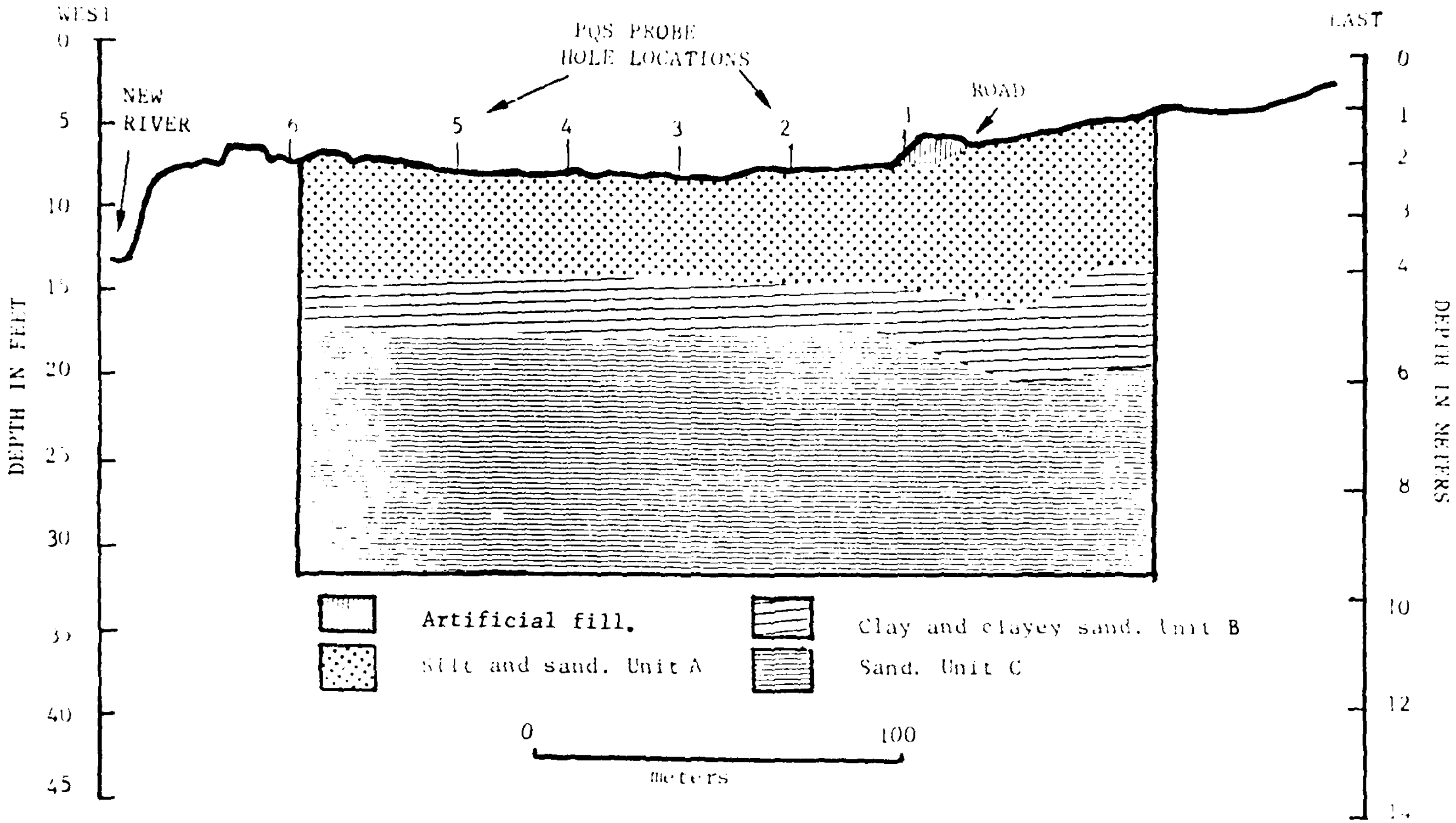


Figure 13. Floodplain. Unit A represents overbank floodplain deposition; Unit B represents flood basin deposition; and Unit C represents channel deposition, probably a pointbar (after Youd and Bennett, 1981)

UNITS	$\bar{N}$	$\bar{q}_c$	$\bar{R}_f$	$\bar{D}_r$	approximate depth, in meters
Fill*	-	18	2.13	-	variable
A- clayey silt to silty sand	3	24	2.57	54%	2.0
B- clay to clayey silt	3	9	3.40	-	3.0
C top** fine to medium sand	7	69	2.28	80%	
C bottom** fine to medium sand	23	138	2.51	102%	
C all** fine to medium sand	21	117	2.41	99%	11.0

$\bar{N}$ = Average N, in blows per foot
$\bar{q}$ = Average cone resistance, in kg/cm^2
$\bar{f}_r$= Average ratio, in per cent
$\bar{D}_r$= Average relative density, in per cent

* Fill replaces units A and B in holes 6 and 9 on the slump
** The top 1-m of unit C is medium dense sand; whereas the bottom of the unit is dense sand

Figure 14. Generalized characteristics of sediment at River Park site. Data include average blows per foot (N), point resistance/friction ratio, and relative density (D_r) (after Bennett et al., 1979)

distinguishable. The pore pressure ratios clearly define unit B, but it is not possible to differentiate between unit A and unit C.

38. After all of the field tests were completed, a series of lab tests were conducted to determine the cause and the effect of the negative pore pressures seen in the partially saturated soils. The tests were qualitative and consisted of simply inserting a properly deaired probe into a container of partially saturated clay and observing the response. Invariably, a negative pore pressure resulted (probably reflecting expectable soil suction), which resulted in a loss of saturation of the probe. Separate tests were conducted with water, silicon oil, and glycerin saturating the probe, and with all three fluids negative pore pressures and saturation loss occurred. When the probe was saturated with water, prolonged exposure of the porous tip element to air had the same effect.

39. It should also be pointed out that negative pore pressures frequently occur when a new rod is added during testing and that can add a variability in the record that is not ordinarily found in the ground. The effect is easily accounted for by an experienced technician as the pore pressure response is rapidly reestablished. The mechanism that causes the negative pore pressures appears to be rebound since the effects of halting penetration to add a rod can be seen in both the q readings and the f_s readings. It is a relatively simple matter for the digitizer to interpolate the response to eliminate the effect in the record. Unfortunately, the effect of the negative pore pressure on the saturation of the tip is unknown, but it does not appear that the effects of negative pore pressures at rod penetration halts are as severe as those in partially saturated soils. The records presented in this report have all been screened to remove the effects of penetration interruptions.

Conclusions

40. Experience with the PQS probe demonstrates that one cannot discriminate between loose and dense sands on the basis of dilative versus contractive pore pressure response probably because the state of stress in the soil at the probe tip is such that even dense sands can exhibit contractive behavior during the probe advance. In each case studied herein where negative pore pressures were seen there was some other explanation, unrelated to dilative response, and

in cases where negative pore pressures might have been expected in dense sands, they were not seen.

41. The magnitude of excess pore pressures developed seems to depend more on the drainage characteristics of the soil than on anything else. The sands without fines produced very small excess pore pressures, while sands with fines produced slightly larger pore pressures and the clays very large pore pressures. The excess pore pressure ratio was useful in distinguishing between the clay layers and the sand layers. In the Imperial Valley data the liquefaction-resistant clays were very obvious from the pore pressure ratio plots. In general, the friction ratio and the pore pressure ratio were both able to identify them very well.

42. The penetration of partially saturated soil may or may not cause the loss of saturation in the probe. Loss of saturation can be prevented by predrilling the hole to the water table and protecting the tip, but this can be an expensive and time-consuming task. Certainly better data can be obtained by predrilling, but more data with less effort can be obtained by pushing from the surface.

PART III: THE IN-SITU EVALUATION OF LIQUEFACTION POTENTIAL USING THE PQS PROBE

43. In Part II, it was concluded that the PQS probe reads the sum of the in-situ hydrostatic pore pressure and the dynamic pore pressure induced by penetration. Using this information, along with the other data provided by the PQS probe, it is possible to estimate the liquefaction potential of in-situ cohesioness material. This section of the report describes how this is done.

44. There are many complex techniques available for the dynamic evaluation of soil deposits. A number use expensive finite element methods which involve extensive field and lab testing; however, one of the simplest and most widely accepted techniques is the simplified procedure using the SPT developed by Professor H. B. Seed and his associates (Seed and Idriss, 1981). This technique is based on observations of the performance of cohesioness deposits in numerous earthquakes.

45. Figure 15 represents a comprehensive collection of site conditions at various locations where some evidence of liquefaction or no liquefaction has been observed during earthquakes. This collection of data has been used as a basis for determining relationships between field values of cyclic stress ratios, τ_h/σ'_v , and normalized blow counts, N_1 (Seed and Idriss, 1981). The cyclic stress ratio is the ratio of the induced shear stress on the horizontal plane, τ_h , and the vertical effective stress, σ'_v . The curves shown represent the dividing line between liquefaction and no liquefaction for various magnitudes of earthquakes.

46. The average cyclic stress ratio at any depth below the ground surface can be computed from

$$\frac{\tau_h}{\sigma'_v} \approx 0.65 \frac{\sigma_v}{\sigma'_v} \frac{a_{max}}{g} r_d$$

In this equation σ_v is the total vertical stress, a_{max} is the maximum horizontal acceleration, g is the acceleration of gravity, and r_d is a stress-reduction factor. The development of this equation is discussed in Seed and Idriss (1982). Figure 15 may be entered with the cyclic stress ratio induced by the earthquake and the normalized blow count N_1 to evaluate on an empirical basis the liquefaction potential of a sand deposit. Points falling above the curve corresponding to the magnitude of the earthquake indicate a high probability of liquefaction, while points falling below the line indicate

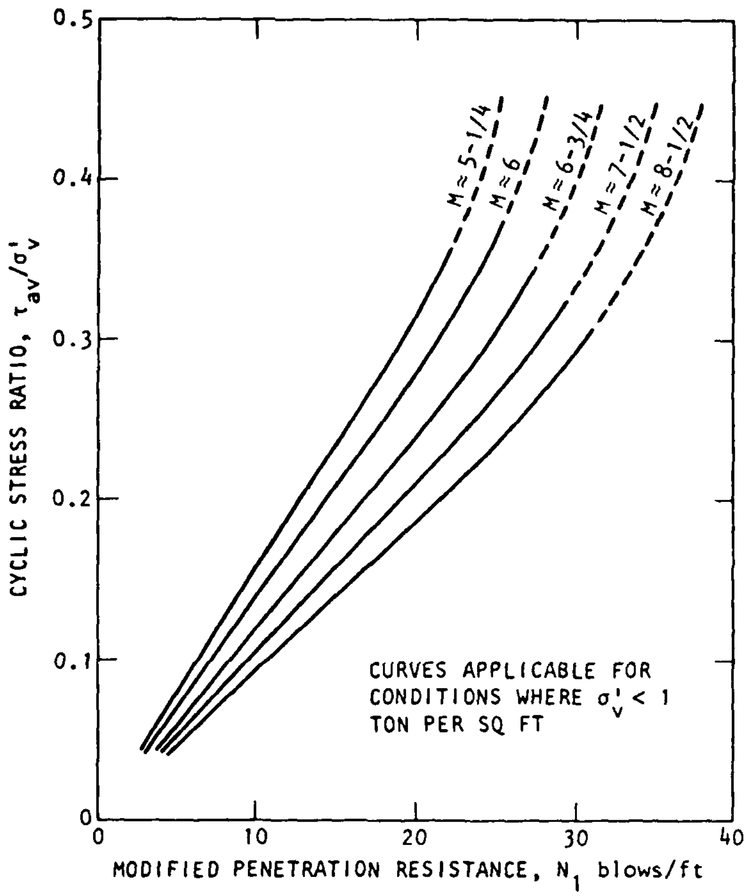


Figure 15. Chart for evaluation of liquefaction potential for sand for different magnitude earthquakes (after Seed and Idriss, 1982)

that liquefaction is unlikely. This procedure provides the basis for a method of estimating the liquefaction potential of cohesionless deposits by use of the PQS probe or a similar device.

47. To use the simplified procedure it is necessary to know the following values:

- a. The blow count from a standard penetration test, N .
- b. The total vertical stress σ_v .
- c. The effective vertical stress σ'_v .
- d. The maximum ground acceleration a_{max} .
- e. The magnitude of the earthquake M .

Also, the soil type must be considered. The curves shown in Figure 15 were developed for clean sands. For silts or silty sands, the same curves may be used after increasing the measured values of N_1 by 7.5. For other soil types the method is not applicable (Seed and Idriss, 1981).

48. The magnitude of the earthquake and the maximum ground acceleration are established by a geological/seismological study. The following information must be obtained from the field investigation:

- a. The N value or its equivalent.
- b. The total overburden stress.
- c. The effective vertical overburden stress.
- d. The soil type.

49. The PQS probe determines penetration resistance in a continuous form and plots it out as q . As yet, no direct site-to-site correlations of liquefaction or no liquefaction using q directly have been made as has been the case with SPT. Thus it is necessary to convert the q value determined by CPT to an equivalent SPT N value and then use the simplified method developed by Seed and Idriss (1981). An example of this procedure is given in Douglas and Olsen (1981). For the purpose of this report it shall be assumed that the SPT N value can be approximated by dividing q (when expressed in tsf) by 4.5 (Seed and Idriss, 1982). This "rule of thumb" has been used for a number of years and is simply the midrange of a number of values with significant scatter. The value of assuming this relationship is that with it preliminary liquefaction analysis can be made on site using only the PQS probe.

50. Once the equivalent N values have been determined, the N_1 values can be computed from the relationship

$$N_1 = N \times C_n$$

where C_n is the overburden correction factor and can be determined by relationships such as the one mentioned in Part II. To calculate the total and effective vertical stresses involves the determination of the density of the material with depth and the location of the water table. The density of the material can be estimated within reasonable limits by the use of q, and the total and effective vertical stresses by locating the water table. Thus from the surface to the water table total weights are used to determine the total stresses and below the water table effective stresses are used.

51. The PQS probe is a tool especially well suited for determining the location of the water table, which is frequently difficult to do in the field. Referring to the field data presented in Appendix A, B, and C, it can be seen that pore pressures do not build up above the water table. And as seen in some of the data, negative pore pressures frequently occur. In addition, in coarse-grained material the pore pressures rapidly decay to hydrostatic pressure if penetration is halted. Thus in any one sounding numerous opportunities are available to estimate the hydrostatic pore pressure existing in situ. Thus, during penetration the PQS probe operator can establish where the hydrostatic ground water table is. From the densities and the water table, the total and effective stresses can be calculated.

52. It has been adequately demonstrated for many years that the friction ratio is a reasonably reliable predictor of soil type for normally consolidated soils. Generally, a low friction ratio indicates the material is a sand, and a high-friction ratio indicates that it is a clay. This method is far from infallible, but is reliable enough to be taken into consideration. In Part II, it was shown that sands did not build up any significant pore pressures. In fact, the pore pressure ratio identified the clay and sand layers very well once the penetration proceeded below the water table. Thus it appears that a combination of high friction ratio (greater than ≈4) and a high pore pressure ratio (greater than ≈10) will very effectively point out any nonliquefiable soils. In preliminary liquefaction evaluations this is what makes the PQS (or any other probe which might measure the same parameters) unique as the remainder of the preliminary investigation is within the capability of any Dutch cone type device.

53. The point has now been reached that for any q as a function of depth determined from PQS or Dutch cone an N_1 value can be determined. Using the formula

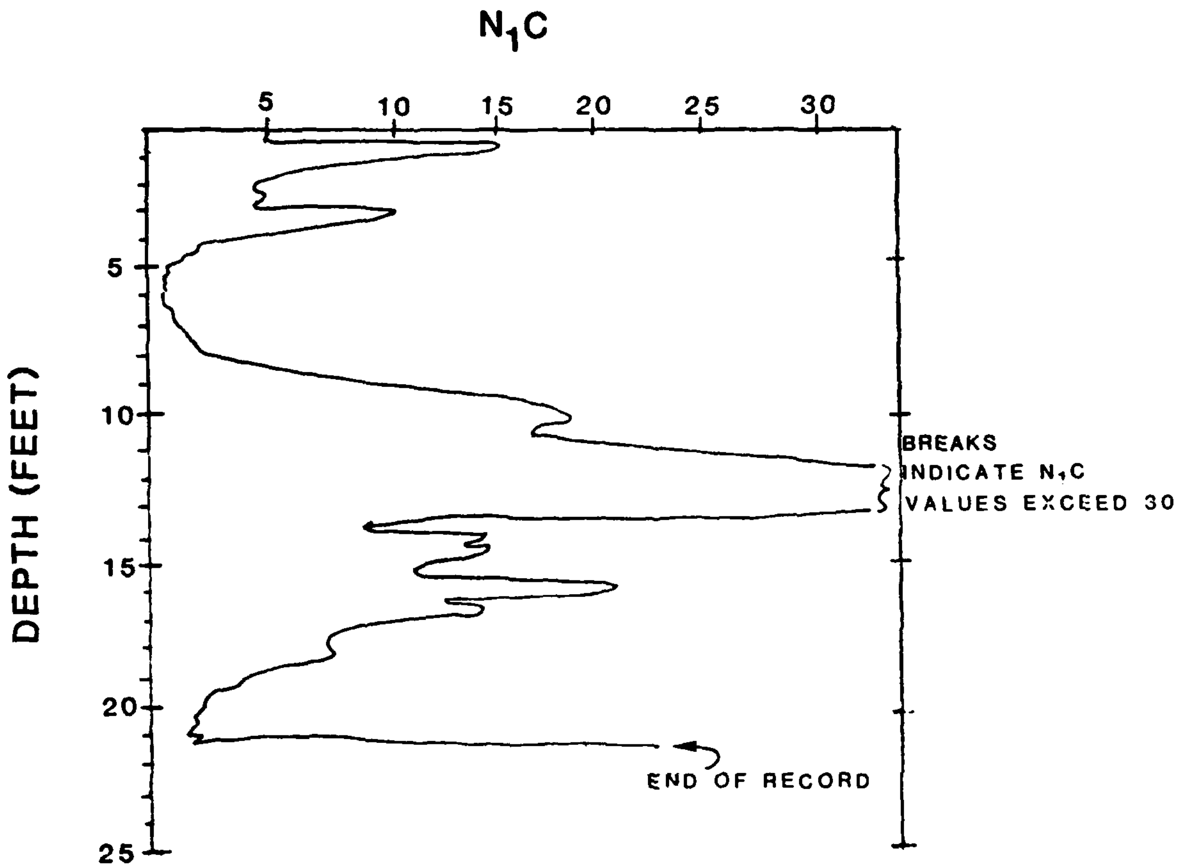
$$\frac{\tau_n}{\sigma'_v} \simeq 0.65 \frac{\sigma_v}{\sigma'_n} \frac{a_{max}}{g} r_d$$

the cyclic shear stress ratio resulting from the specified ground motion can also be predicted (Seed and Idriss, 1981). Using the combination of N_1 and cyclic shear stress ratio, τ_n/σ'_v, the position on the liquefaction chart can be determined. Having had the magnitude of the earthquake specified by the seismologist, it can be determined whether that point lies to the left or right of the liquefaction line. If the line lies to the left of the point, then liquefaction is likely. If it lies to the right of the point, then liquefaction is unlikely and normally this is how the liquefaction analysis is done in the simplified procedure. However, literally there are hundreds of q's measured by the PQS probe or other cone penetrometers. In fact, one advantage of the PQS probe is that it gathers a lot of data in a continuous stream during a push. Thus a point-by-point plot on the liquefaction chart is not the best way to present the data. With each q value that is in digital form, there is a computed N_1 value, and N_1's are just as easily plotted with depths and q. An example of this is shown in Figure 16. In Figure 16, the q values converted to equivalent N_1 values are plotted as N_1C values to designate them as having been derived from a cone penetrometer. These values are determined as follows

$$N_1C = \frac{q}{4.5} C_N$$

54. Because the cyclic shear stress ratio induced by the earthquake at the point of the q reading for a maximum acceleration is known for any particular magnitude of earthquake, the N_1 value required to resist liquefaction can be determined. This value can be plotted for each depth on the same plot as the in-situ N_1C, and a chart of the form of Figure 17 can be derived. This chart shows clearly the N_1C values existing in situ and N_1 values that are required to resist liquefaction at the various elevations.

55. Figure 18 is a chart of the N_1 values with the friction ratio, the excess pore pressure ratio, and the absolute value of the excess pore pressure generated shown all together. In areas of very high N_1 values obviously there is no liquefaction problem. In areas where N_1 values below those



$$N_1C = \frac{q}{4.5} N_1$$

Figure 16. N_1C values determined from cone penetration resistance values

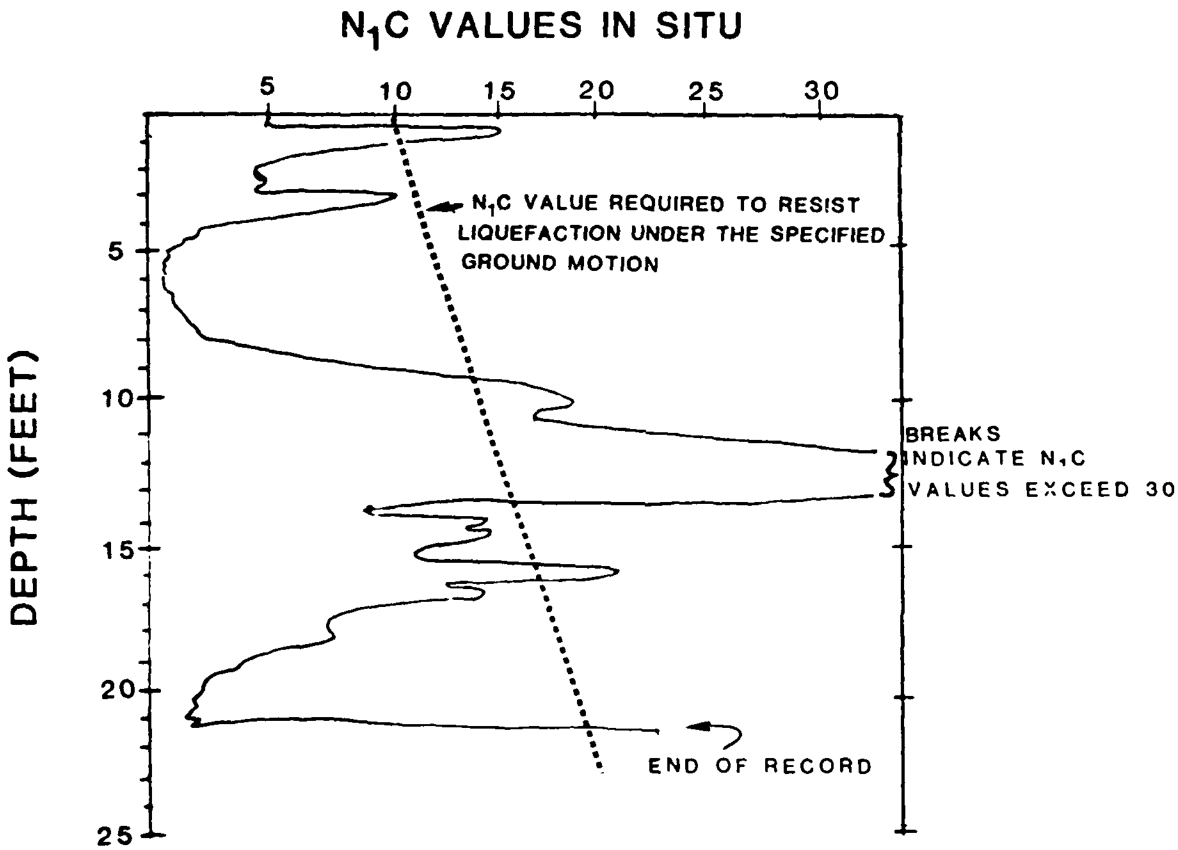


Figure 17. N_1 values required to resist liquefaction plotted on the N_1C depth sounding

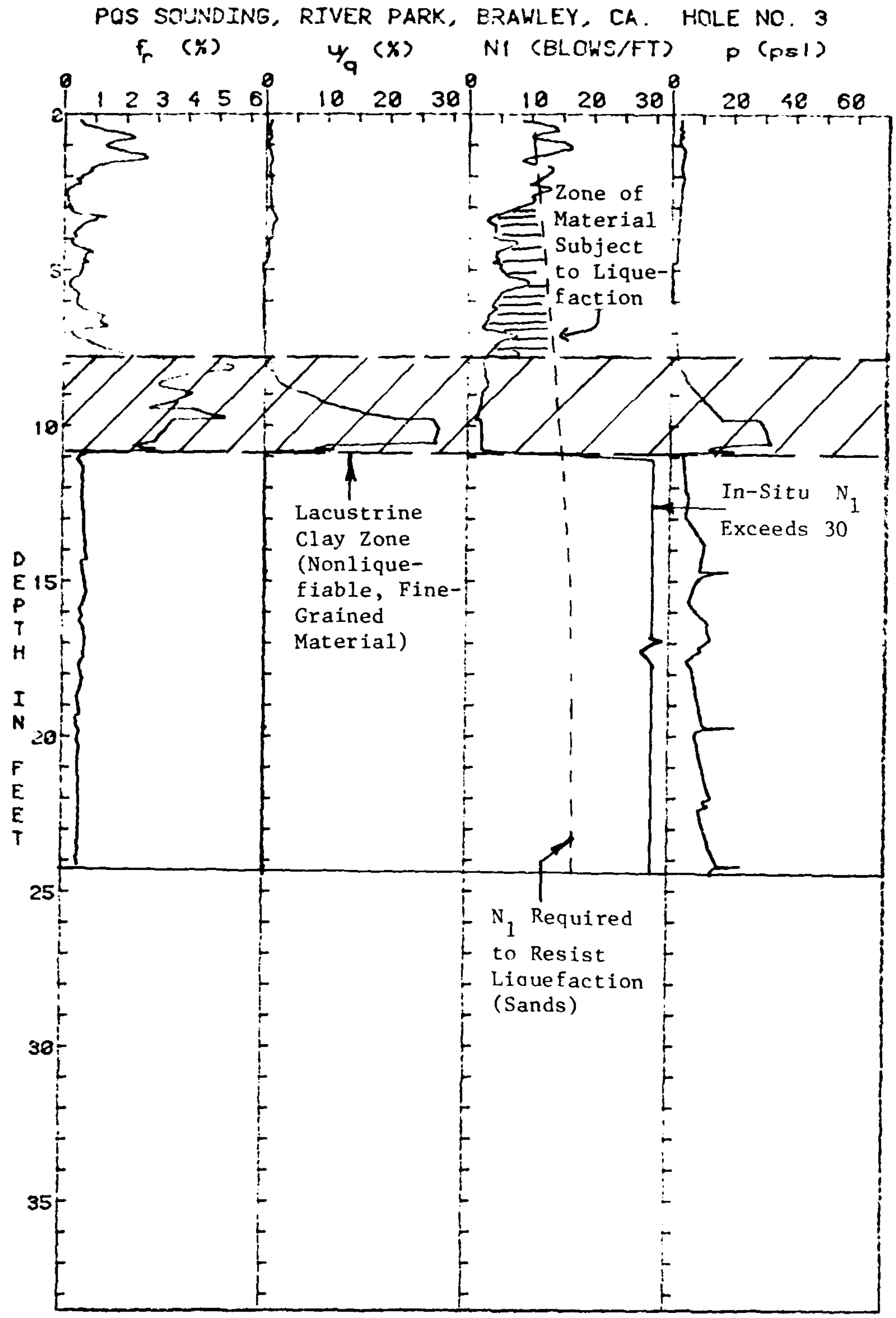


Figure 18. Liquefaction analysis using the PQS probe

necessary to resist liquefaction are present in situ and there are low friction and low excess pore pressure ratios, then a liquefaction problem exists. In those sections where there are low N_1 values and high friction ratios and high positive excess pore pressure ratios, then that material is probably a nonliquefiable material. Of course, some confirmatory borings would be required before a final determination could be made. In any case, the PQS method would not supplant normal test procedures but would simply tell the engineer where to perform general sample borings and SPT tests and what results to expect. This should reduce the number required for adequate site coverage.

56. Using this approach, a rapid site evaluation can be made. The digitized information from the PQS probe sounding is input into a computer code which provides a plot similar to that of Figure 18. Currently at WES, this processing has to be done in the office using digitizing equipment. However, equipment is available which can digitize and process the data in the field.

57. The PQS method presented herein consists of comparing a site of interest with other sites that have and have not liquefied during earthquakes. The comparison is purely empirical, but it is effective and simple. The major disadvantage of the PQS method is that the comparison is not made directly but through use of the SPT, which introduces all of the uncertainties involved with the correlation of the cone with the SPT and with the SPT itself. A much better solution would be to use a normalized q directly instead of converting to N_1 values, but the large amount of data that exists today exists as N_1 values and conversion to a normalized q would require extensive retesting. This may be the direction of the future, but for now the existing data base must suffice.

58. The PQS method is not a foolproof system. Both confirmation borings and the judgment of a qualified engineer are important ingredients in making the system work. For instance, the determination of the water table location, the decision as to when enough soundings have been made, and the decision where to make the next sounding require experience and a knowledge of earthquake engineering principles. Not the least of the problems is the fact that the PQS probe, supporting computer equipment, and the necessary software do not exist commercially as a system and step-by-step development has been required.

59. The advantages, however, are many and most of them involve ultimate savings in time and money. Generally, CPT can be done four times as fast as drilling and at about one-fourth the cost, and much more information is

obtained. However, other very important advantages exist in the reliability and repeatability of the PQS probe tests, which, in addition to the thoroughness with which a site can be investigated, make initial field testing with the PQS probe a very attractive alternative to random SPT work.

PART IV: SITE EVALUATIONS

60. The PQS soundings for the three sites discussed in Part II have been processed by the technique presented in Part III. The records for the Montz site are in Appendix D. The Heber Road site and the River Park site records are in Appendices E and F, respectively. Each of the three sites is discussed separately below.

Montz, La., Site

61. The primary reason for testing the PQS probe at the Montz, La., site was that an extensive field investigation was being conducted using state-of-the-art in-situ testing techniques. In Part II, it was shown that the follow-on laboratory testing allowed for classification of the soil and the establishment of the volume change characteristics. In Figure 4, the results of the SPT tests are shown; these data are useful in verifying the assumed q_c versus N correlations, and in Appendix D, the derived N_1 values determined from the SPT are plotted on the same graph as the N_1C values determined by the PQS probe. The agreement is not exact, but it is certainly reasonable because the two soundings were 20 ft apart and located only approximately in the same vicinity as SPT 2 ($\pm$20 ft). The correlation seems acceptable since the alternative is a site-specific correlation, such as the one discussed in Douglas and Olsen (1981). That refinement does not seem justified when using the PQS probe as a rapid means of determining liquefaction potential in situ since it requires that SPT tests be performed. The proposed method uses the PQS probe alone as a tool for rapid preliminary surveys.

Heber Road Site

62. The Heber Road site was selected for field testing the PQS probe because the site liquefied under earthquake loading, but the evaluation of the site using the simplified procedure presents a peculiar problem. When the simplified procedure was introduced, there were no near-field earthquake records available to accurately determine near-field accelerations. Thus accelerations were estimated based on evidence such as local intensity. The Heber Road site is located approximately 2 km from the Imperial fault and near

the epicenter of the 15 October 1979 earthquake. Youd and Bennett (1981) estimated the peak acceleration at the Heber Road site to be 0.8 g's. This estimate is based on a recorded acceleration of 0.81 g's measured at Bond's Corner, 3.8 miles (6 km) southeast of the Heber Road site. The accelerations measured at Bond's Corner were far above those estimated for other sites previously used to establish the data base for the simplified procedure, and the data point generated by plotting the Heber Road information is far out of the data range. Figure 19 shows the general relation of the Heber Road data point to the rest of the data points. Keeping in mind that the simplified procedure is a site-to-site comparison, it can be seen that the procedure is not appropriate in this case. There are no other data to compare it with. Heber Road is simply the first site in a new data base where near-field accelerations have been measured relatively close to the site under consideration. In this case, the N_1C values are presented in Appendix E, but the N_1 values to resist the liquefaction are not.

River Park Site

63. The acceleration records for the River Park site and its relation to the epicenter of the earthquake indicate that it can be evaluated as a far-field site. Youd and Bennett (1981) chose an acceleration of 0.2 g's and that figure seems reasonable. Using this value and the magnitude of the earthquake, the N_1 value required to resist liquefaction can be determined as a function of depth as has been done for the plots in Appendix F.

64. Figure 18 shows how the data can be interpreted using the concepts discussed in Part III. The three geologic units described in Part II are shown in Figure 18. It can be seen that unit A does indeed appear to be susceptible to liquefaction. It is a material exhibiting a low friction ratio, weak pore pressure response, and N_1C values less than that required to resist liquefaction. Unit B, on the other hand, although it has N_1C values less than that required to resist liquefaction, has a strong pore pressure response and a high friction ratio, all indicative of a fine-grained soil. The site analysis previously presented in Part II shows that this is the case. Unit C demonstrates a low pore pressure response and friction ratio, but has a N_1C value well in excess of that required to resist liquefaction; thus the soil is nonliquefiable due to strength.

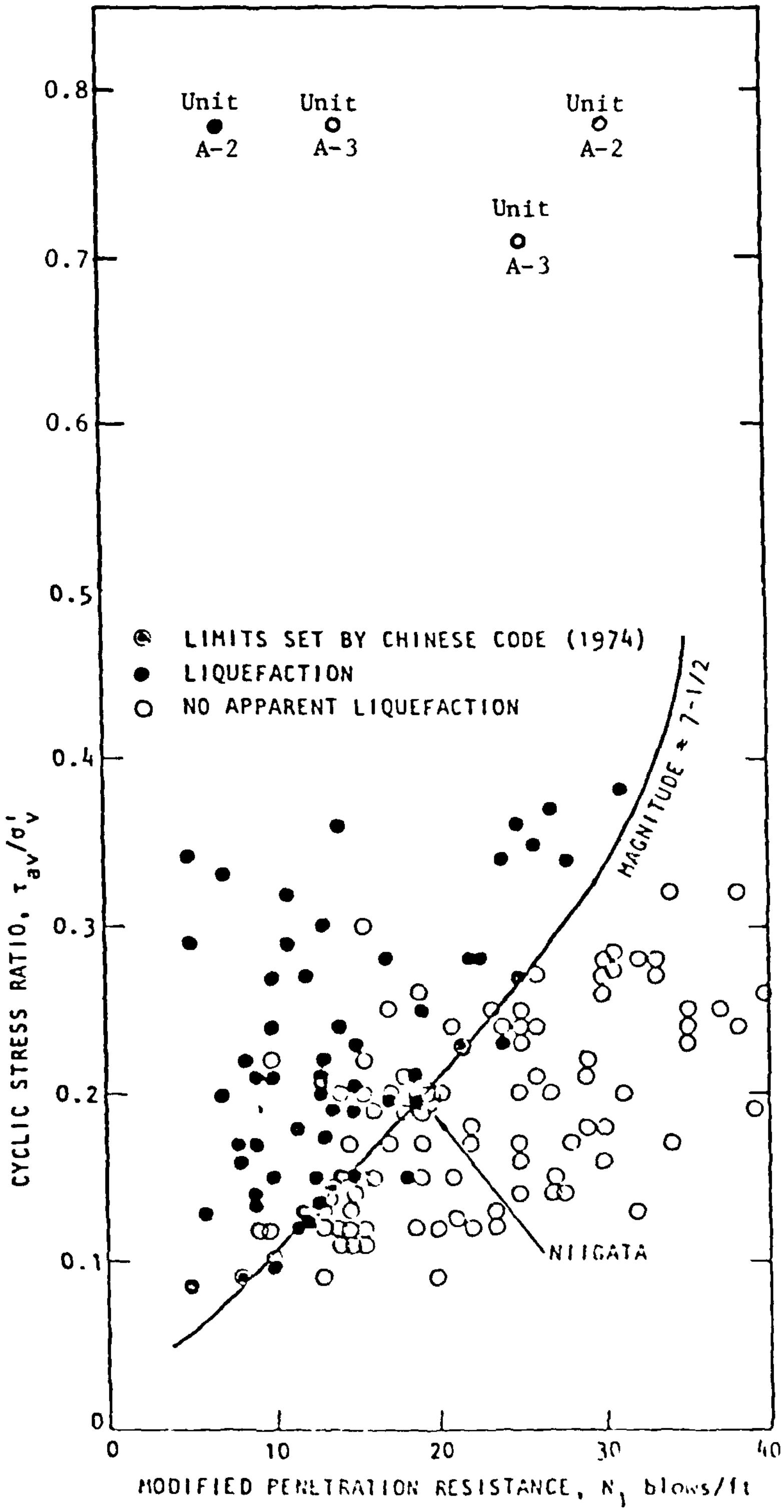


Figure 19. Correlation of liquefaction behavior of sands (D_{50} > 0.25) under level ground conditions and standard penetration resistance- (all data) effective overburden pressure less than 1 tsf (after Seed and Idriss, 1982)

65. An analysis similar to that in Figure 18 has been made for each of the PQS soundings in Appendix F, with similar results. The loose fill in unit A is susceptible to liquefaction; the lacustrine clay is not, nor is the point-bar deposit. This conclusion is reached using only the records generated by the PQS probe and the criteria previously described. The records indicate the potential problem areas very well and would assist a great deal in planning a detailed investigation program.

PART V: SUMMARY AND CONCLUSIONS

66. This report documents the field testing of a penetration device, the PQS probe, capable of simultaneously measuring penetration resistance, friction resistance, and pore pressure response. The probe is evaluated as a tool to measure liquefaction related soil characteristics in situ. Of special interest is the study of pore pressure response in order to determine whether it is diagnostic of contractive or dilative behavior of cohesionless soils and thus their liquefaction potential. In addition, a procedure using the penetration resistance, q, to assess liquefaction is presented.

67. Evaluation of the pore pressure data recorded during penetration led to the following conclusions:

- a. Positive pore pressures in situ are likely to occur in nonliquefiable sands even though they tend to dilate in shear. The PQS probe could not distinguish between liquefiable and nonliquefiable soils on the basis of positive or negative pore pressure response. The original hypothesis failed.
- b. Negative pore pressures were observed in two situations not related to contractive or dilative behavior, one during the penetration of partially saturated soils above the water table and the other during temporary halts to add additional push rods.
- c. The excess pore pressure ratio, u/q, and the friction ratio, f_s/q, behaved similarly in cohesive deposits and appear to be a reliable index to nonliquefiable material. However, since the PQS probe does not produce a sample for evaluation, the test is not conclusive.

68. A comparison of the PQS field test results with SPT data indicated that q is a reasonable measure of N, and it can be used to evaluate the liquefaction potential of a soil by using the simplified procedure. Three sites were evaluated using the PQS probe, and it was found that the procedure worked well when the simplified procedure was appropriate. Since the quantity q is used in the analysis, any cone penetrometer that produces that quantity can be used. The advantage of using the PQS probe is that it adds the capability of determining the elevation of the groundwater table and provides an indicator of cohesive soils.

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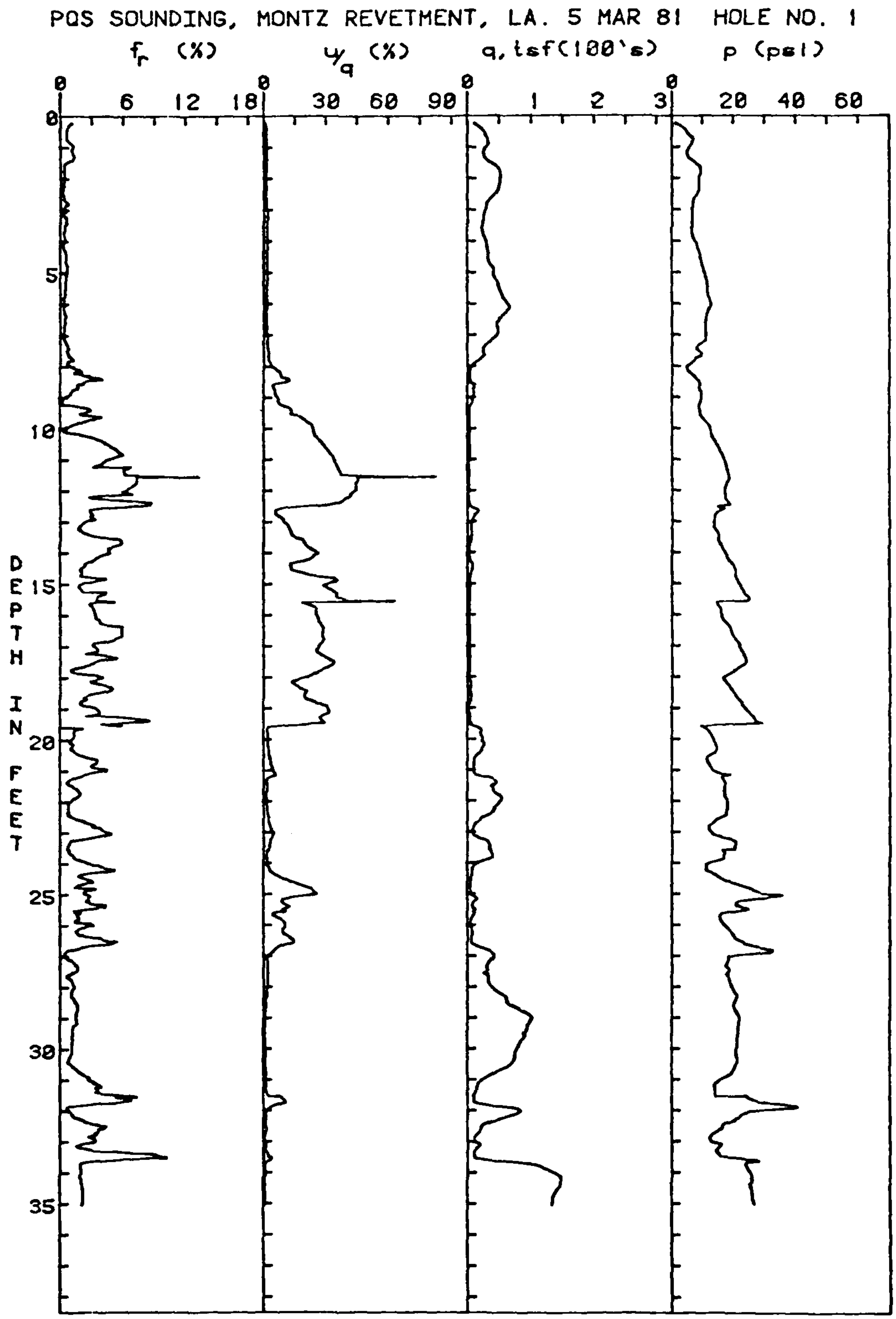
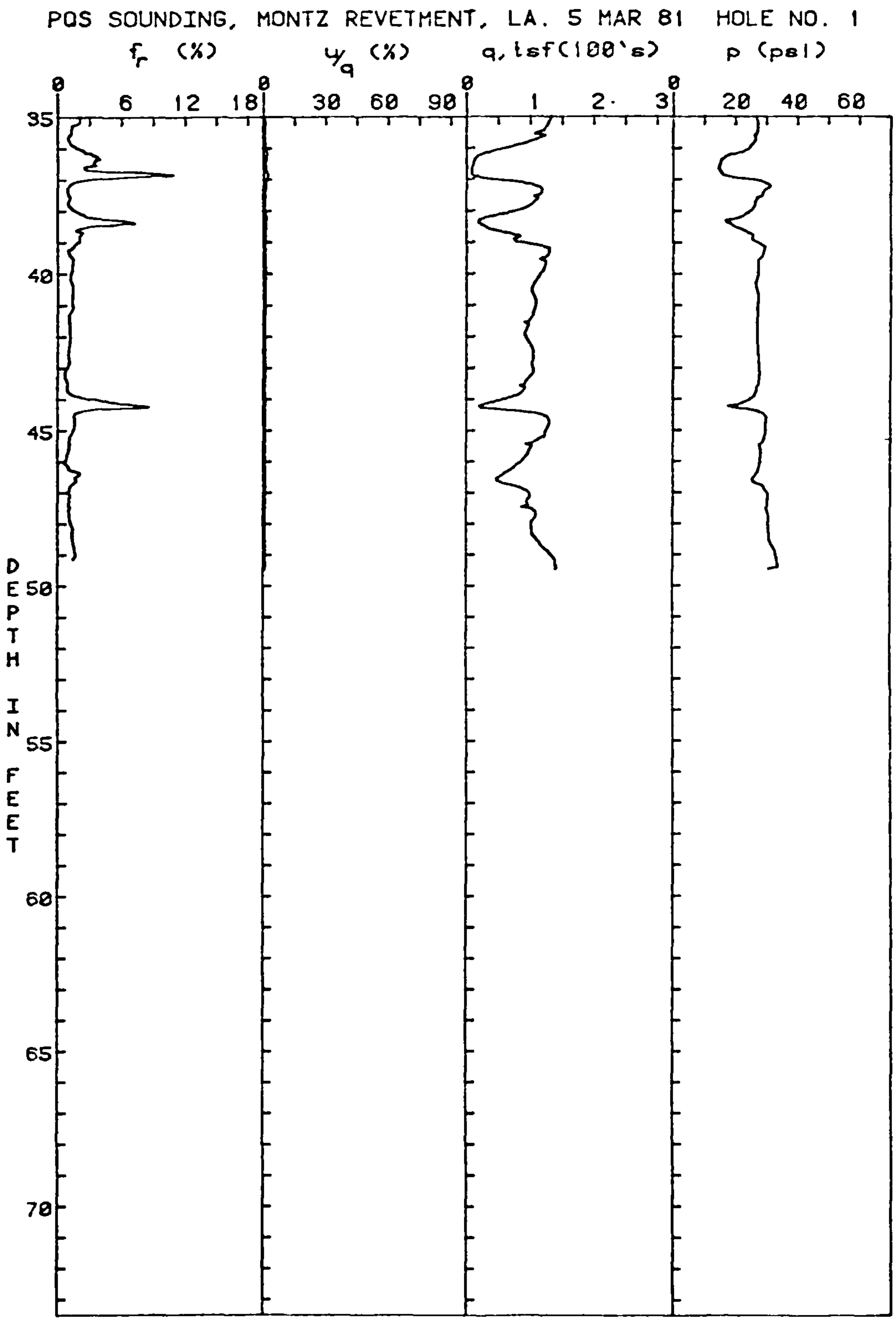
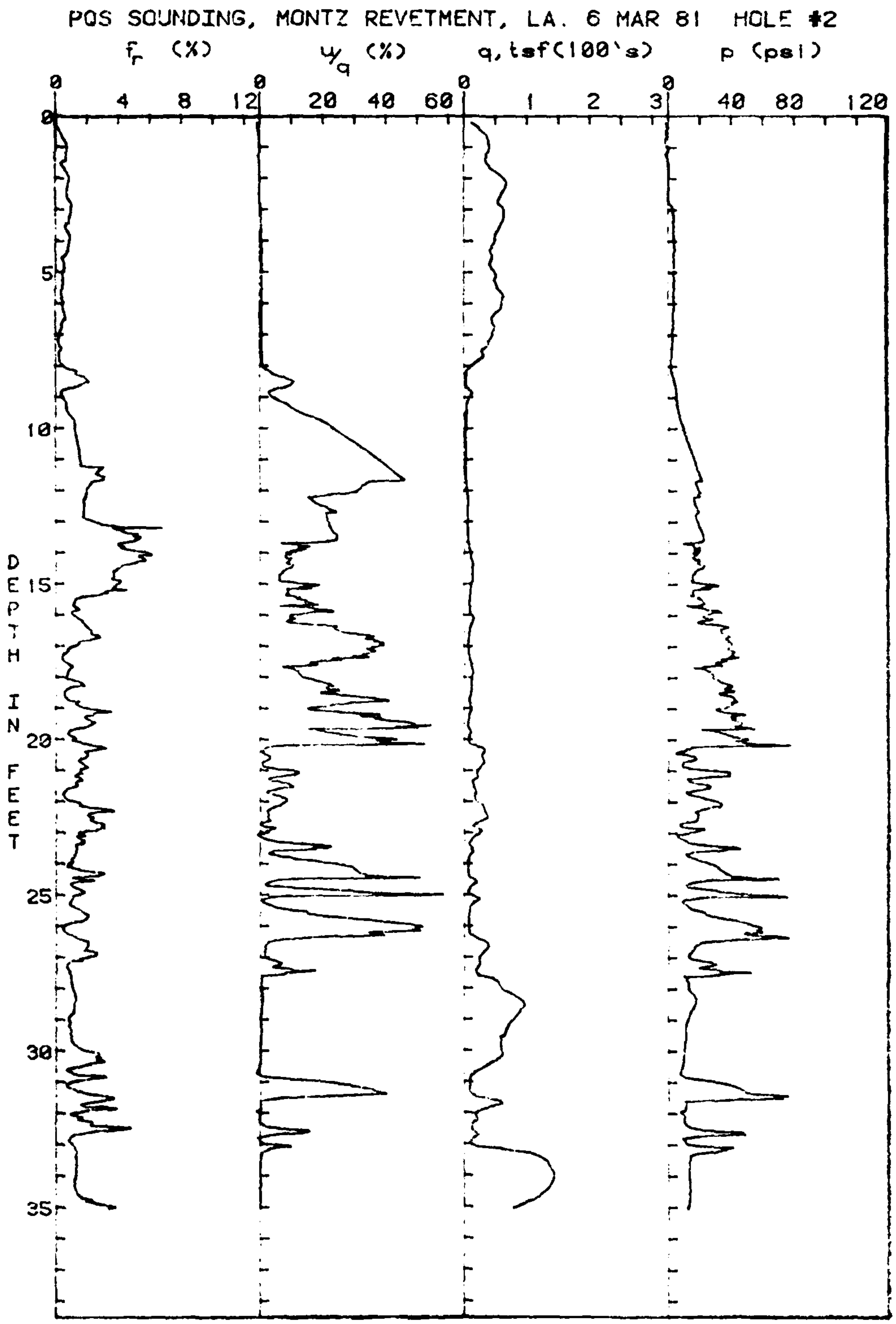
APPENDIX A

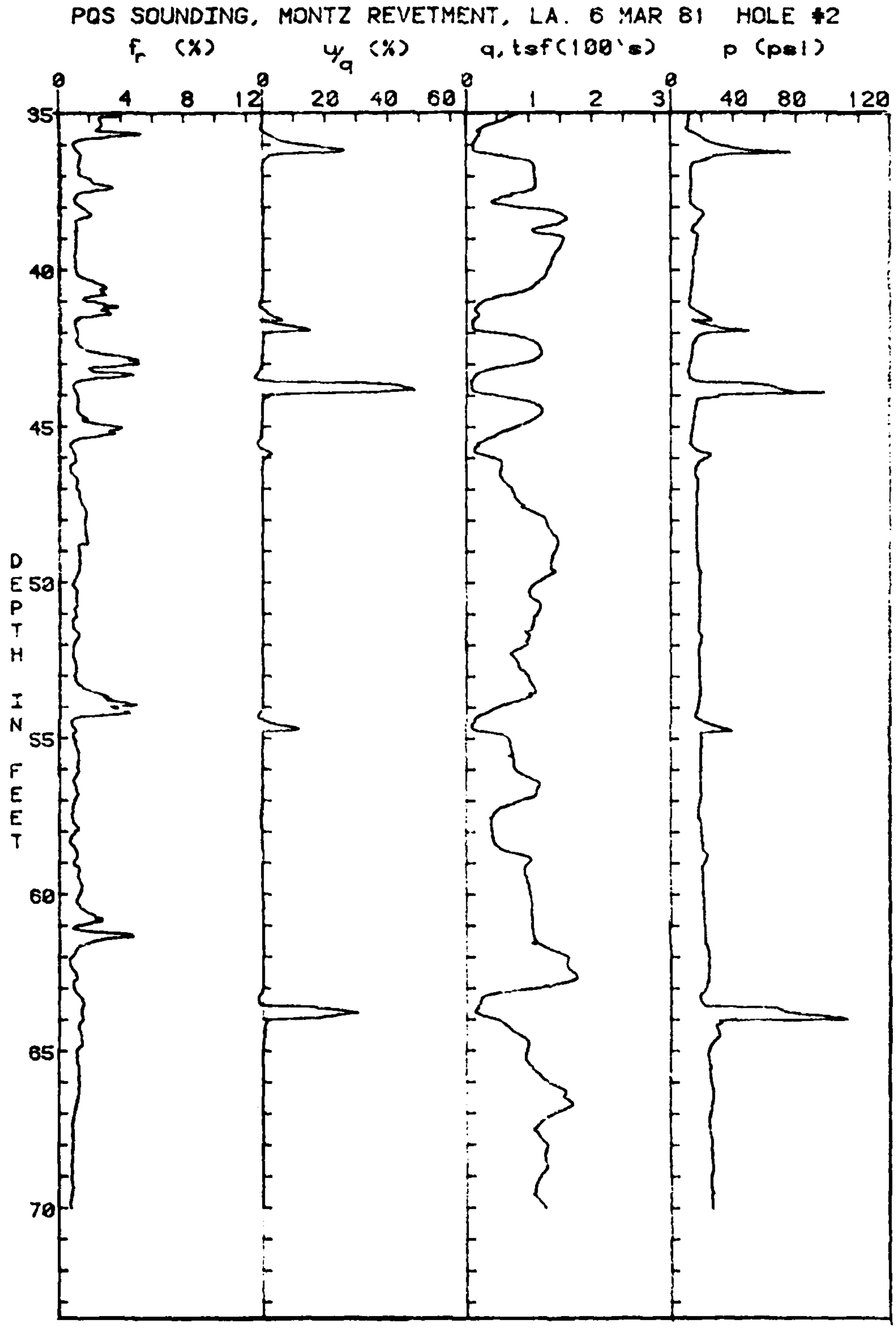
LOGS OF PQS HOLES AT MONTZ SITE


PQS SOUNDING, MONTZ REVETMENT, LA. 5 MAR 81 HOLE NO. 1
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 6 12 18
0 30 60 90
0 1 2 3
0 20 40 60
0 5 10 15 20 25 30 35
DEPTH IN FEET


PQS SOUNDING, MONTZ REVETMENT, LA. 5 MAR 81 HOLE NO. 1
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 6 12 18
0 30 60 90
0 1 2 3
0 20 40 60
35
40
45
50
55
60
65
70
DEPTH IN FEET



PQS SOUNDING, MONTZ REVETMENT, LA. 6 MAR 81 HOLE #2
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
DEPTH IN FEET

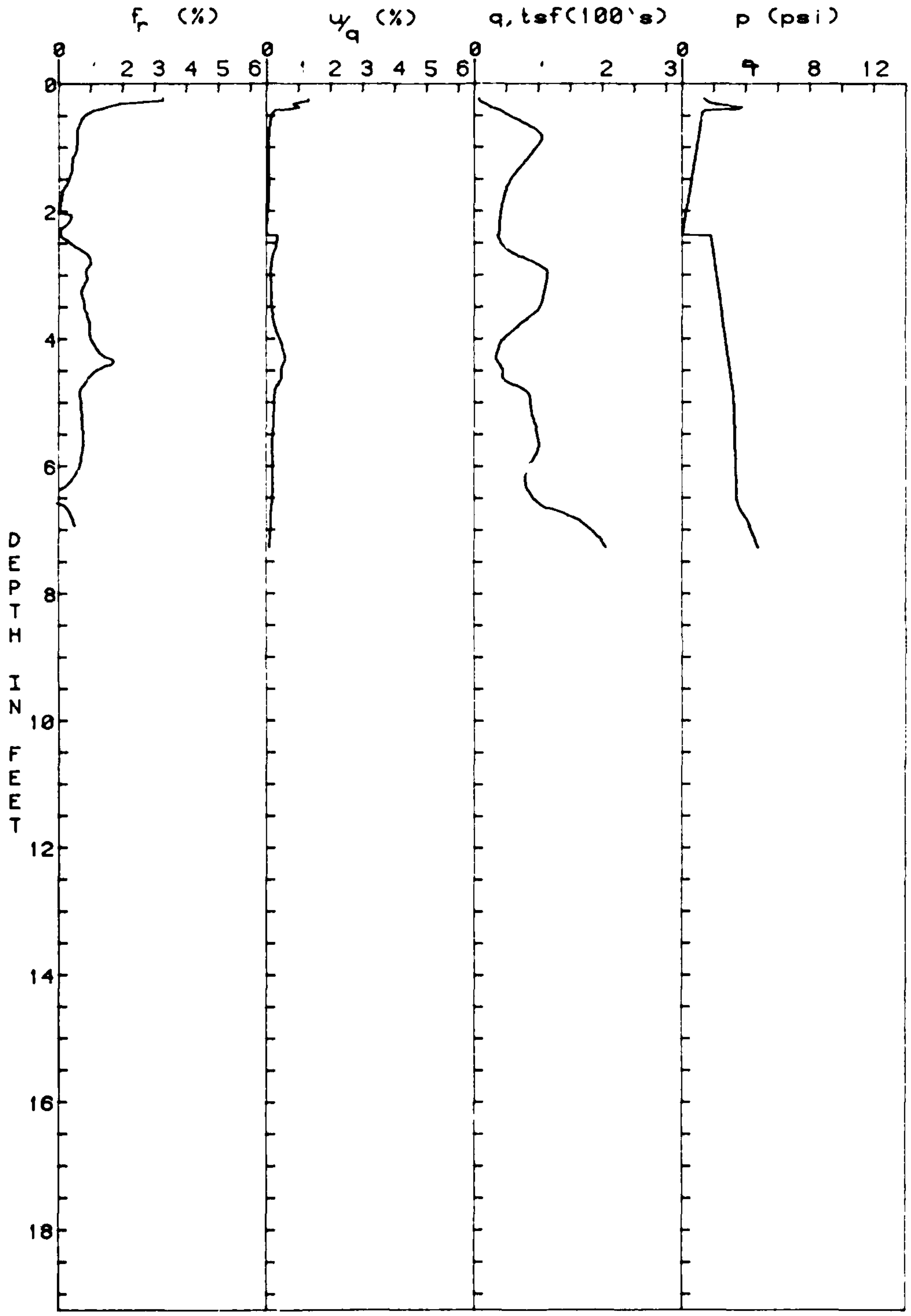


PQS SOUNDING, MONTZ REVETMENT, LA. 6 MAR 81 HOLE #2
fr (%)
u/q (%)
q, tsf(100's)
p (psi)
0 4 8 12
0 20 40 60
0 1 2 3
0 40 80 120
35
40
45
50
55
60
65
70
DEPTH IN FEET

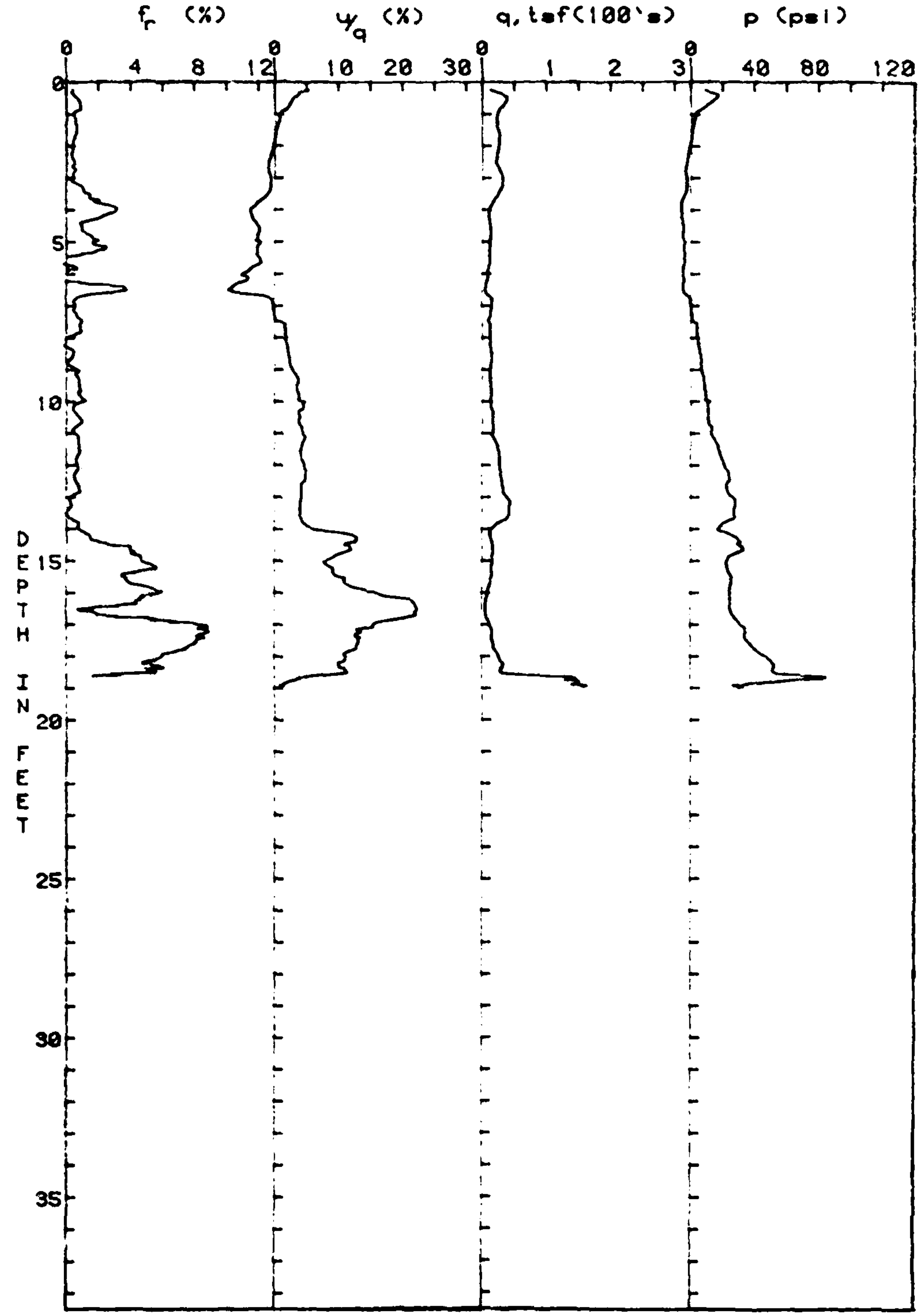
APPENDIX B

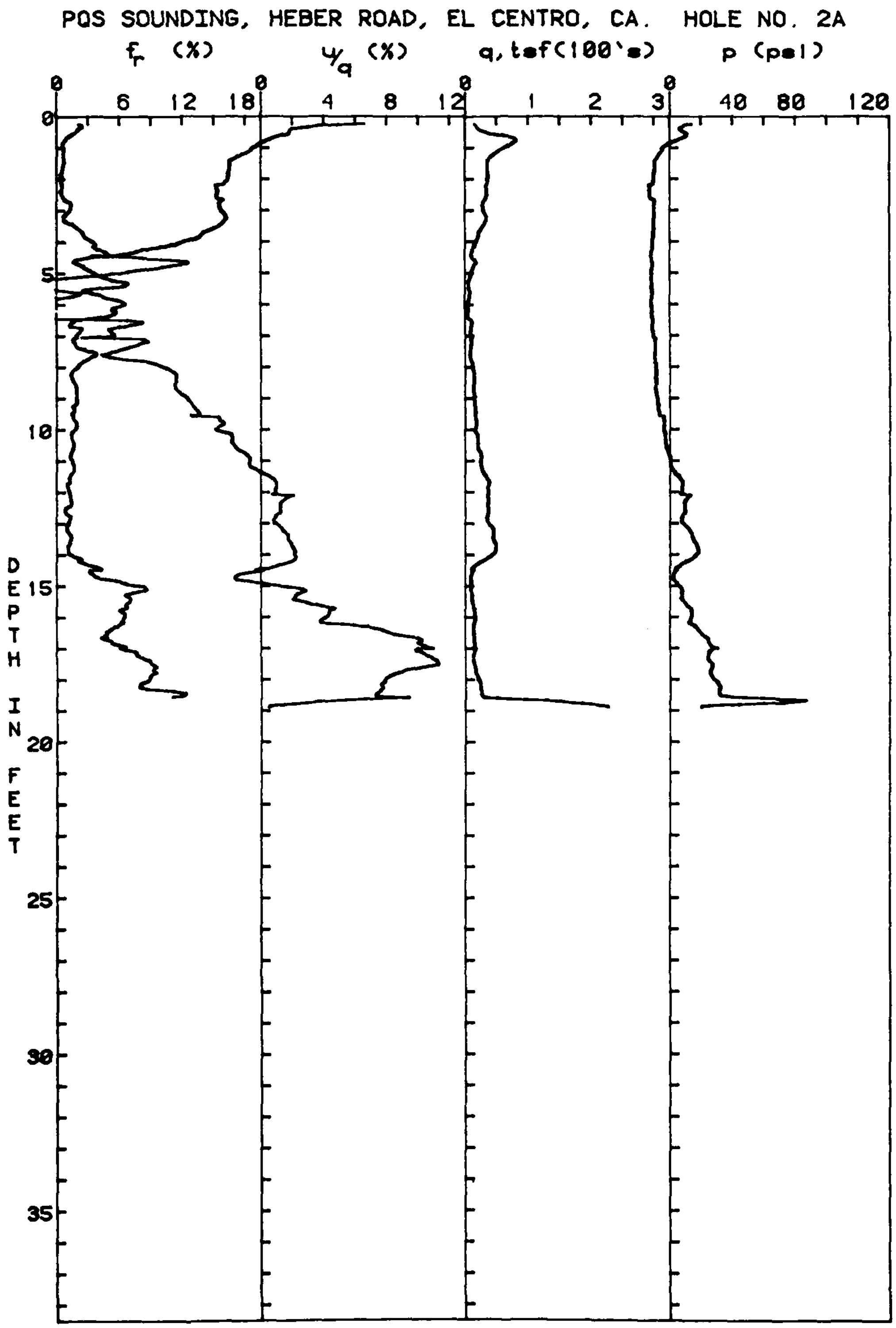
PENETRATION LOGS OF PQS HOLES AT HEBER ROAD SITE

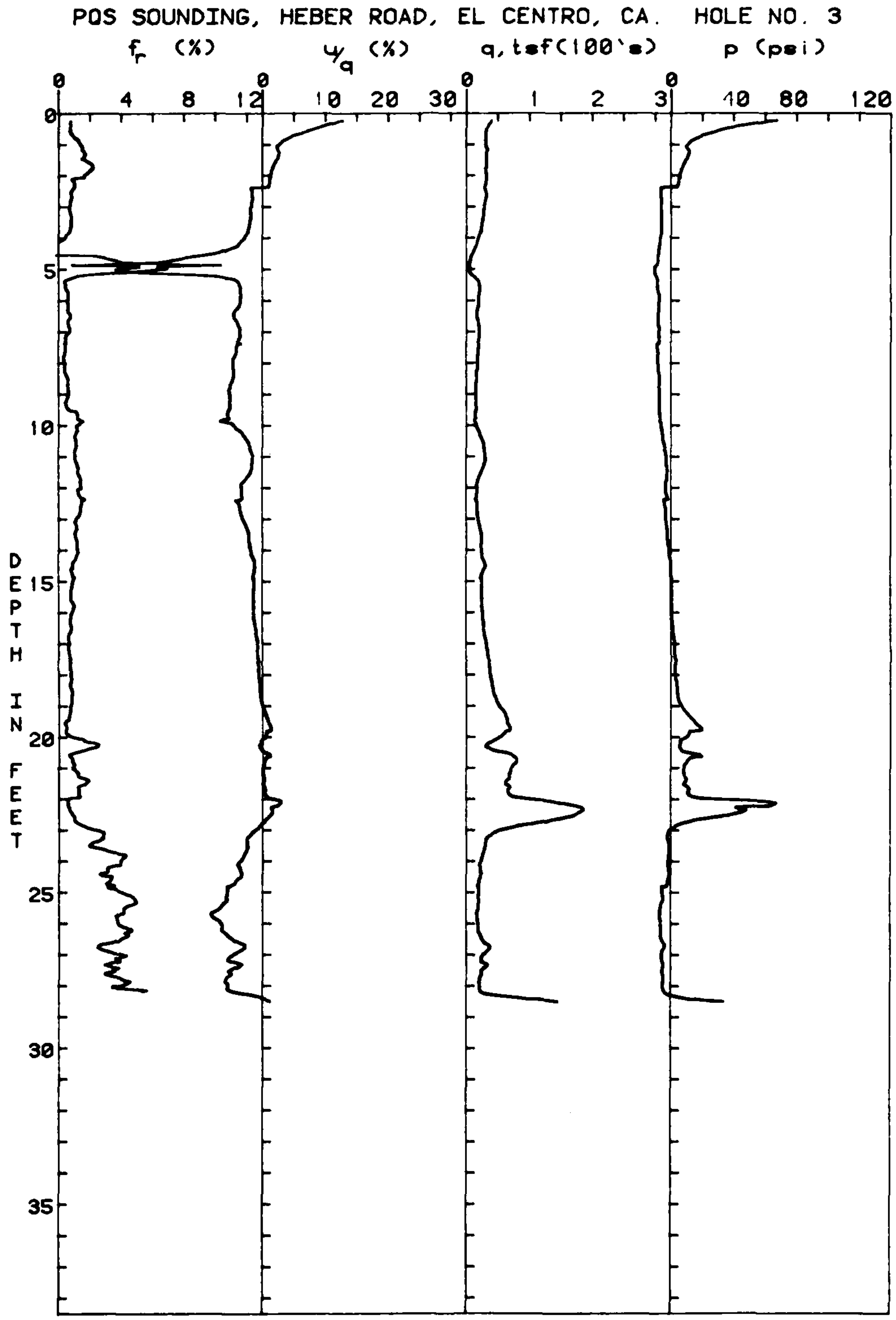
PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 1

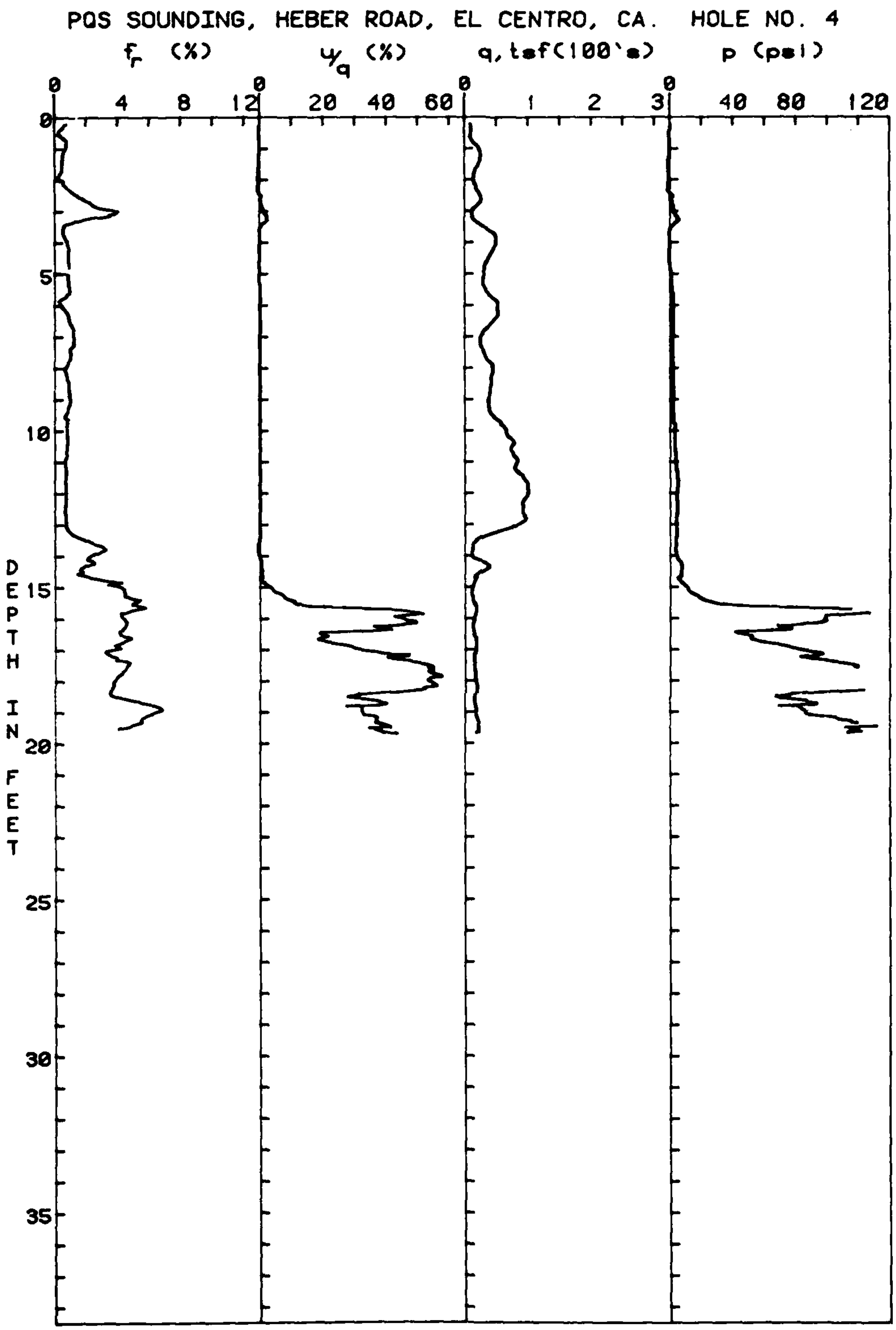


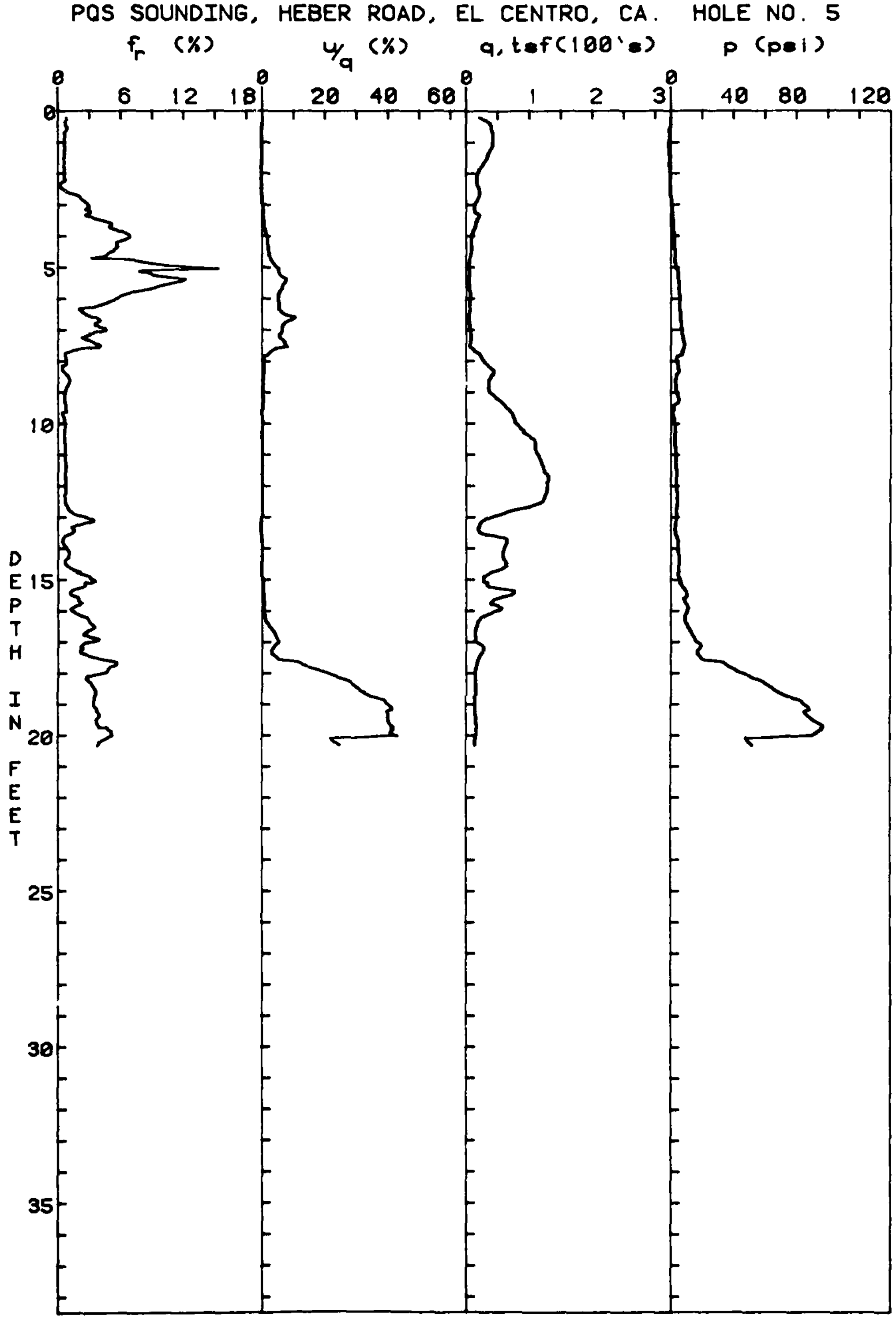
PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 2

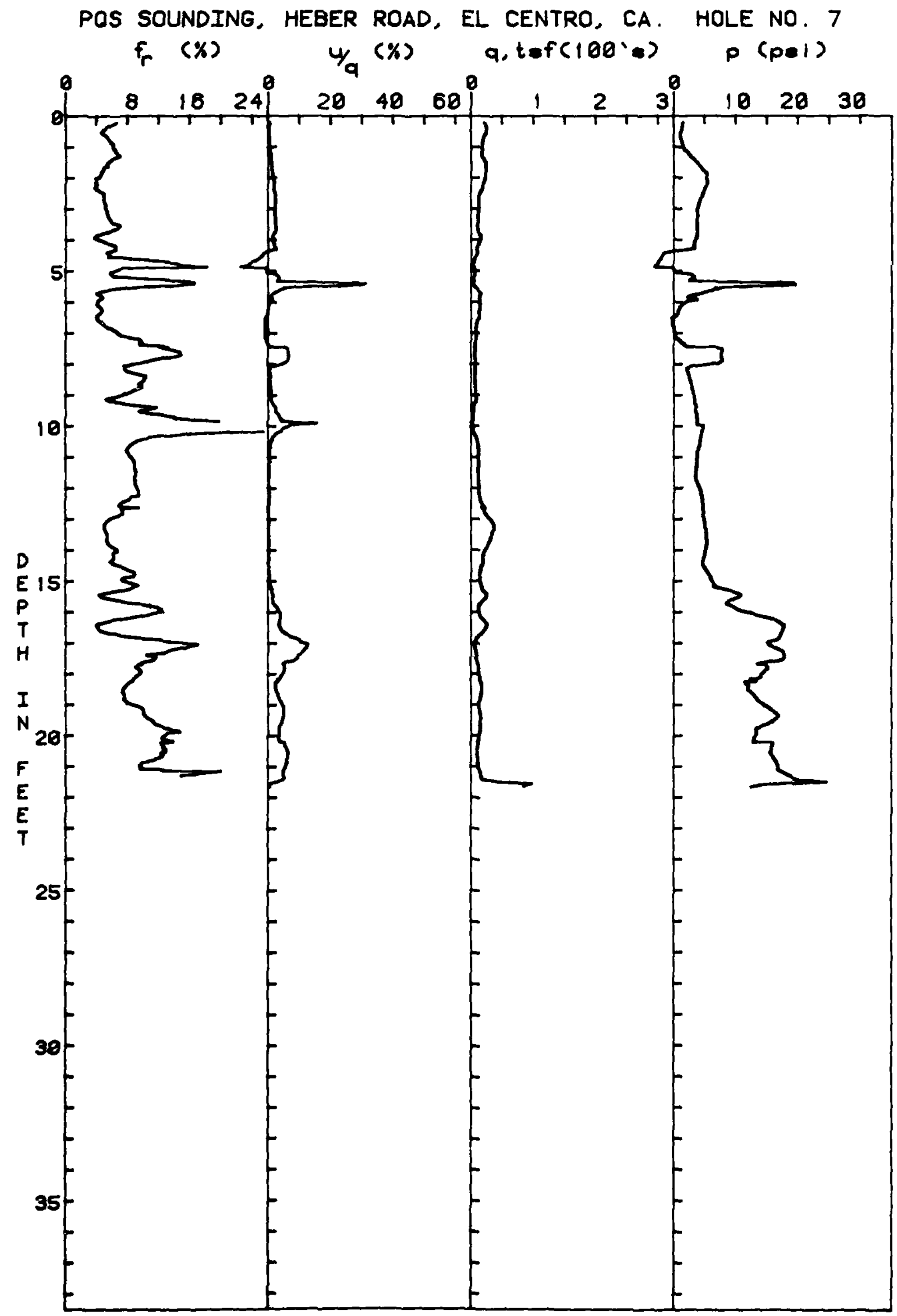


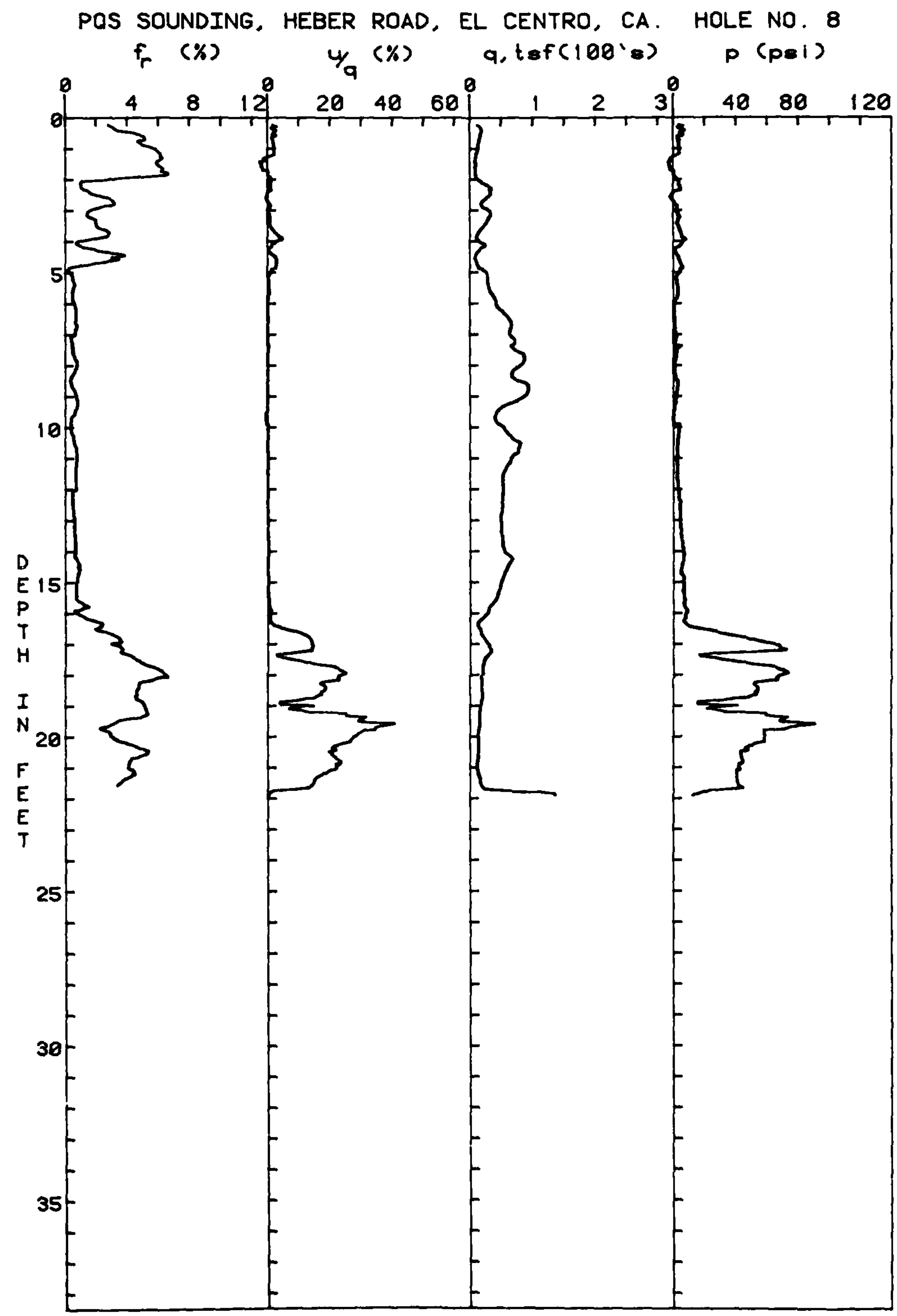

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 2A
f_r (%)
u/q (%)
q, tsf (100's)
p (psi)
0 6 12 18
0 4 8 12
0 1 2 3
0 40 80 120
0
5
10
15
20
25
30
35
DEPTH IN FEET

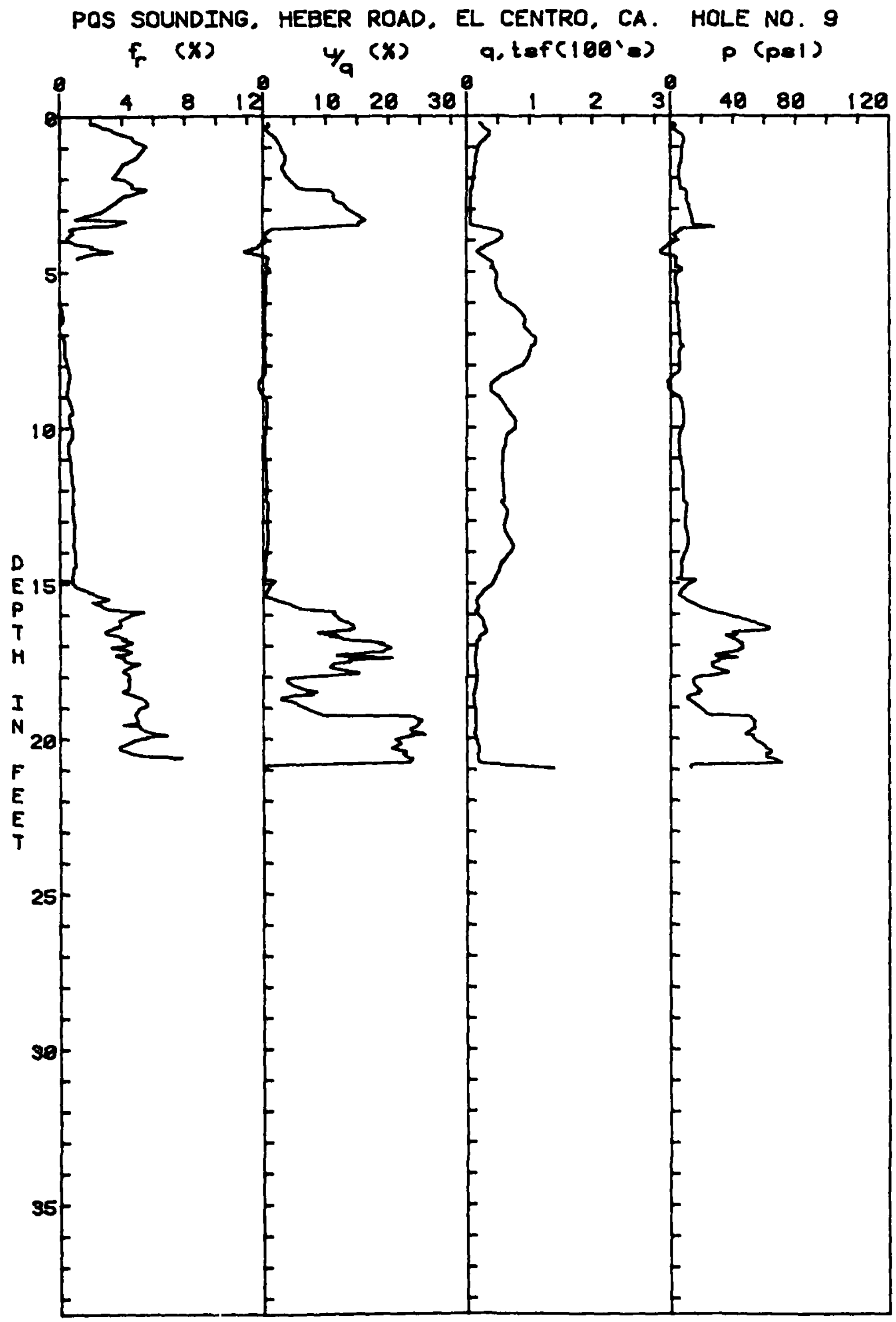

POS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 3
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 4 8 12
0 10 20 30
0 1 2 3
0 40 80 120
0 5 10 15 20 25 30 35
DEPTH IN FEET

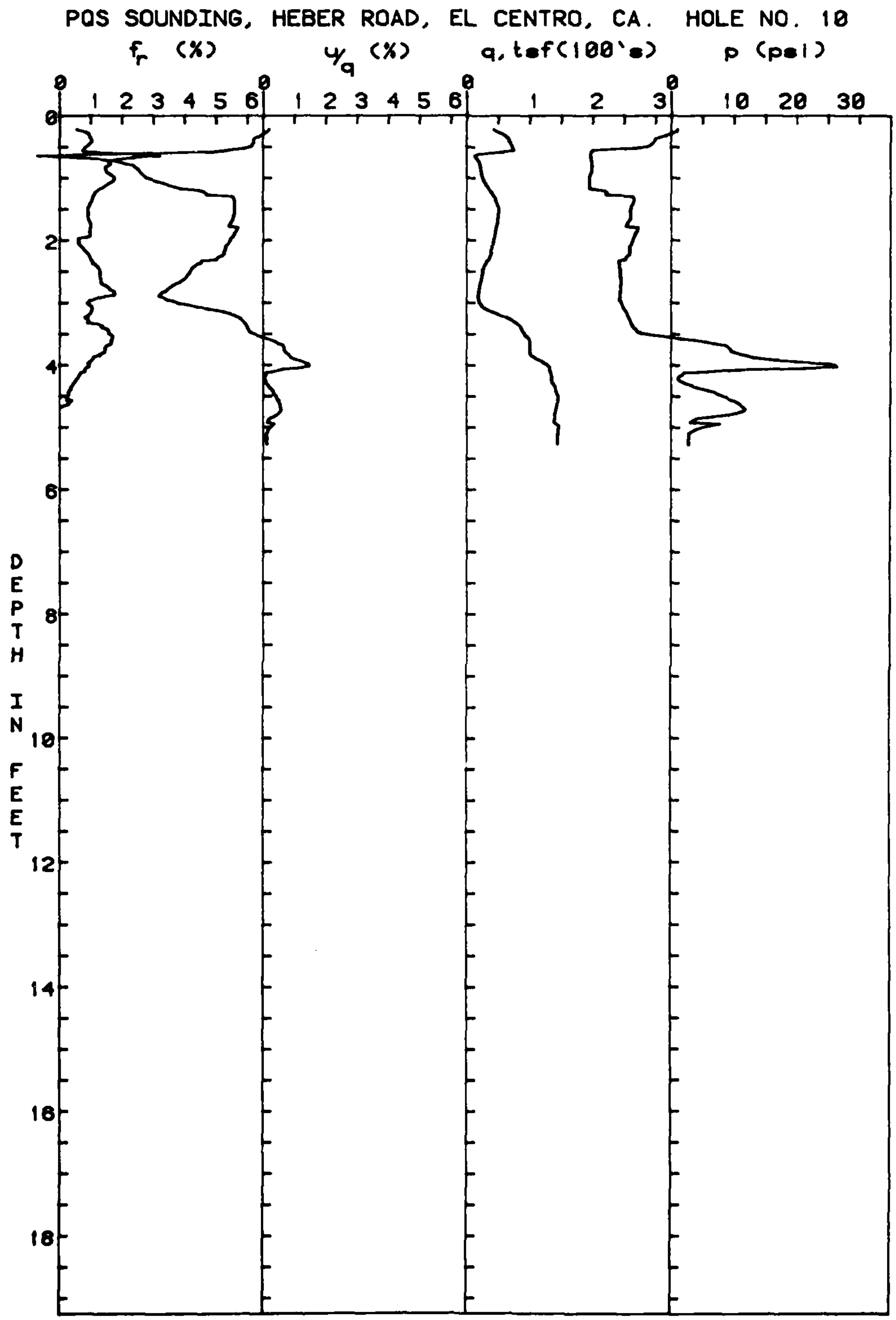

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 4
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 4 8 12
0 20 40 60
0 1 2 3
0 40 80 120
DEPTH IN FEET
0
5
10
15
20
25
30
35

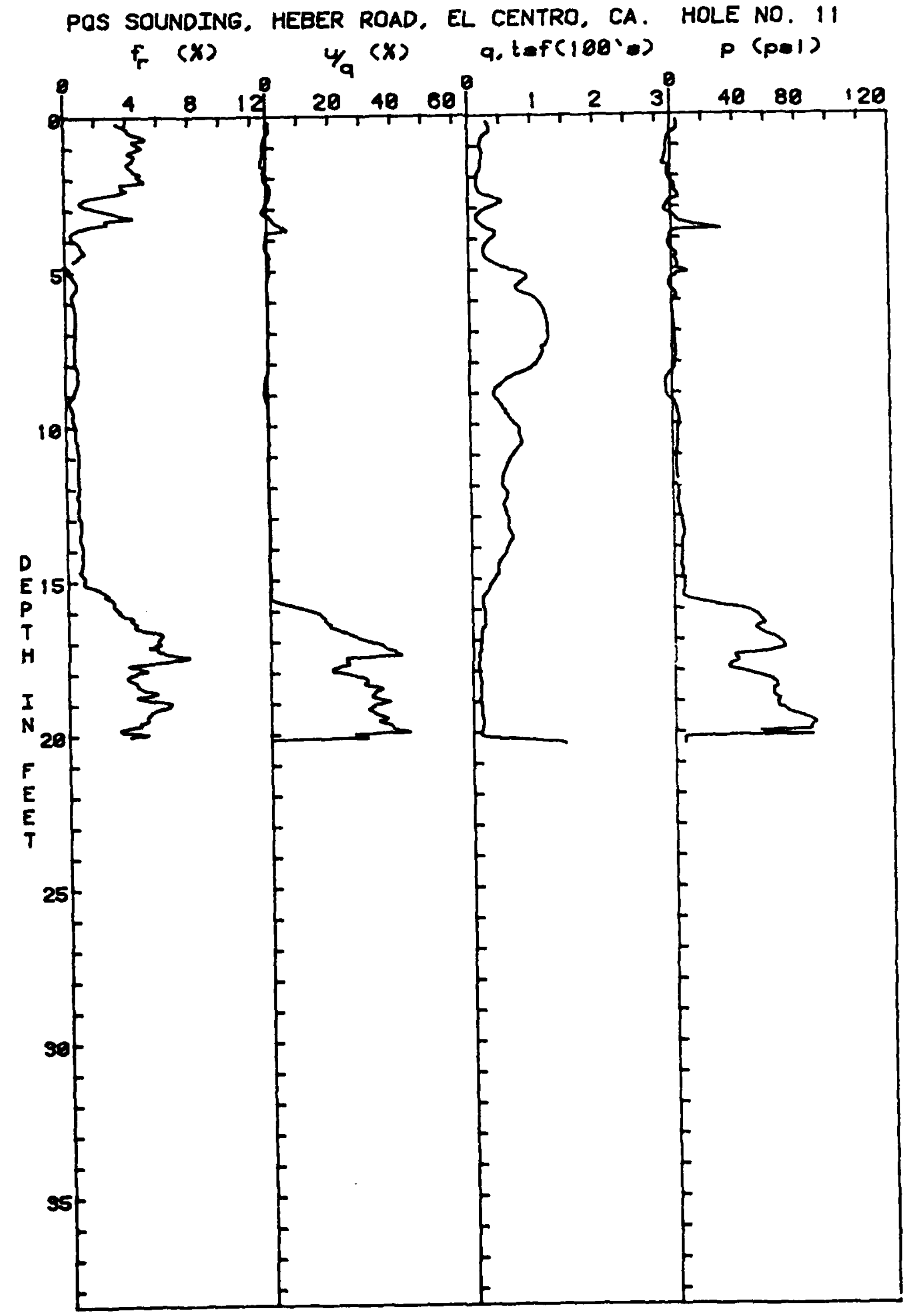

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 5
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 6 12 18
0 20 40 60
0 1 2 3
0 40 80 120
0
5
10
15
20
25
30
35
DEPTH IN FEET


POS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 7
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 8 16 24
0 20 40 60
0 1 2 3
0 10 20 30
0 5 10 15 20 25 30 35
DEPTH IN FEET


PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 8
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 4 8 12
0 20 40 60
0 1 2 3
0 40 80 120
DEPTH IN FEET
0
5
10
15
20
25
30
35

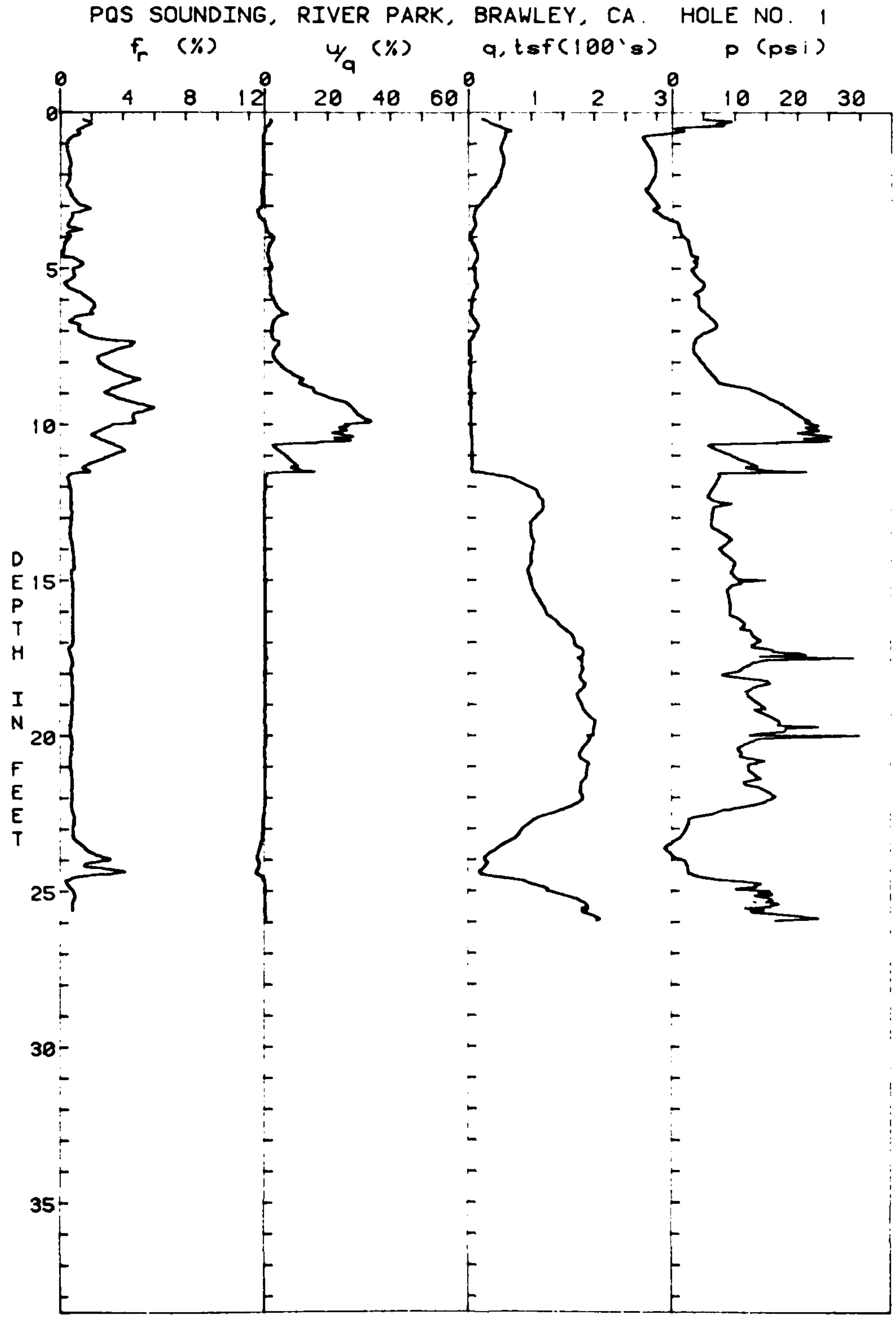

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 9
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 4 8 12
0 10 20 30
0 1 2 3
0 40 80 120
0 5 10 15 20 25 30 35
DEPTH IN FEET


PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 10
fr (%)
u/q (%)
q, tsf(100's)
p (psi)
0 1 2 3 4 5 6
0 1 2 3 4 5 6
0 1 2 3
0 10 20 30
DEPTH IN FEET
0
2
4
6
8
10
12
14
16
18

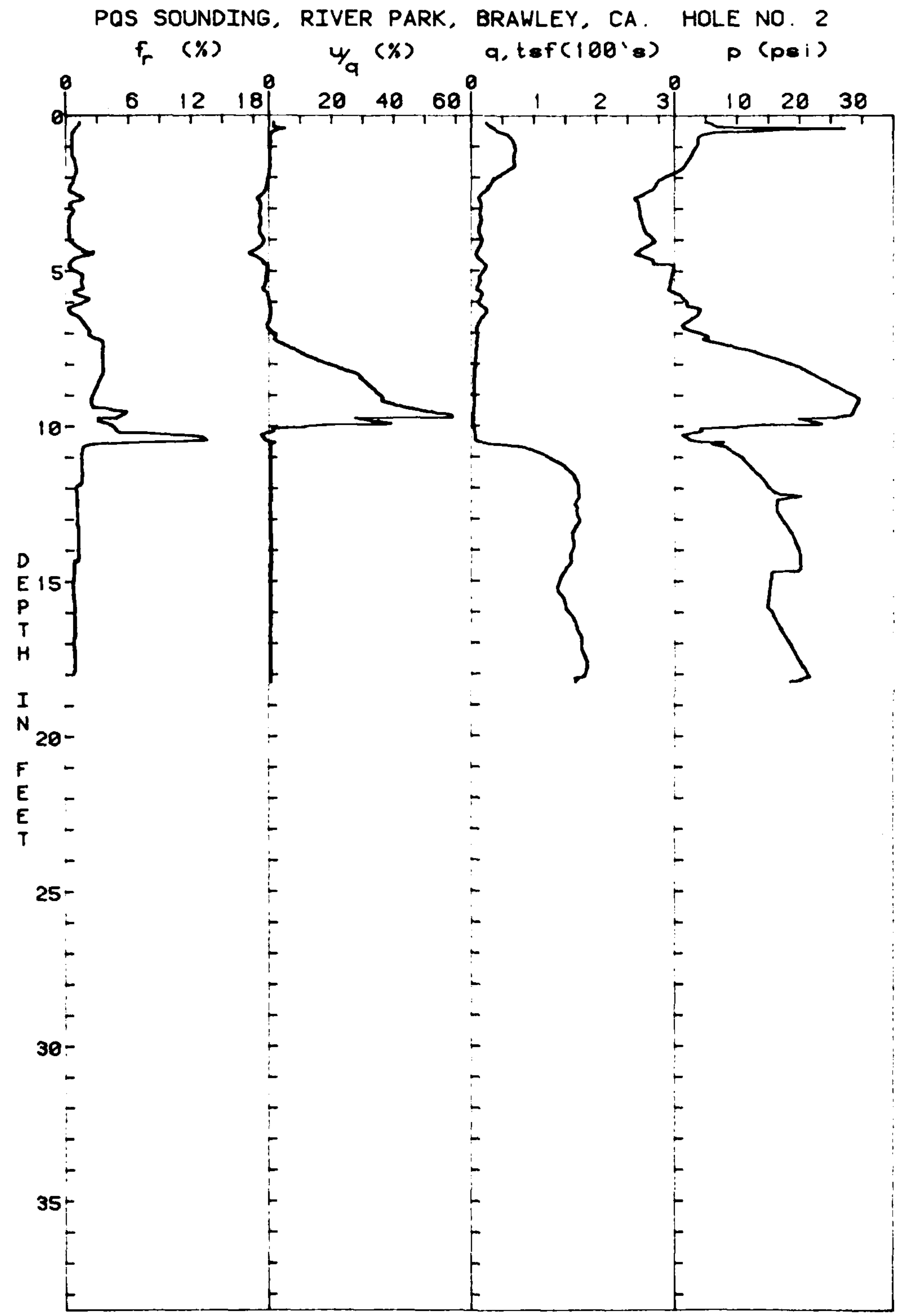

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 11
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 4 8 12
0 20 40 60
0 1 2 3
0 40 80 120
DEPTH IN FEET
0
5
10
15
20
25
30
35

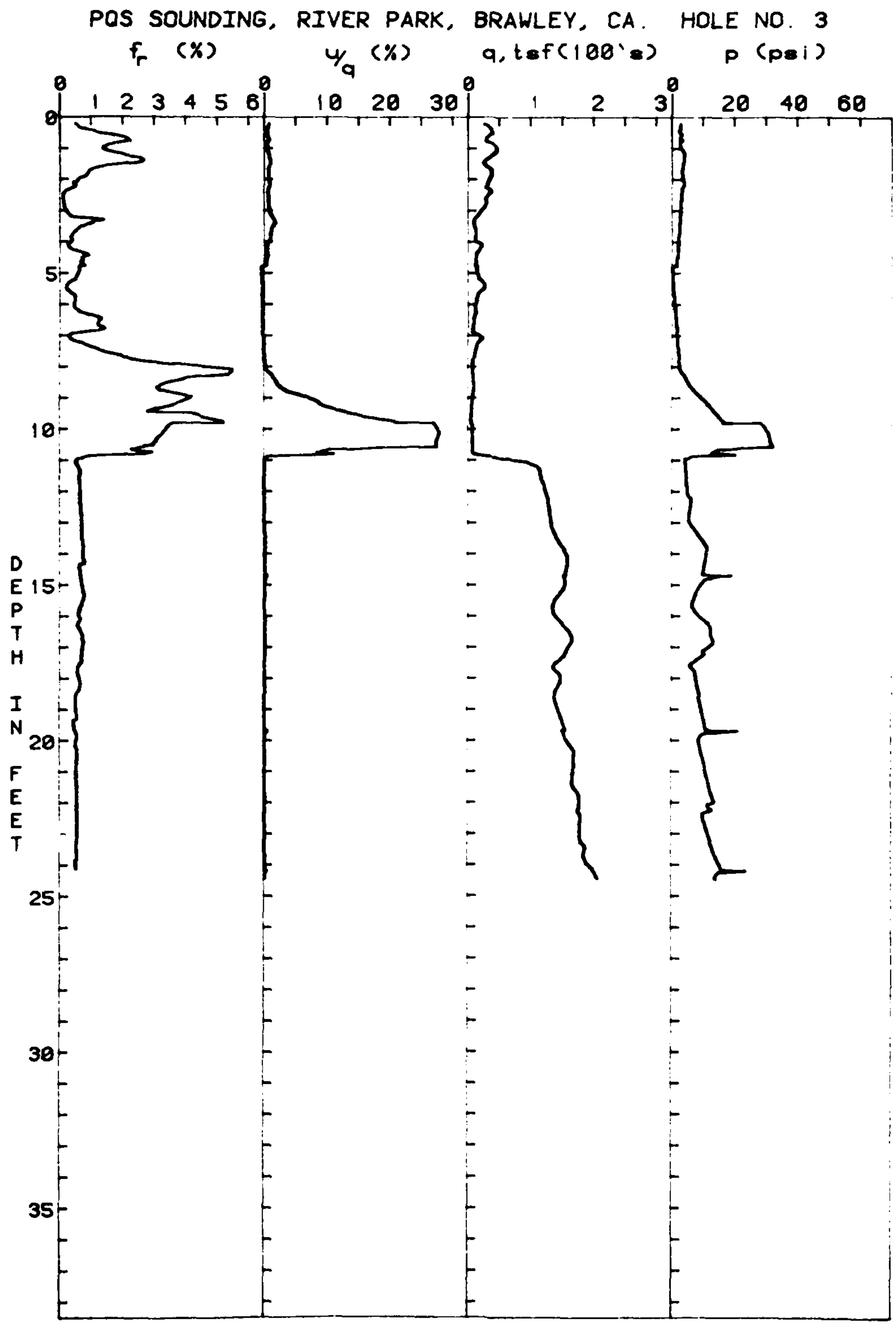
APPENDIX C

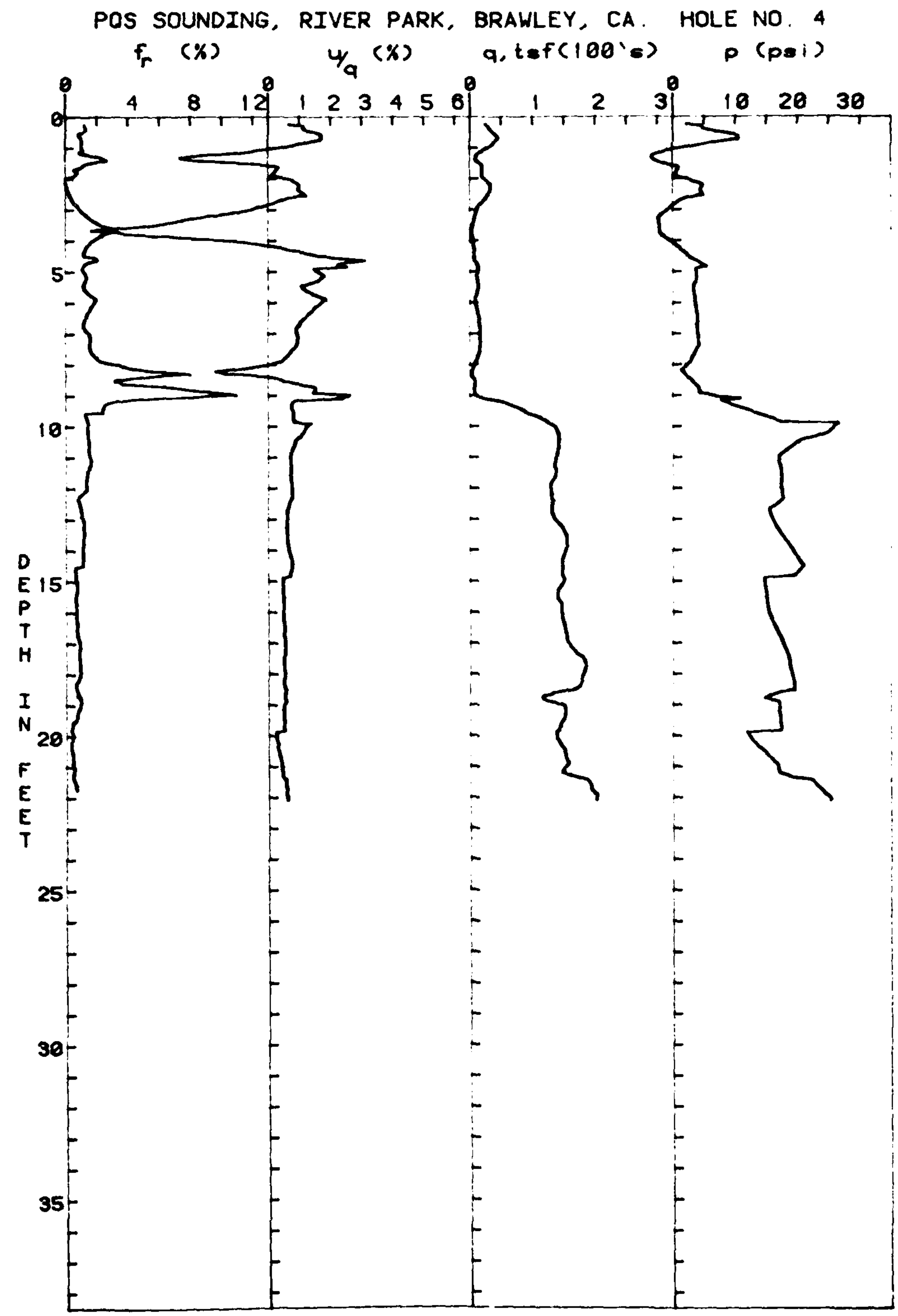
LOGS OF PQS HOLES AT RIVER PARK SITE

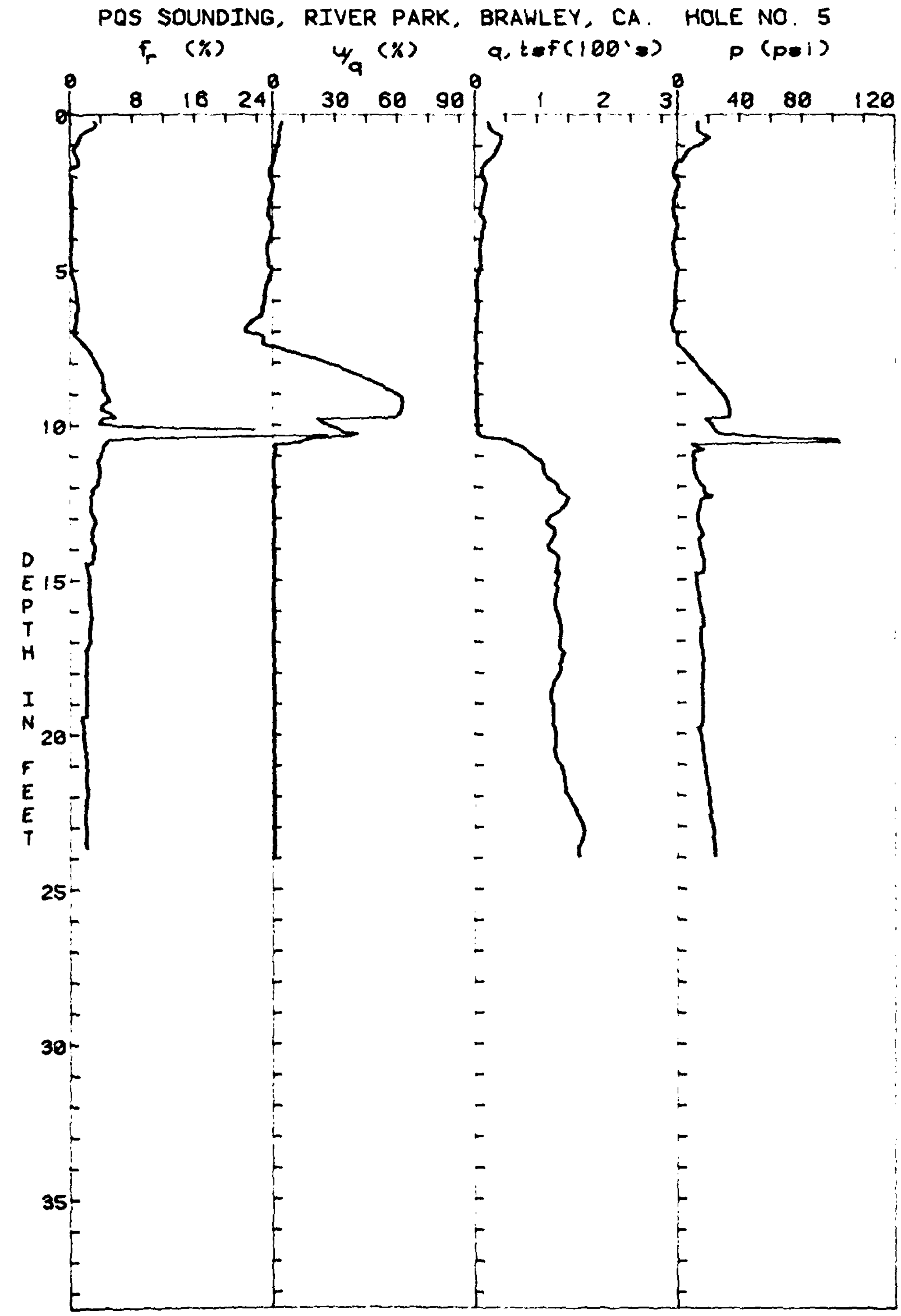


PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 1
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
DEPTH IN FEET

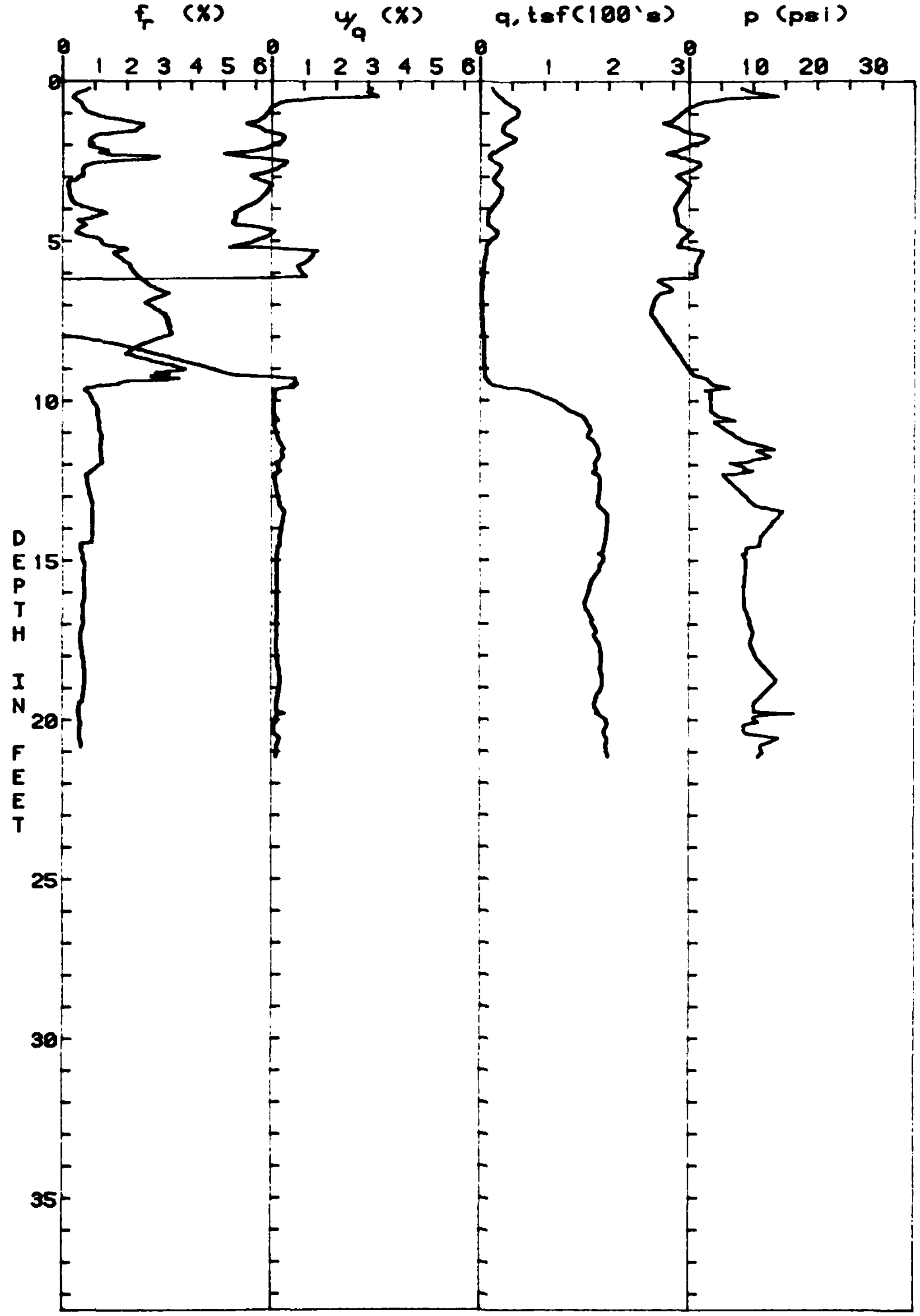

PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 2
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 6 12 18
0 20 40 60
0 1 2 3
0 10 20 30
0
5
10
15
20
25
30
35
DEPTH IN FEET


PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 3
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 1 2 3 4 5 6
0 10 20 30
0 1 2 3
0 20 40 60
0 5 10 15 20 25 30 35
DEPTH IN FEET

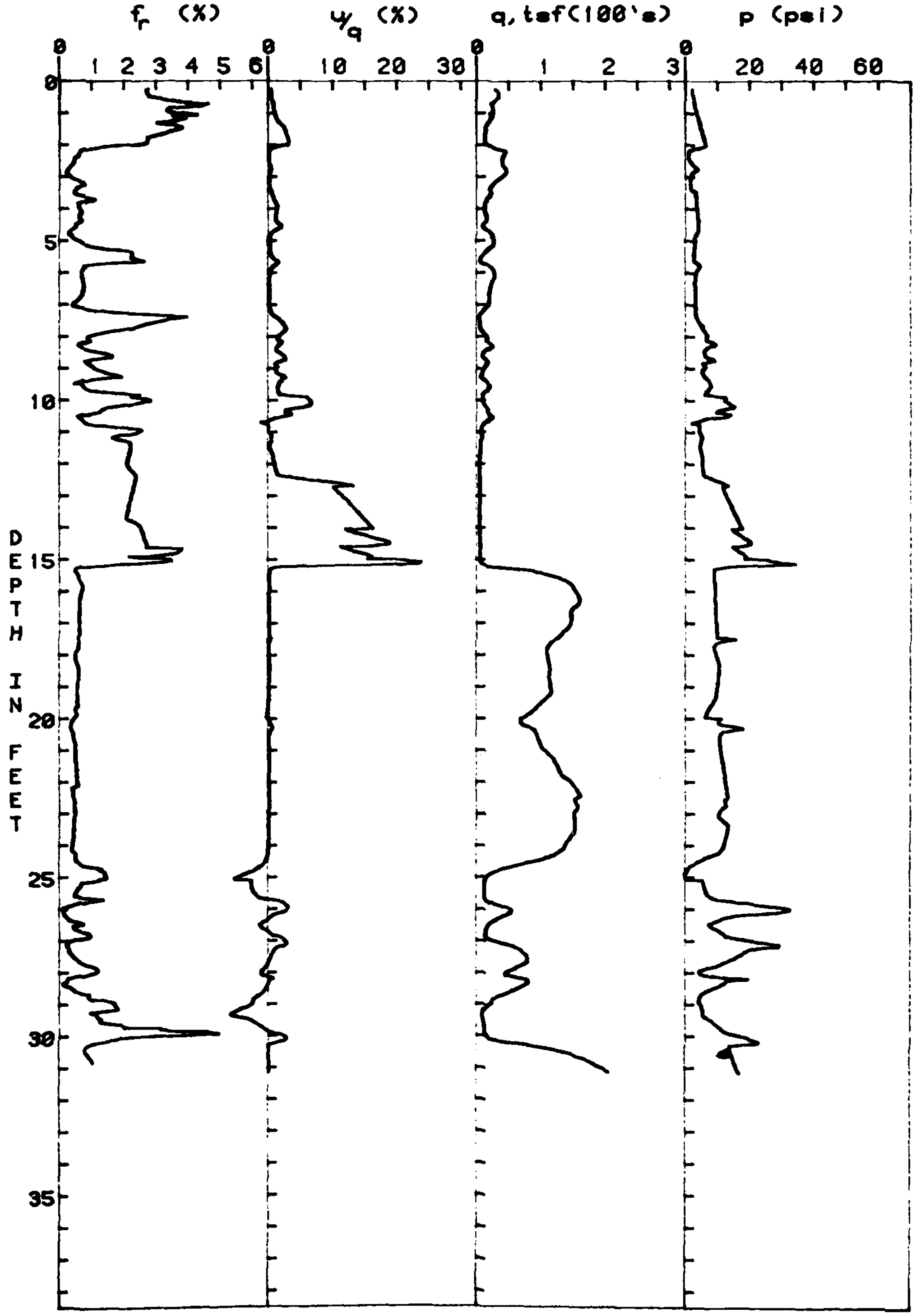

PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 4
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 4 8 12
0 1 2 3 4 5 6
0 1 2 3
0 10 20 30
DEPTH IN FEET
0 5 10 15 20 25 30 35


PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 5
fr (%)
u/q (%)
q, tsf(100's)
p (psi)
0 8 16 24
0 30 60 90
0 1 2 3
0 40 80 120
DEPTH IN FEET
0
5
10
15
20
25
30
35

POS SOUNDING, RIVER PARK BRAWLEY, CA. HOLE NO. 6

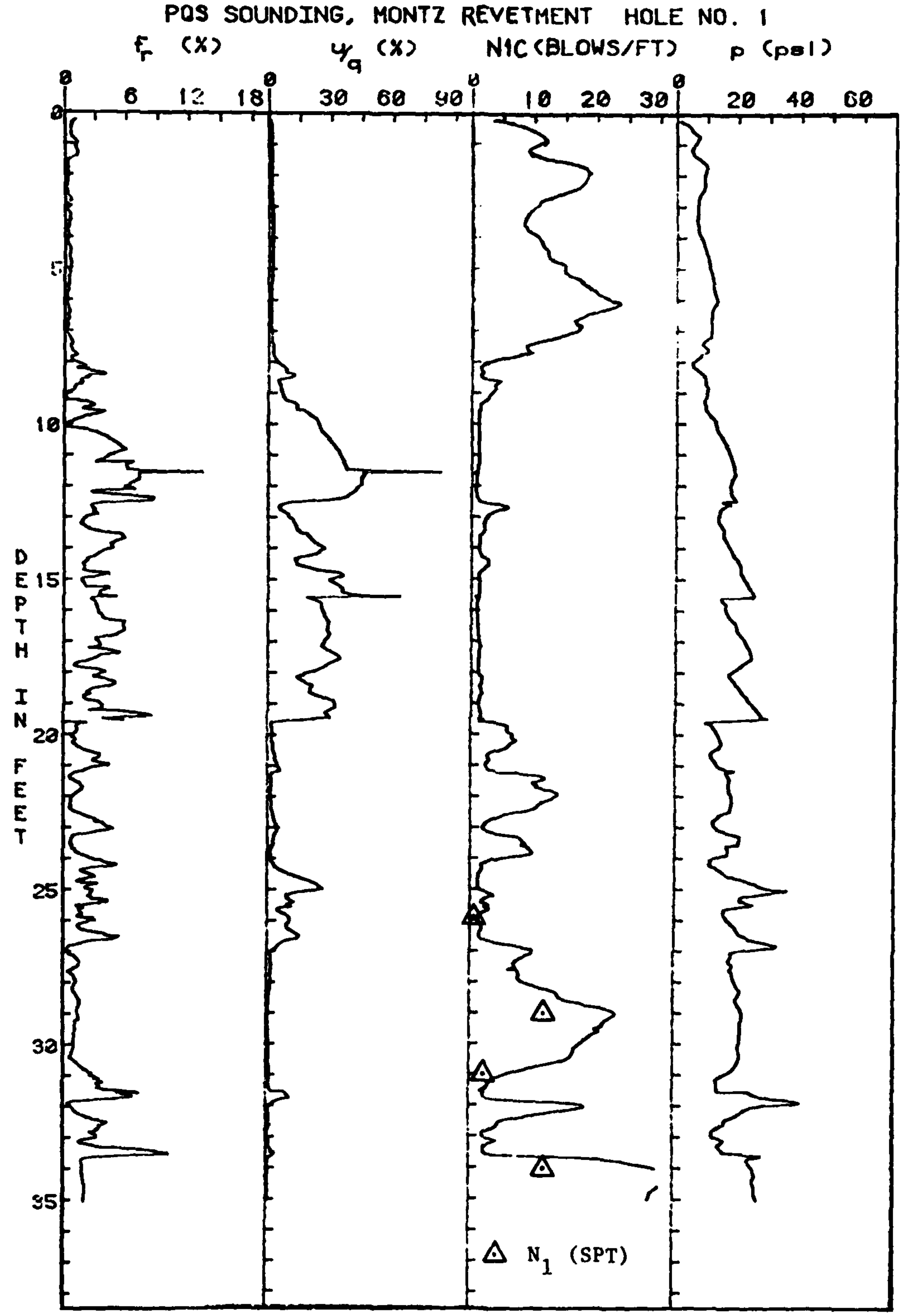


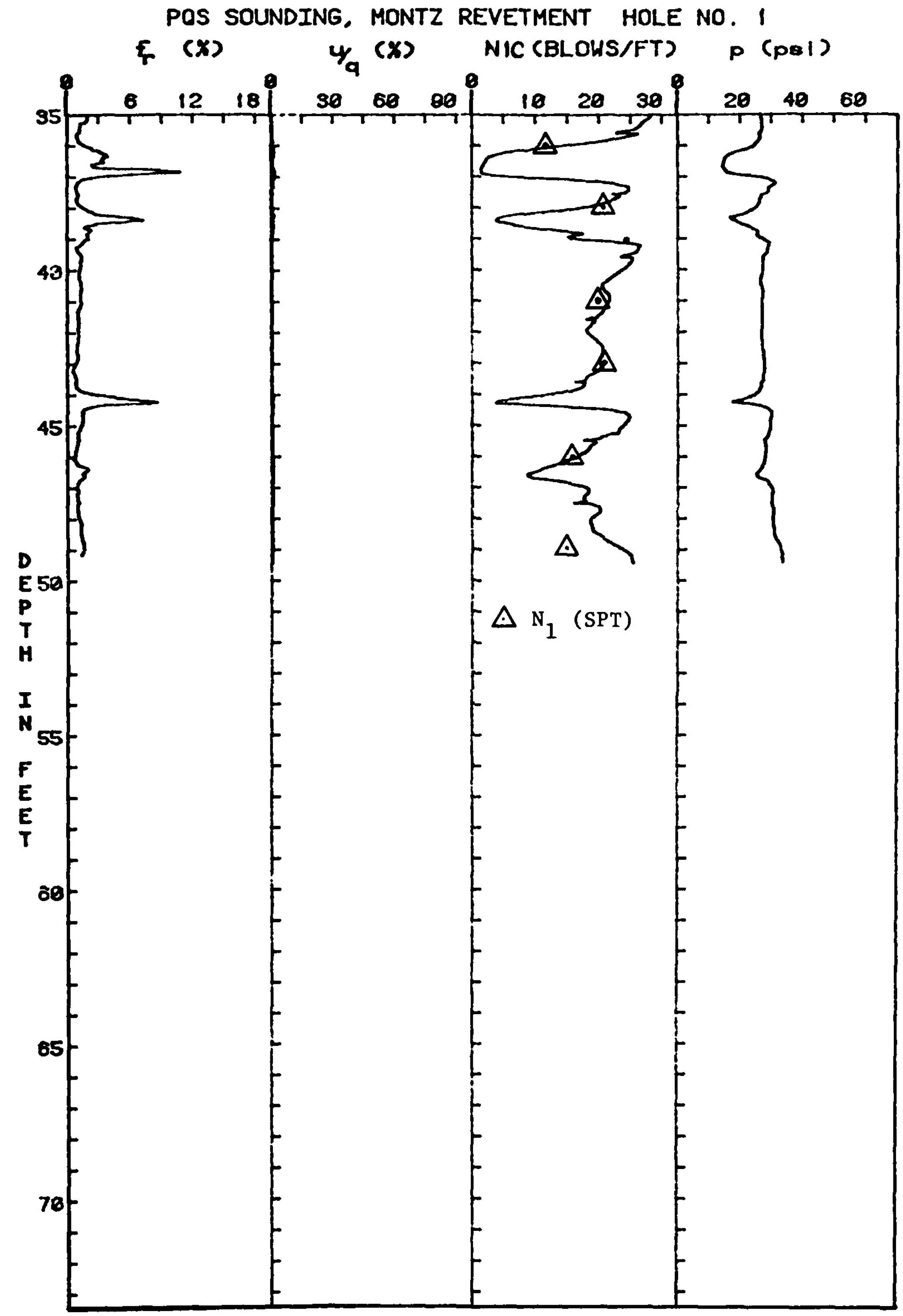
PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 7



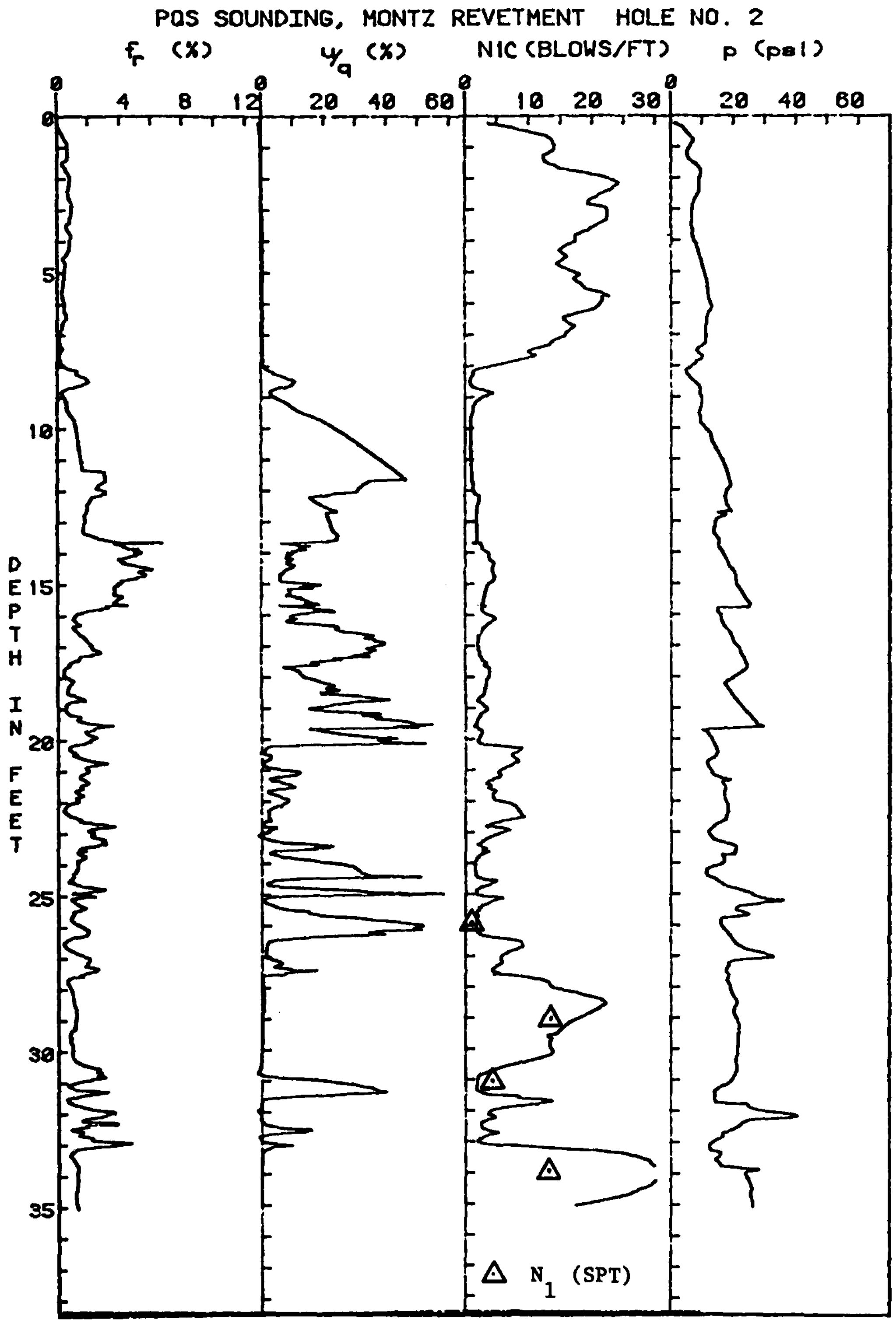
APPENDIX D

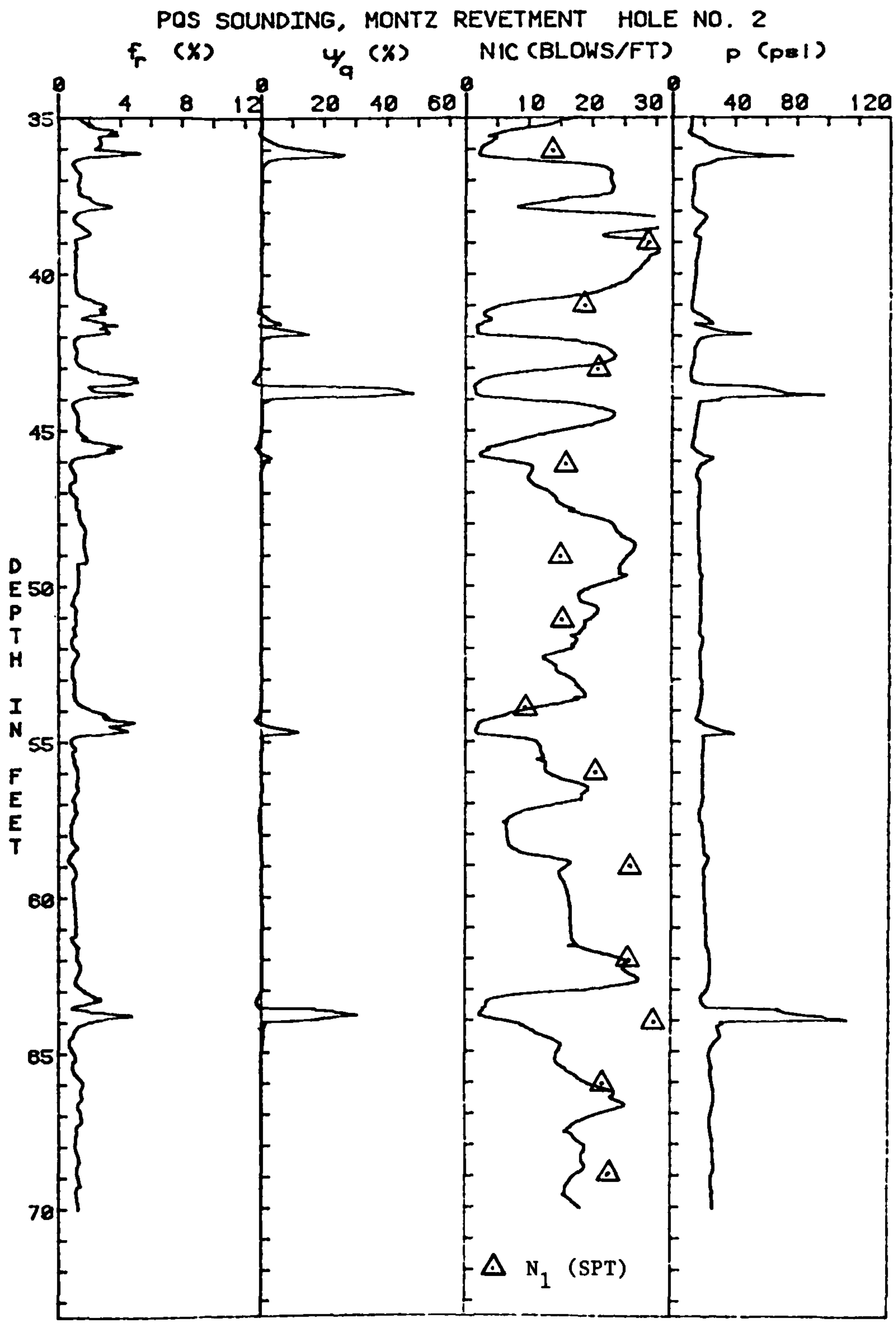
PQS SOUNDINGS FOR MONTZ SITE

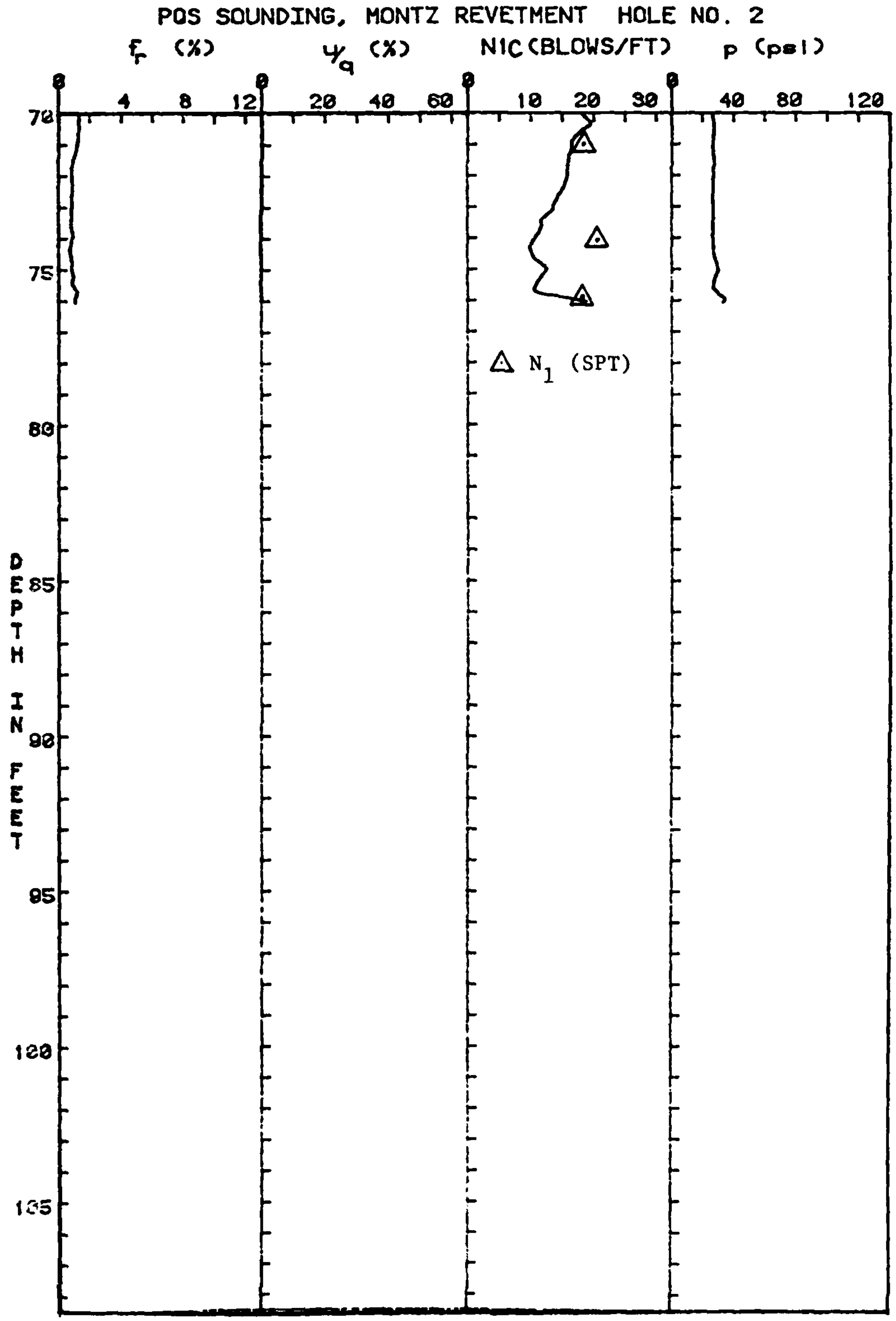

PQS SOUNDING, MONTZ REVETMENT HOLE NO. 1
fr (%)
u/q (%)
N1C (BLOWS/FT)
p (psi)
0 6 12 18
0 30 60 90
0 10 20 30
0 20 40 60
0
5
10
15
20
25
30
35
DEPTH IN FEET
N1 (SPT)



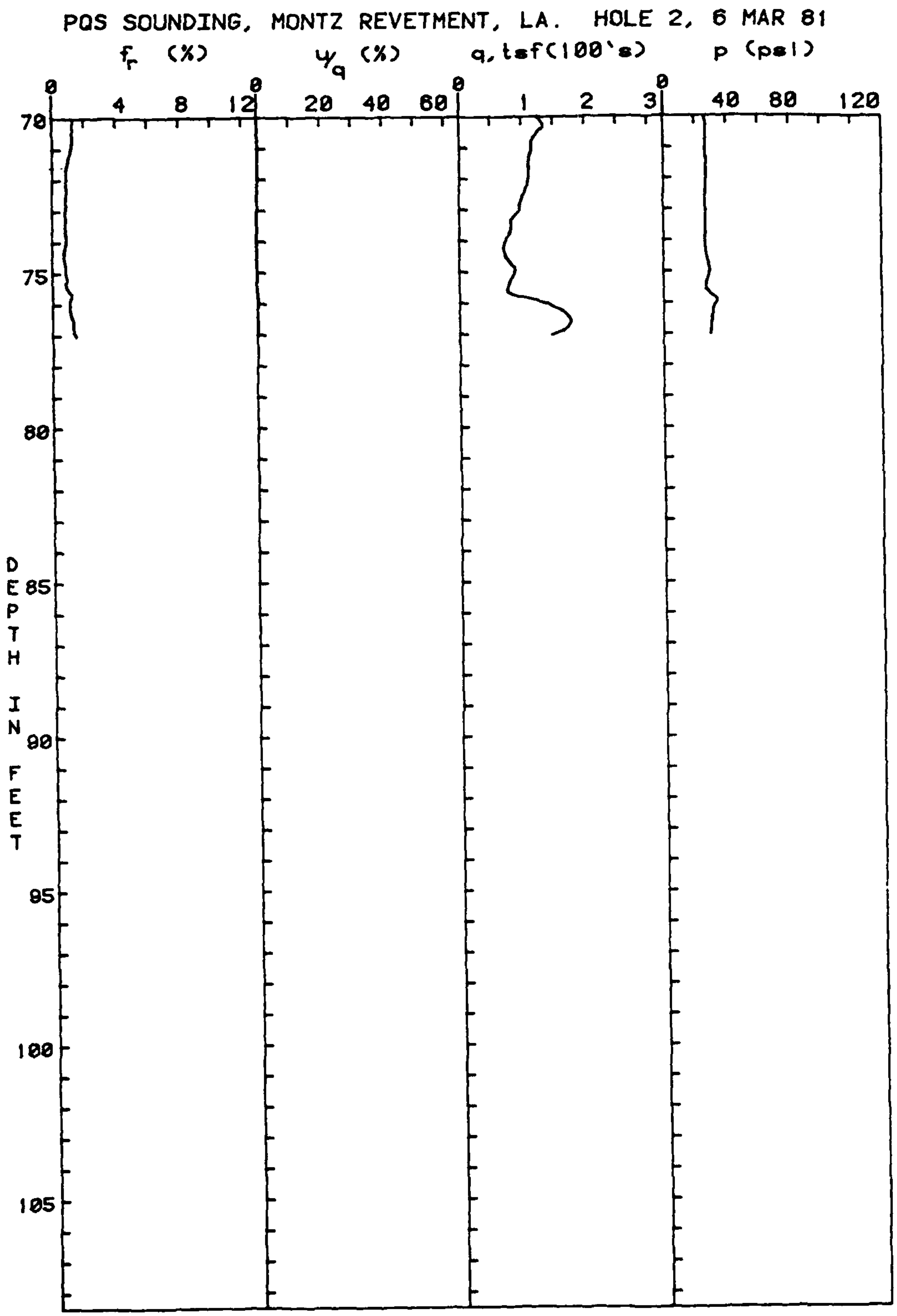
PQS SOUNDING, MONTZ REVETMENT HOLE NO. 1
f_r (%)
ψ_q (%)
N1C (BLOWS/FT)
p (psi)
0 6 12 18
0 30 60 90
0 10 20 30
0 20 40 60
DEPTH IN FEET
35
40
45
50
55
60
65
70
△ N_1 (SPT)


PQS SOUNDING, MONTZ REVETMENT HOLE NO. 2
f_r (%)
u/q (%)
N1C (BLOWS/FT)
p (psi)
0 4 8 12
0 20 40 60
0 10 20 30
0 20 40 60
DEPTH IN FEET
0
5
10
15
20
25
30
35
△ N_1 (SPT)


PQS SOUNDING, MONTZ REVETMENT HOLE NO. 2
f_r (%)
ψ_q (%)
N1C (BLOWS/FT)
p (psi)
0 4 8 12
0 20 40 60
0 10 20 30
0 40 80 120
35
40
45
50
55
60
65
70
DEPTH IN FEET
N1 (SPT)

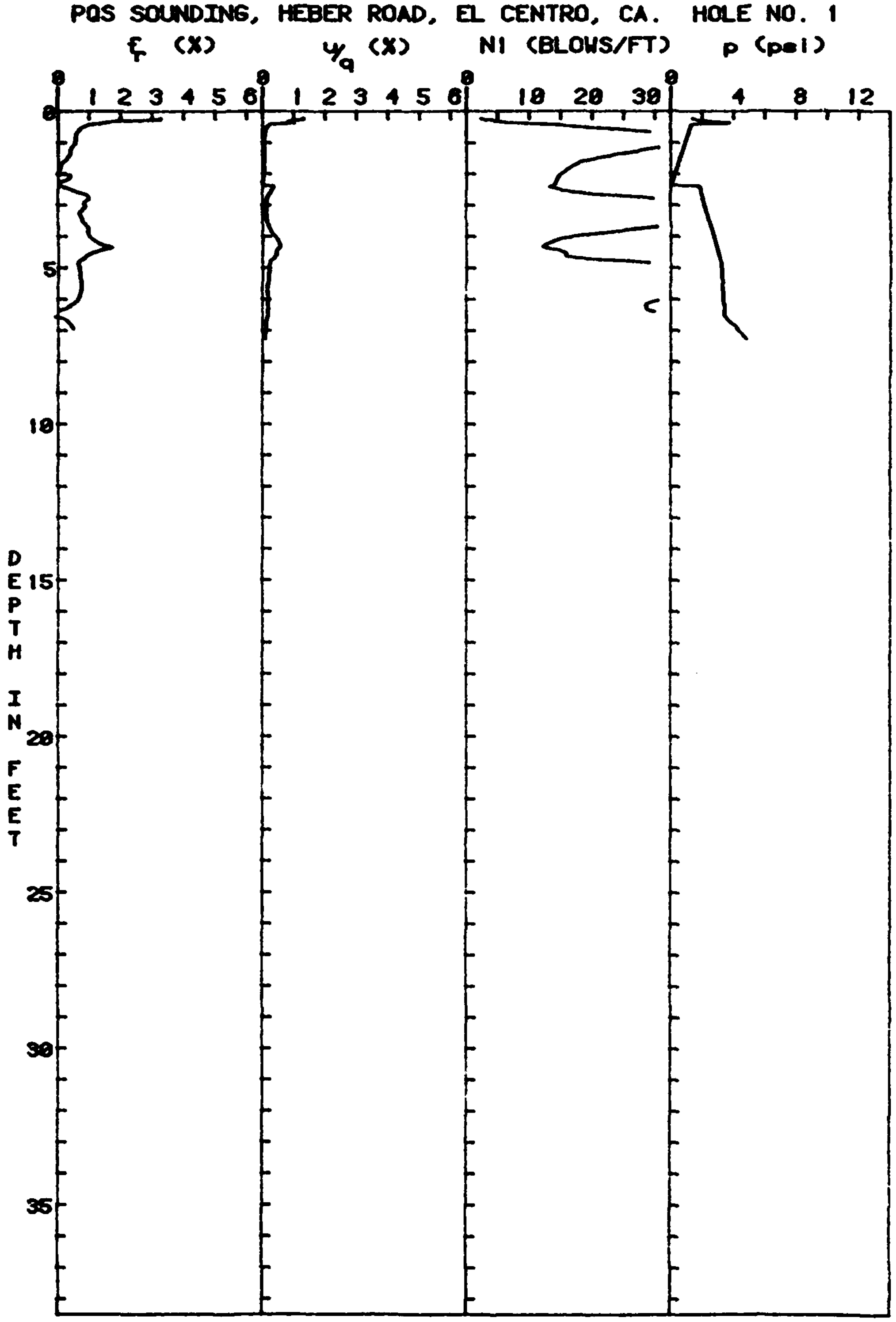


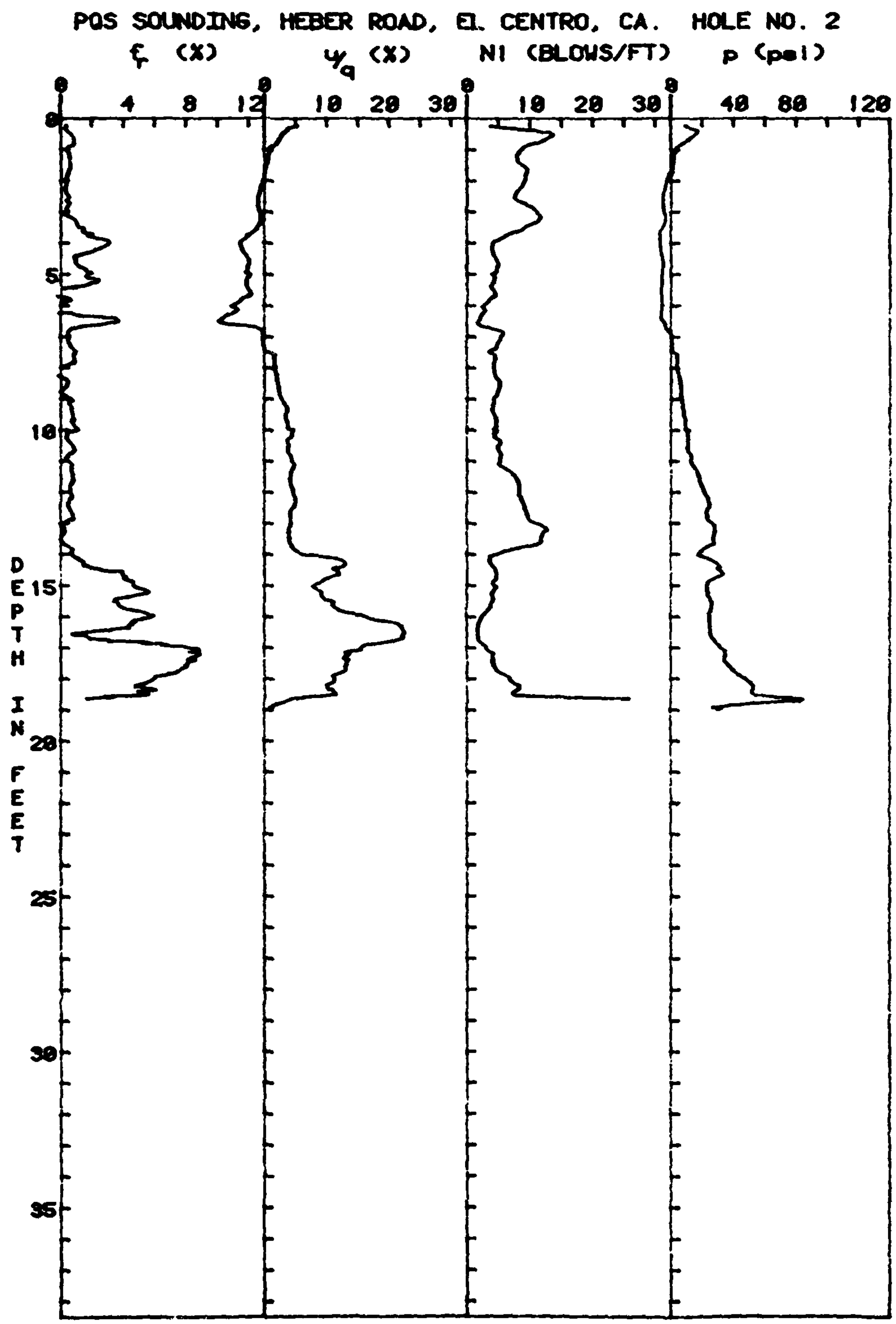
POS SOUNDING, MONTZ REVETMENT HOLE NO. 2
f_r (%)
ψ_q (%)
N1C (BLOWS/FT)
p (psi)
0
4
8
12
0
20
40
60
0
10
20
30
0
40
80
120
70
75
80
85
90
95
100
105
DEPTH IN FEET
△ N_1 (SPT)

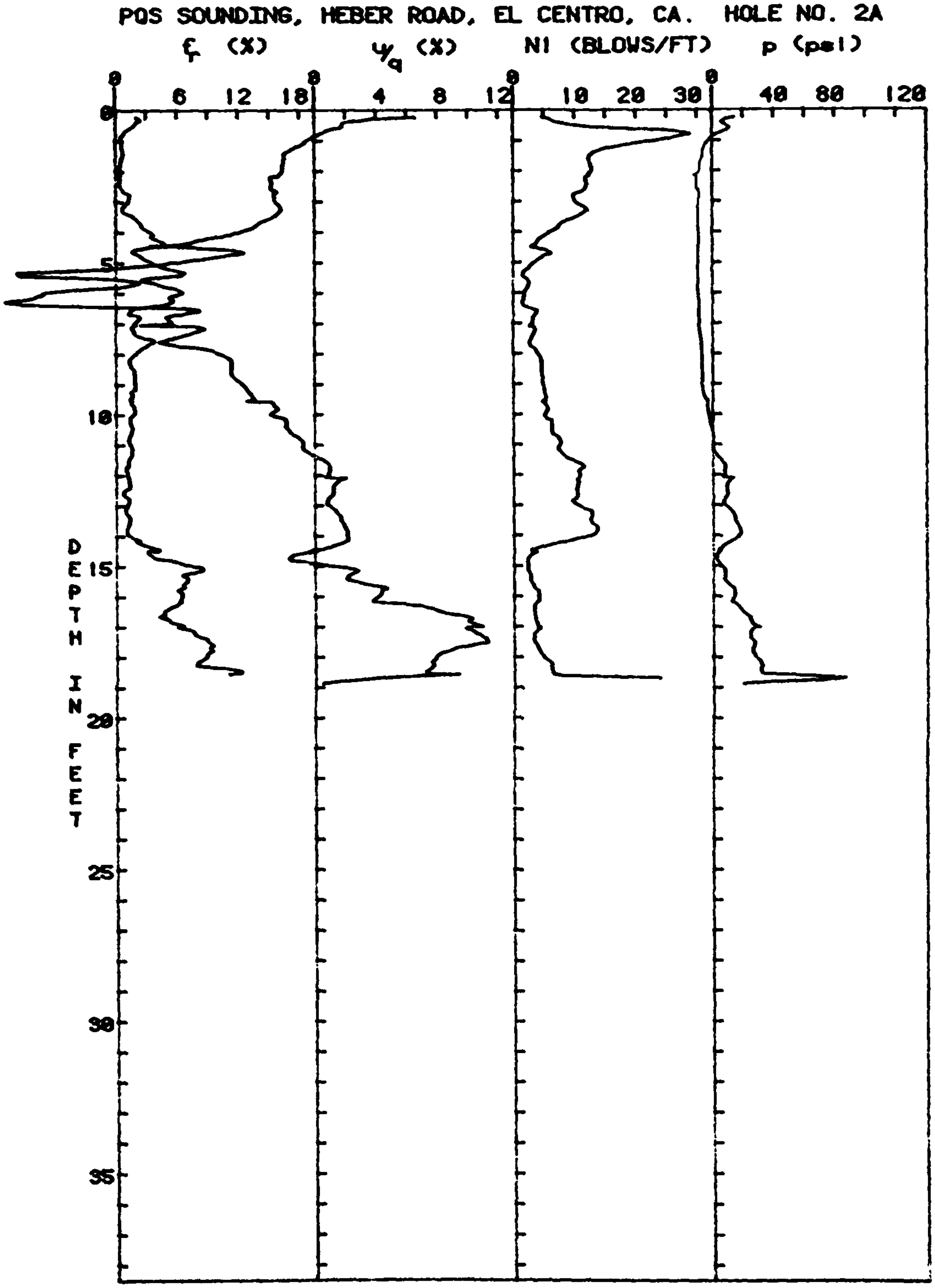

PQS SOUNDING, MONTZ REVETMENT, LA. HOLE 2, 6 MAR 81
f_r (%)
u/q (%)
q, tsf(100's)
p (psi)
0 4 8 12
0 20 40 60
0 1 2 3
0 40 80 120
DEPTH IN FEET
70
75
80
85
90
95
100
105

APPENDIX E

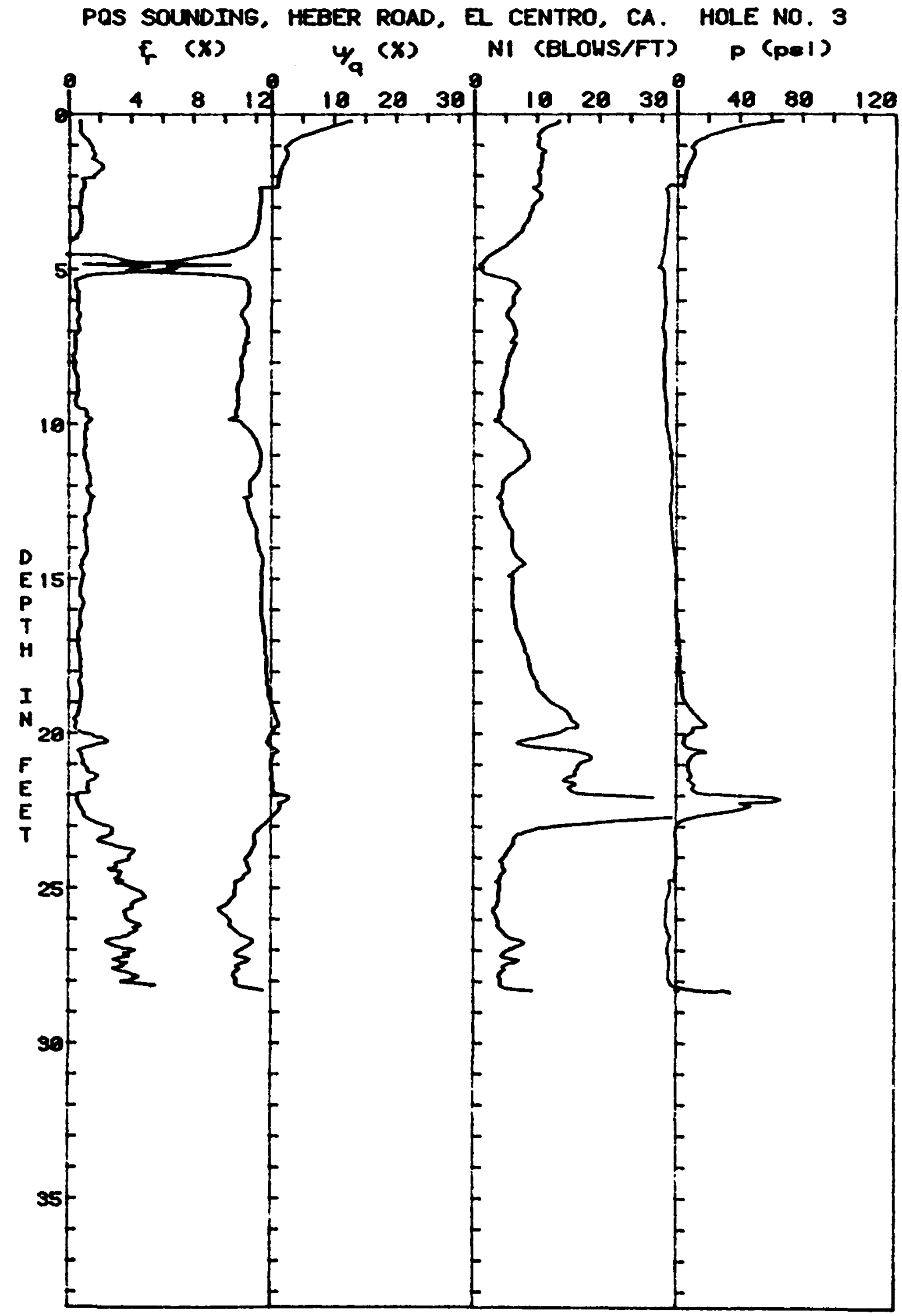
PQS SOUNDINGS FOR HEBER ROAD SITE


PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 1
f (%)
ψ/q (%)
N1 (BLOWS/FT)
p (psi)
0 1 2 3 4 5 6
0 1 2 3 4 5 6
0 10 20 30
0 4 8 12
DEPTH IN FEET
0
5
10
15
20
25
30
35

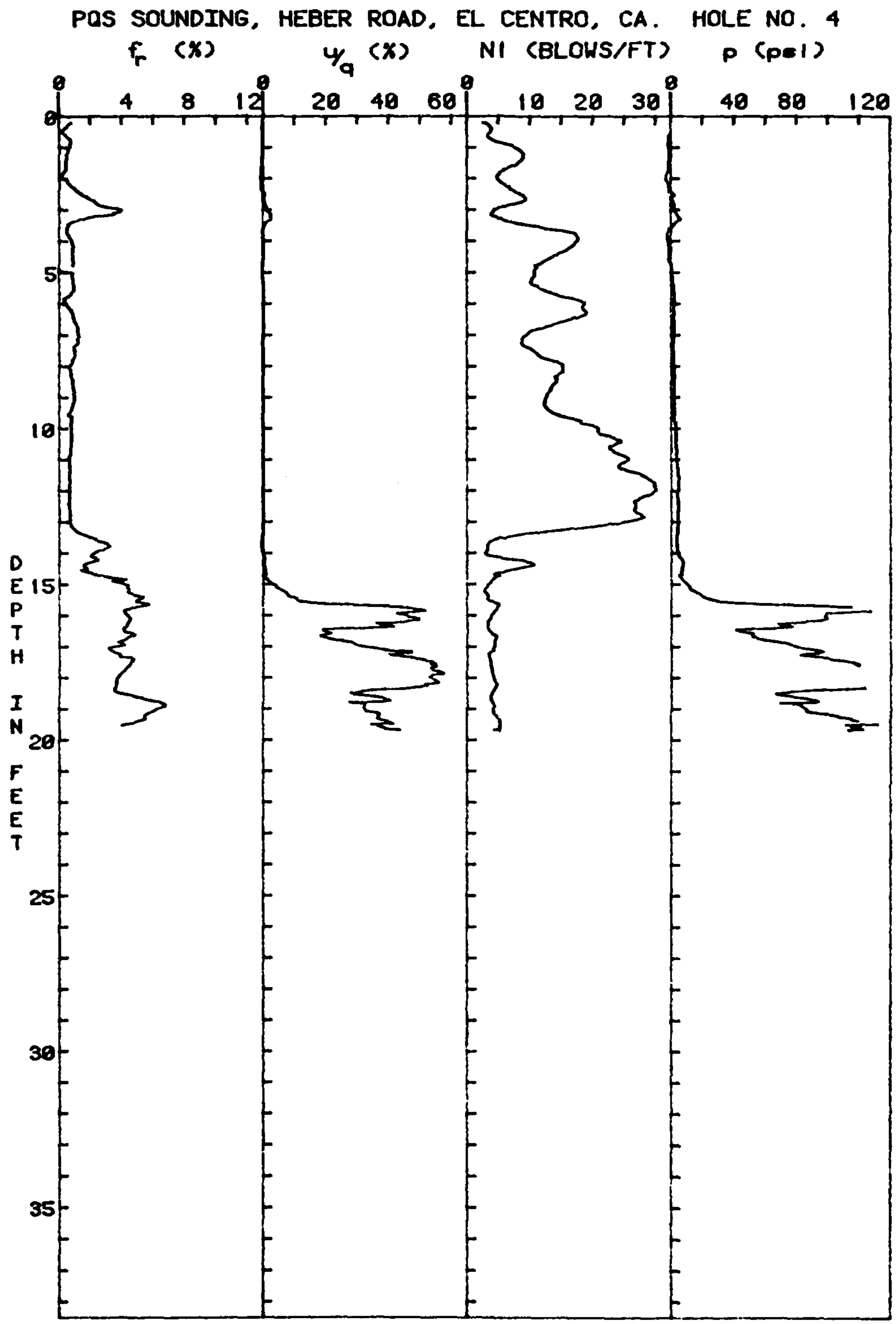

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 2
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
DEPTH IN FEET

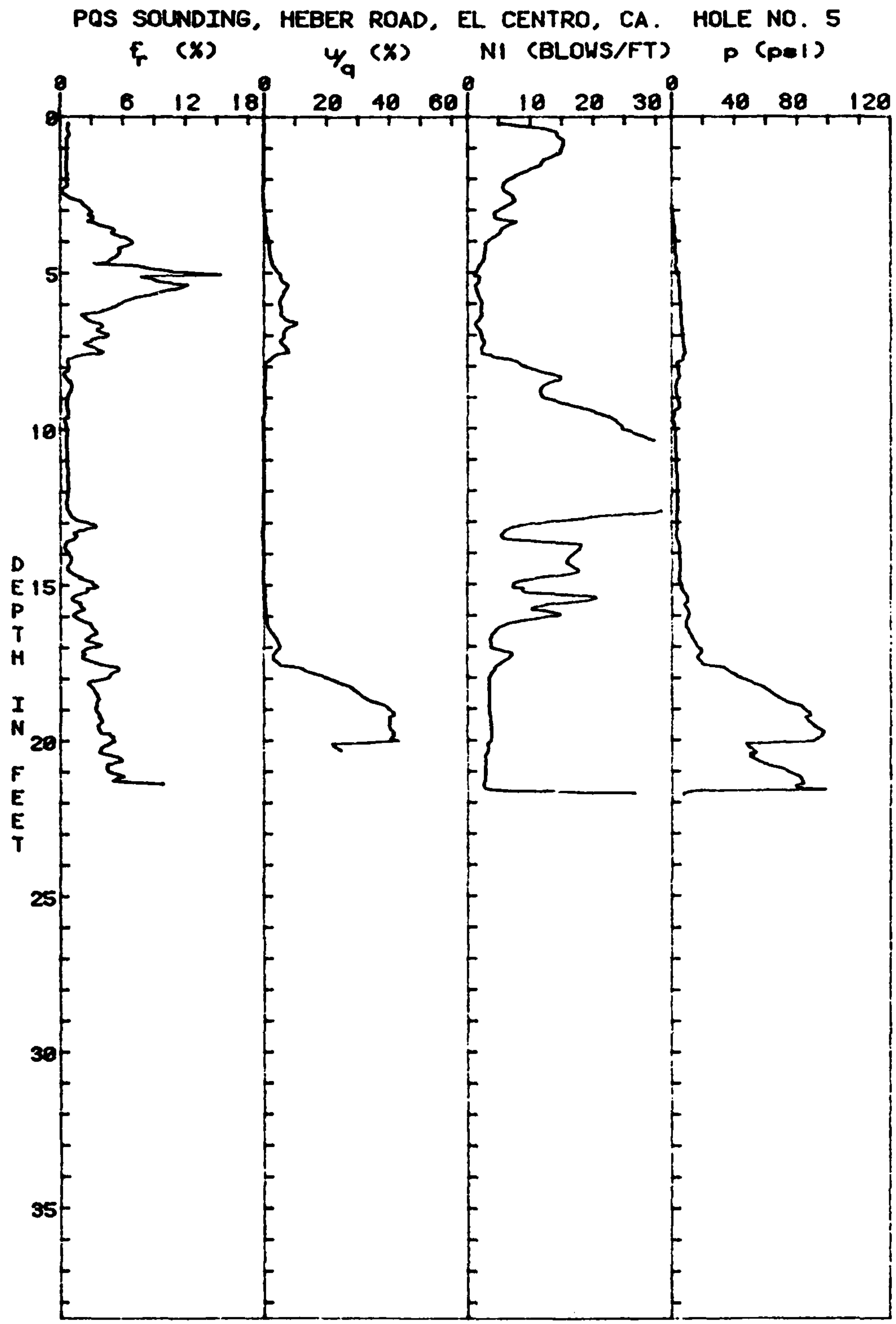


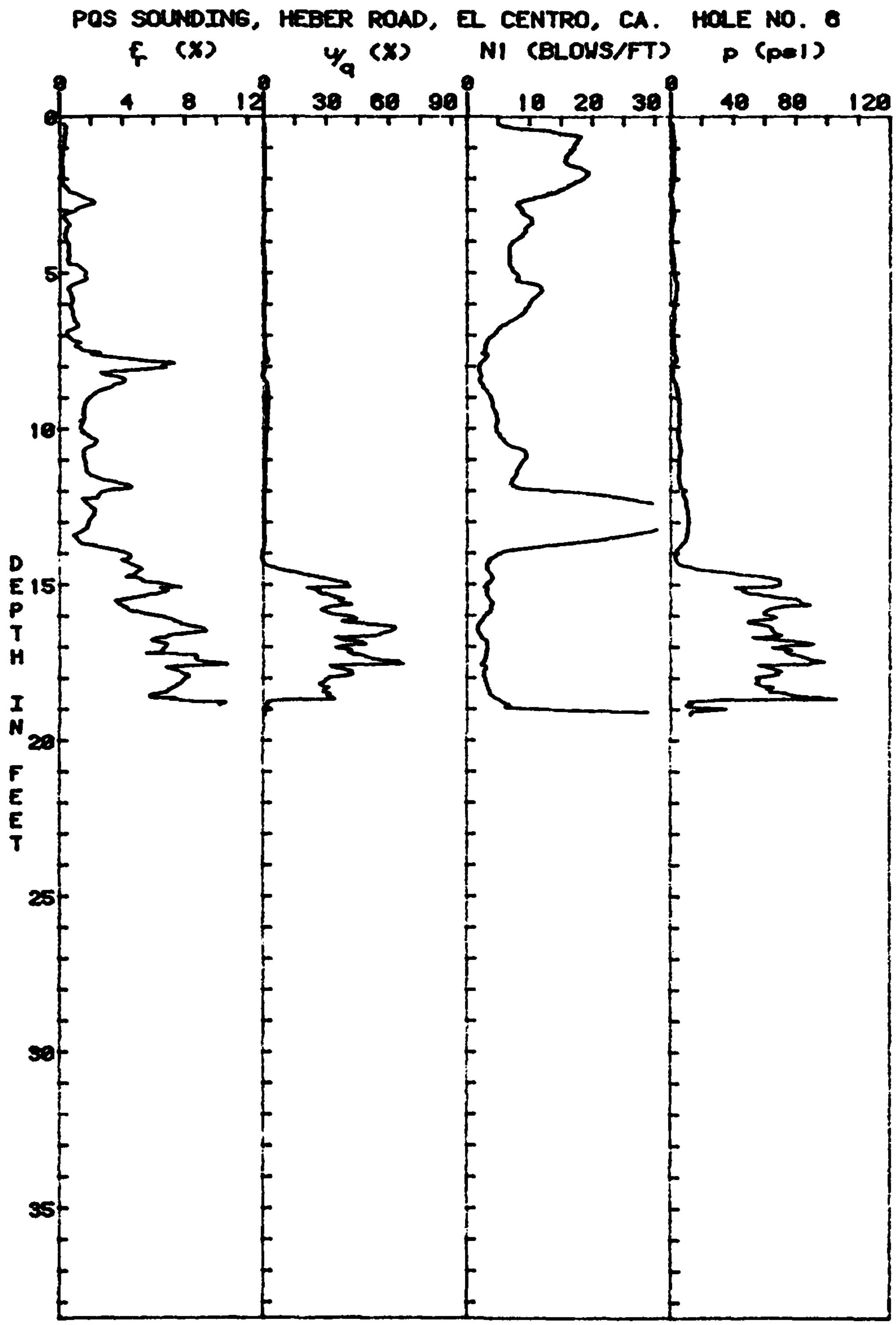
PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 2A
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 6 12 18
0 4 8 12
0 10 20 30
0 40 80 120
0
5
10
15
20
25
30
35
DEPTH IN FEET

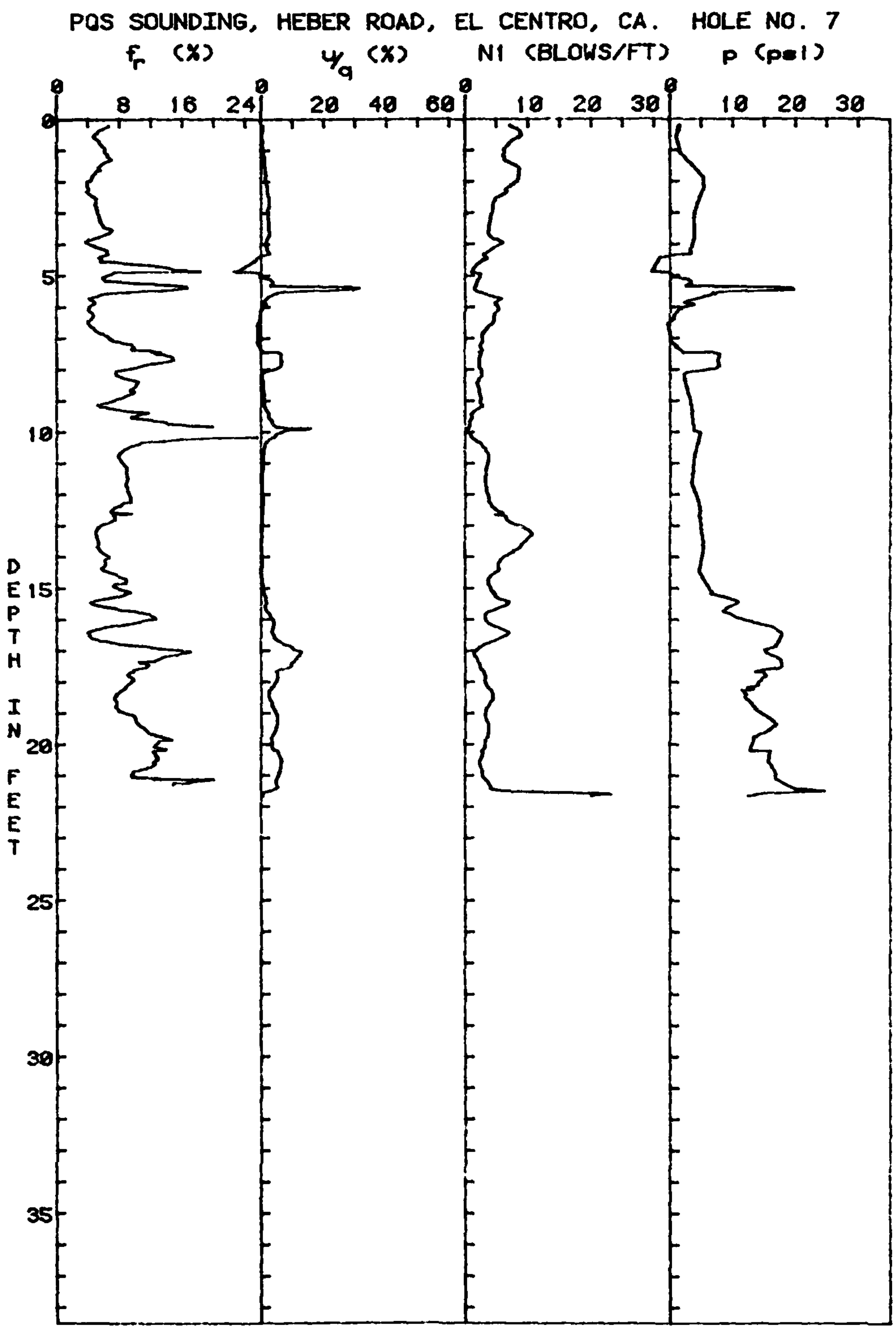


PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 3
f_r (%)
u/q (%)
NI (BLOWS/FT)
p (psi)
0
4
8
12
0
10
20
30
0
10
20
30
0
40
80
120
0
5
10
15
20
25
30
35
DEPTH IN FEET

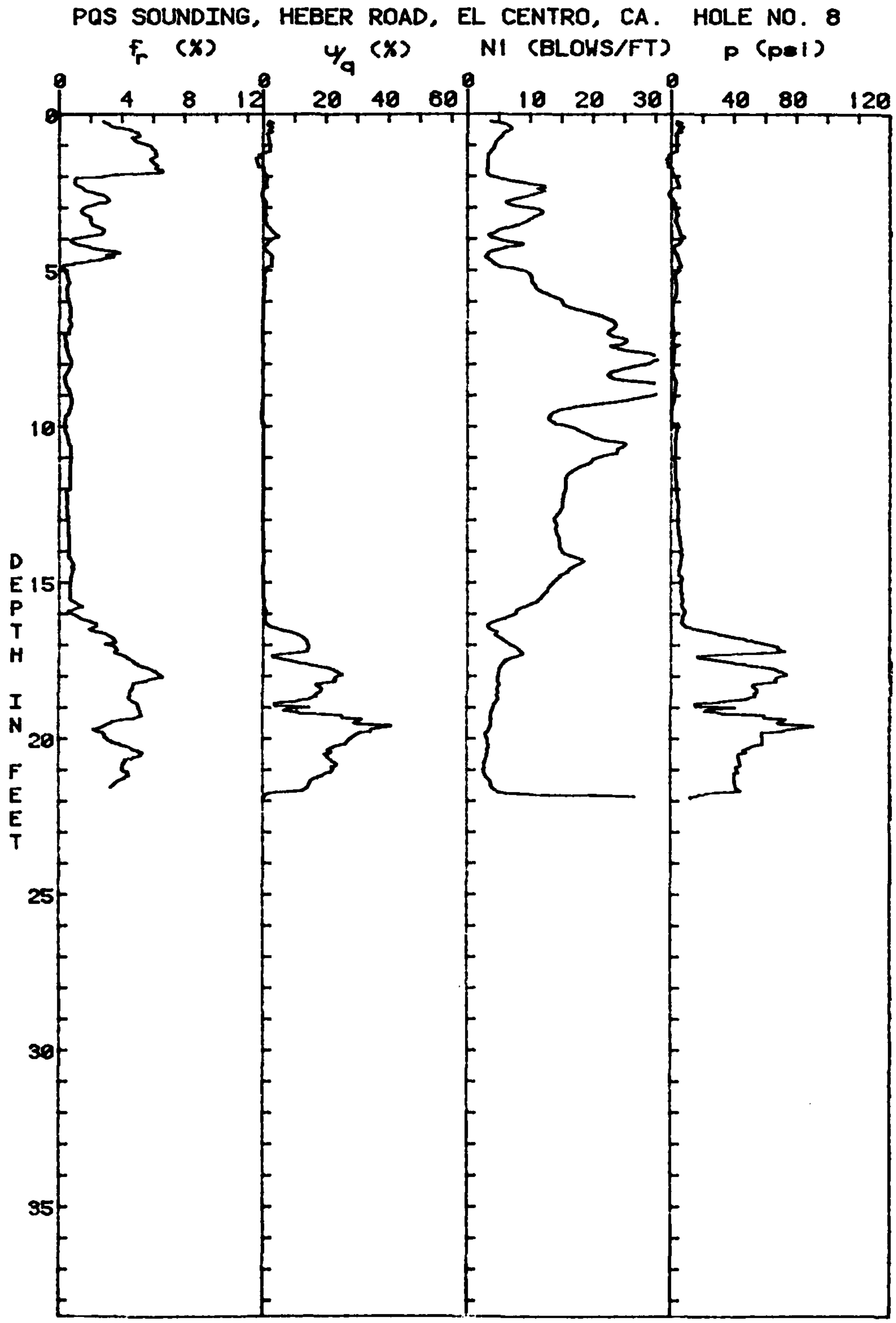

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 4
f_r (%)
u/q (%)
NI (BLOWS/FT)
p (psi)
0 4 8 12
0 20 40 60
0 10 20 30
0 40 80 120
DEPTH IN FEET
0 5 10 15 20 25 30 35


PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 5
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0
6
12
18
20
40
60
10
20
30
40
80
120
5
10
15
20
25
30
35
DEPTH IN FEET

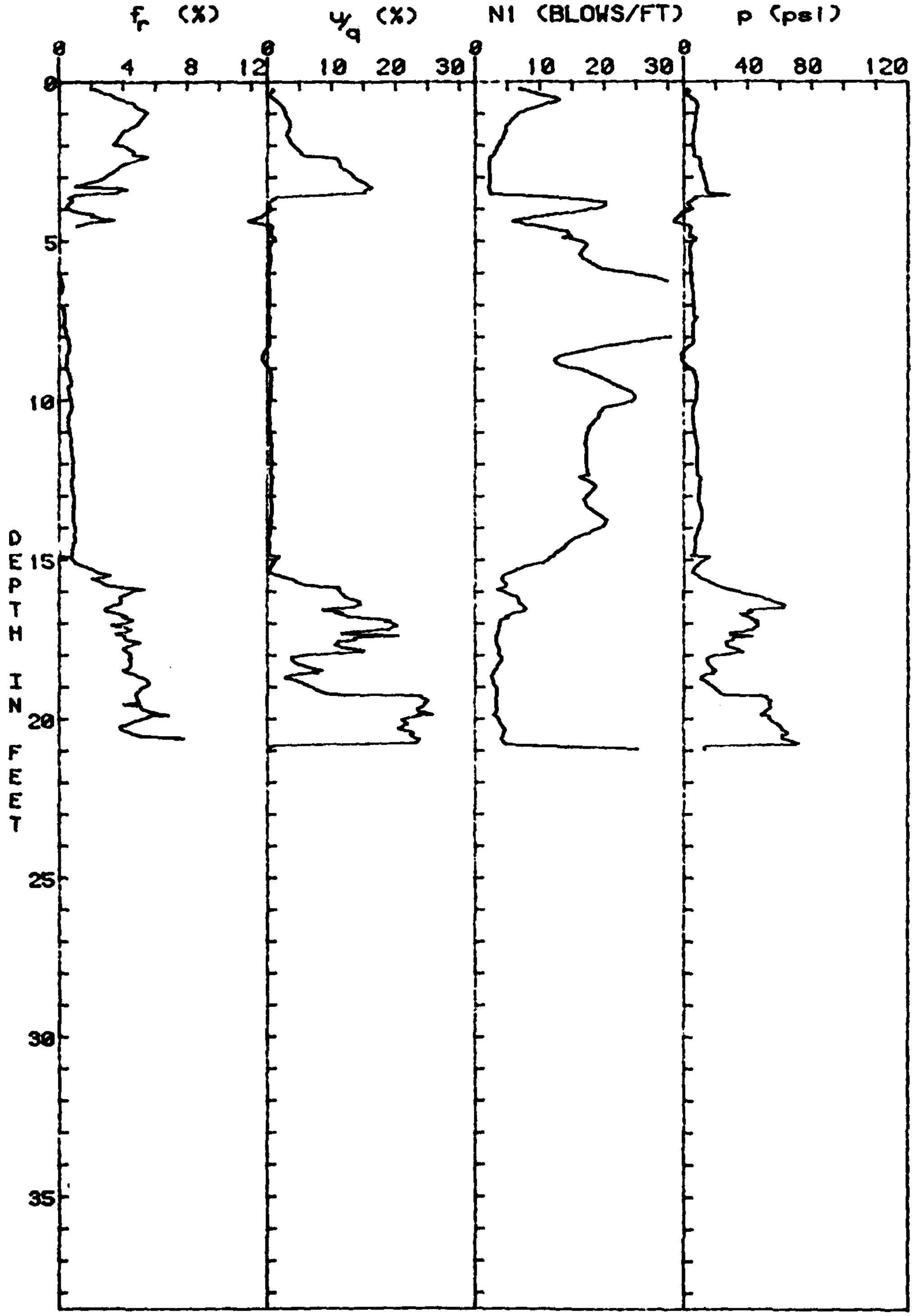

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 6
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 4 8 12
0 30 60 90
0 10 20 30
0 40 80 120
DEPTH IN FEET
0
5
10
15
20
25
30
35

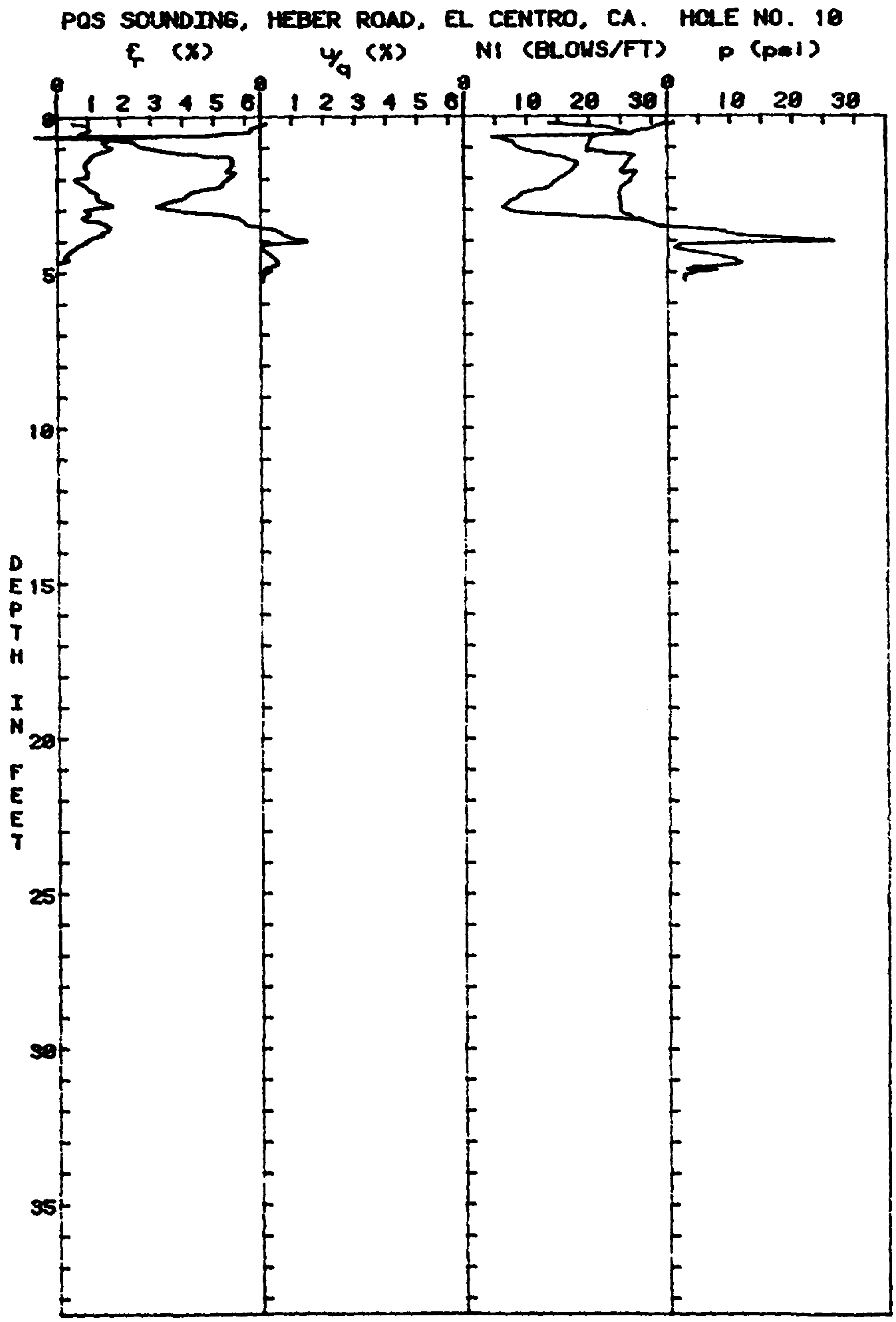


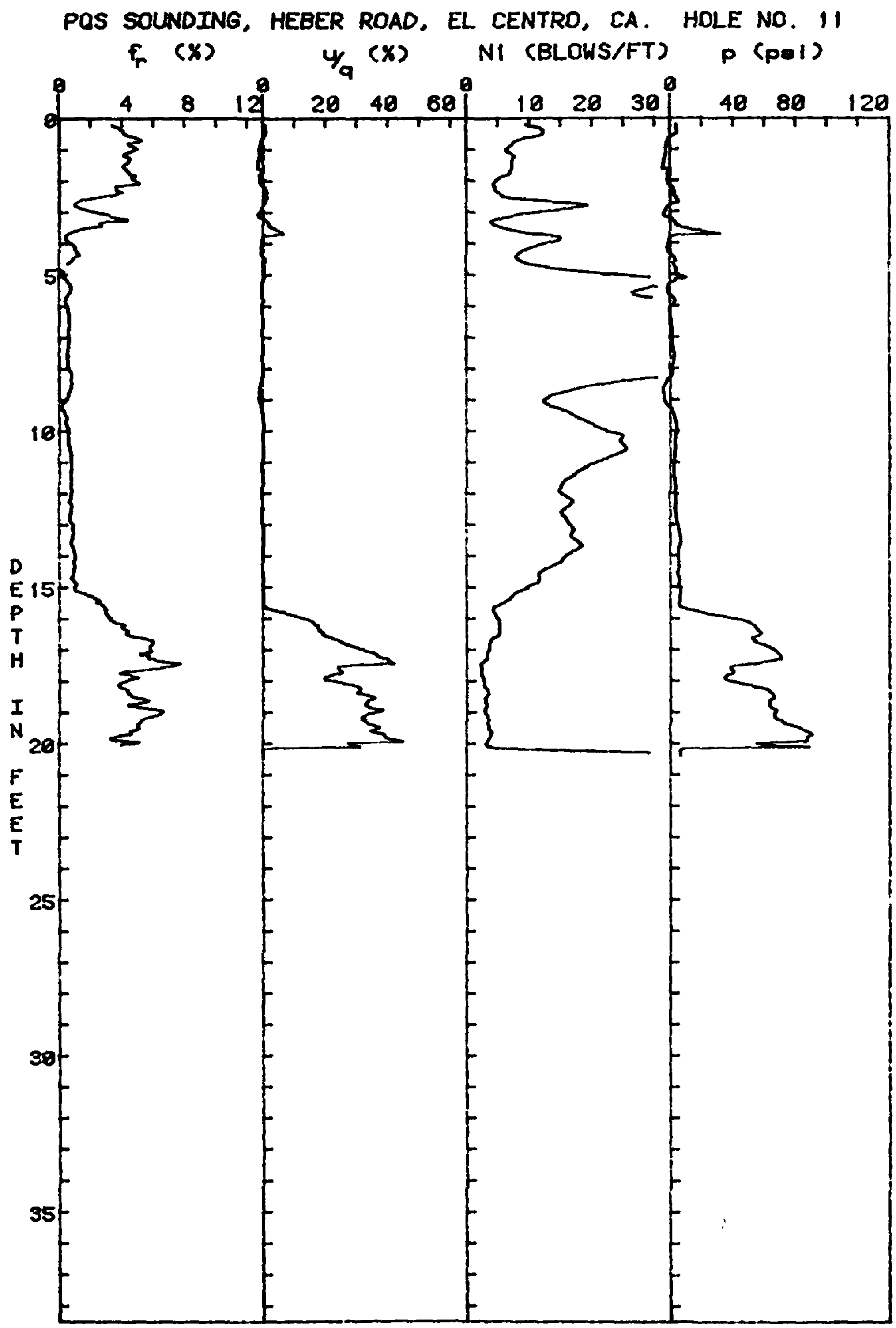
PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 7
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 8 16 24
0 20 40 60
0 10 20 30
0 10 20 30
0 5 10 15 20 25 30 35
DEPTH IN FEET


PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 8
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 4 8 12
0 20 40 60
0 10 20 30
0 40 80 120
DEPTH IN FEET
0
5
10
15
20
25
30
35

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 9



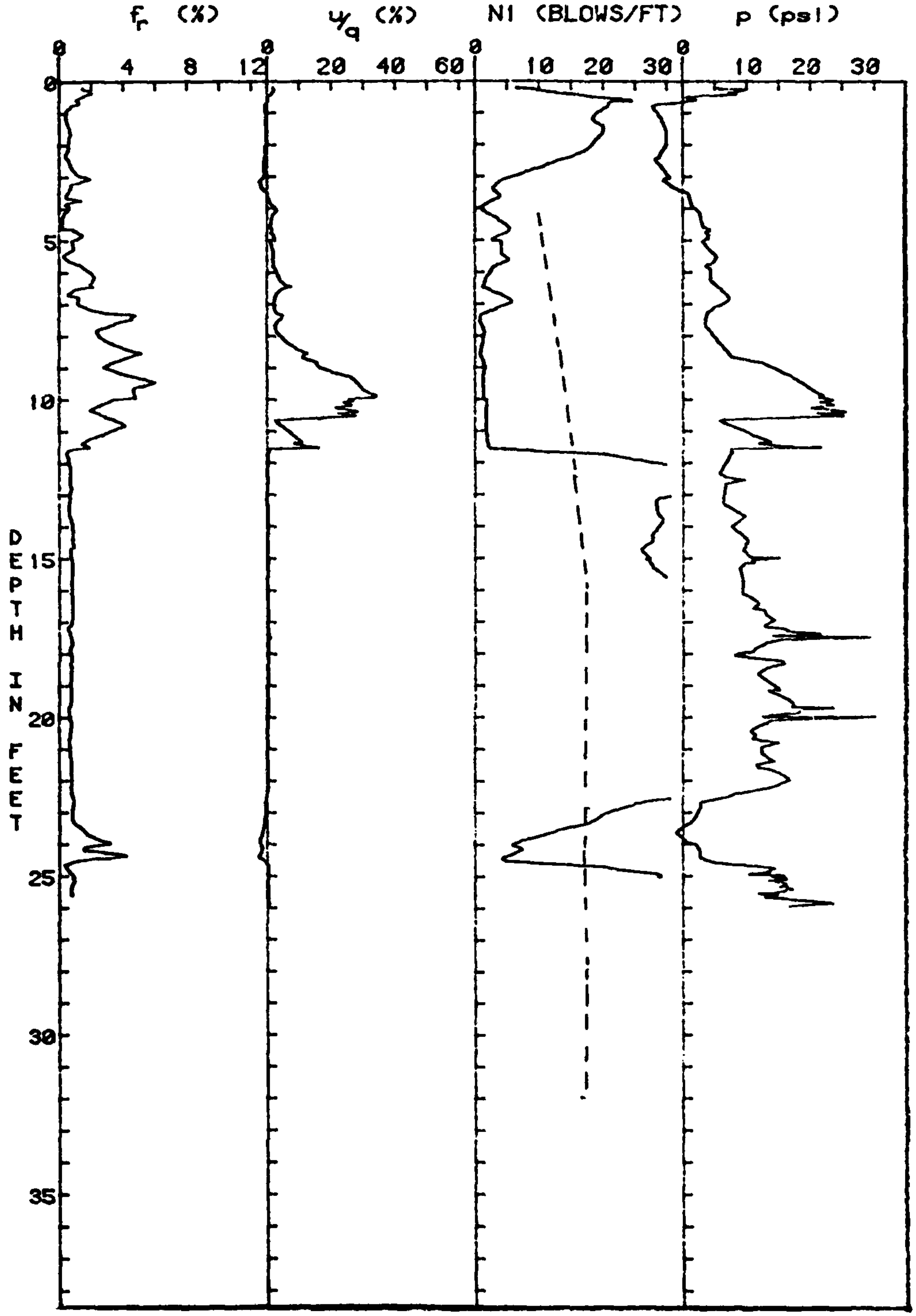

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 10
ξ_f (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 1 2 3 4 5 6
0 1 2 3 4 5 6
0 10 20 30
0 10 20 30
DEPTH IN FEET
0
5
10
15
20
25
30
35

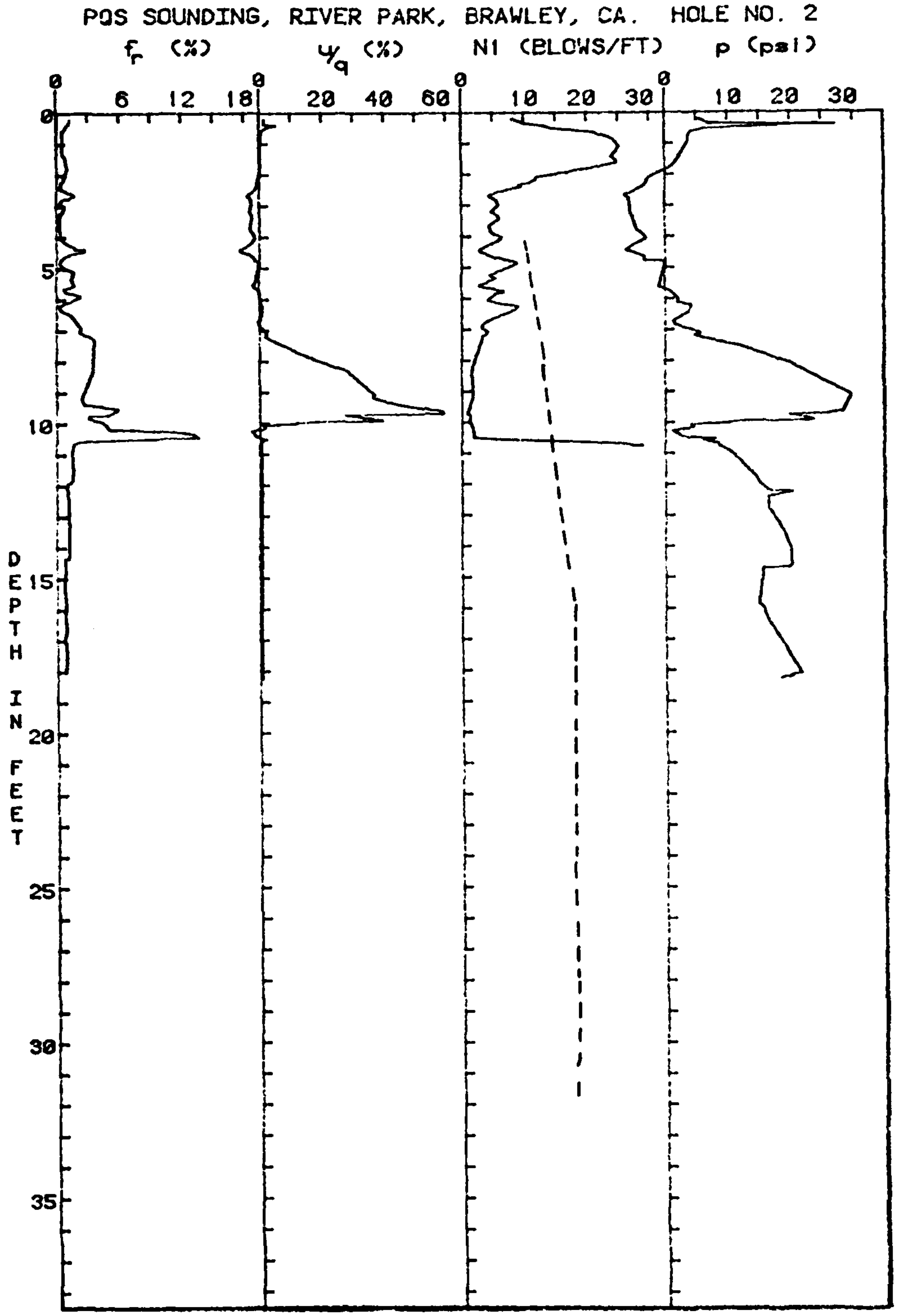

PQS SOUNDING, HEBER ROAD, EL CENTRO, CA. HOLE NO. 11
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 4 8 12
0 20 40 60
0 10 20 30
0 40 80 120
DEPTH IN FEET
0
5
10
15
20
25
30
35

APPENDIX F

PQS SOUNDINGS FOR RIVER PARK SITE

PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 1



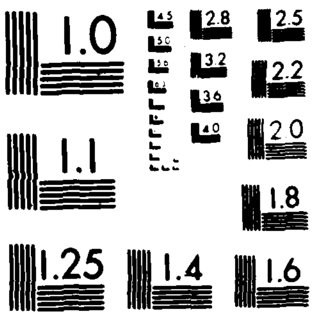


PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 2
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0
6
12
18
0
20
40
60
0
10
20
30
0
10
20
30
0
5
10
15
20
25
30
35
DEPTH IN FEET

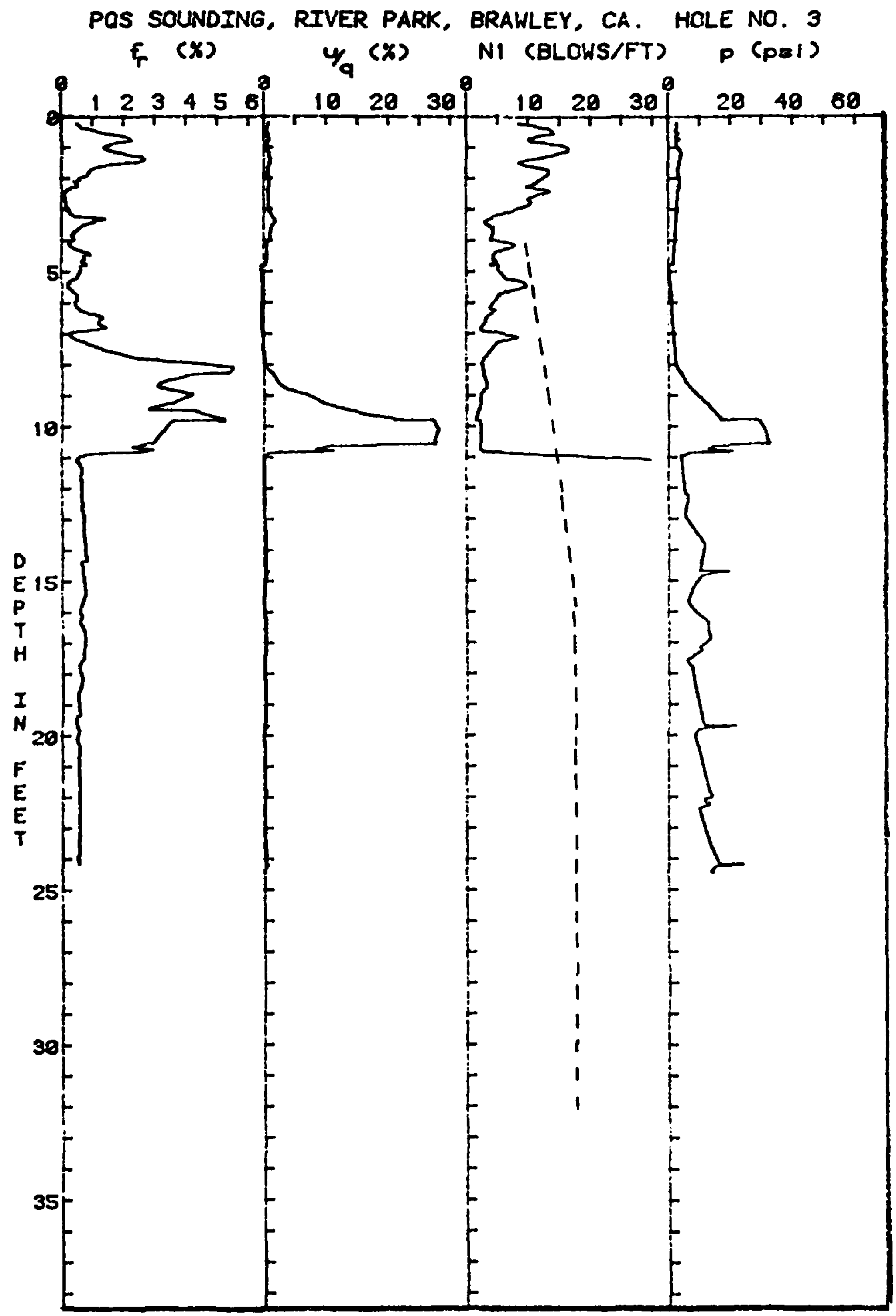
AD-A136 853 IN SITU DETERMINATION OF LIQUEFACTION POTENTIAL USING THE PQS PROBE(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS GEOTECHNICAL LAB W E NORTON 2/2

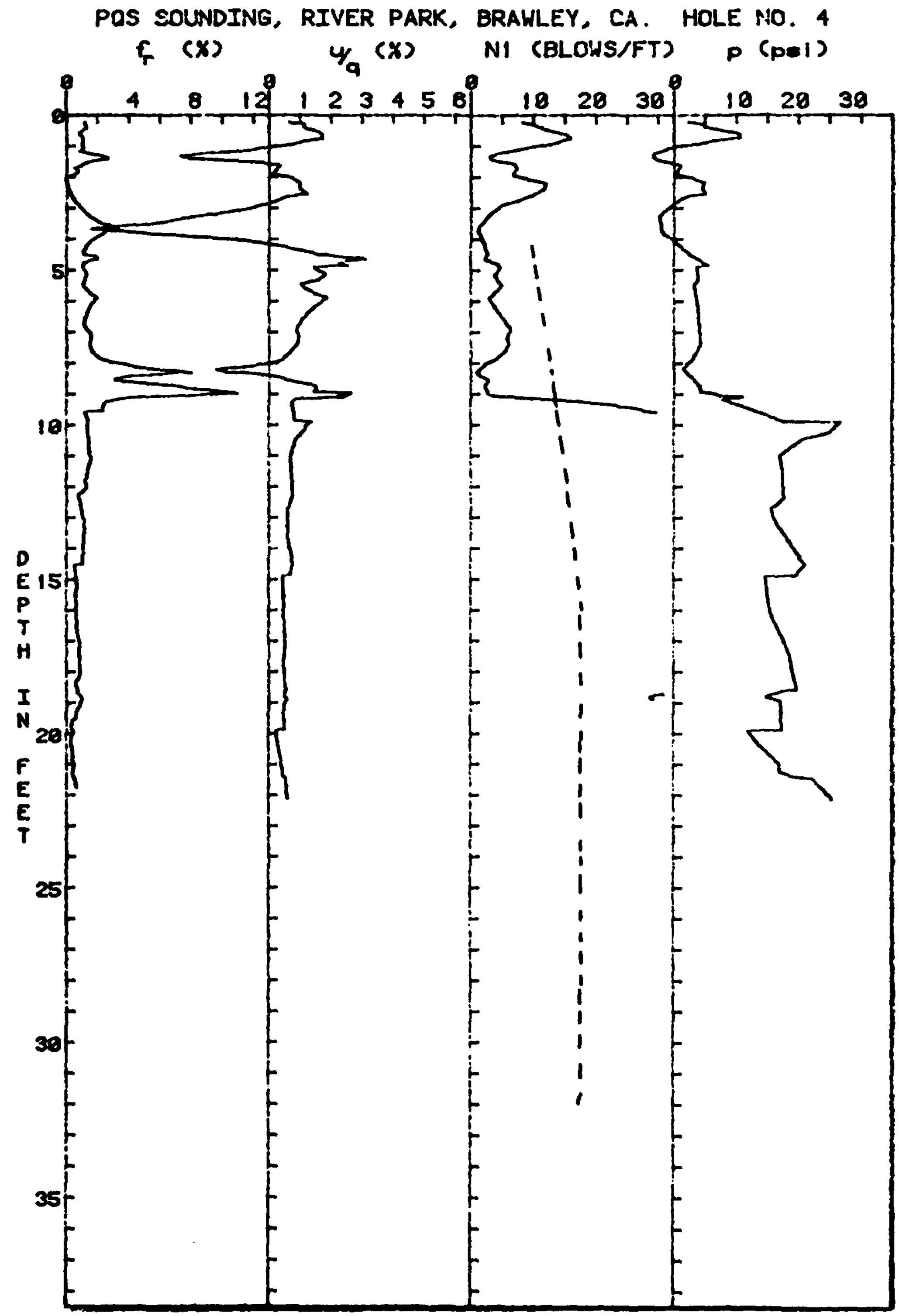
UNCLASSIFIED SEP 83 WES/TR/GL-83-15 F/G 8/13 NL

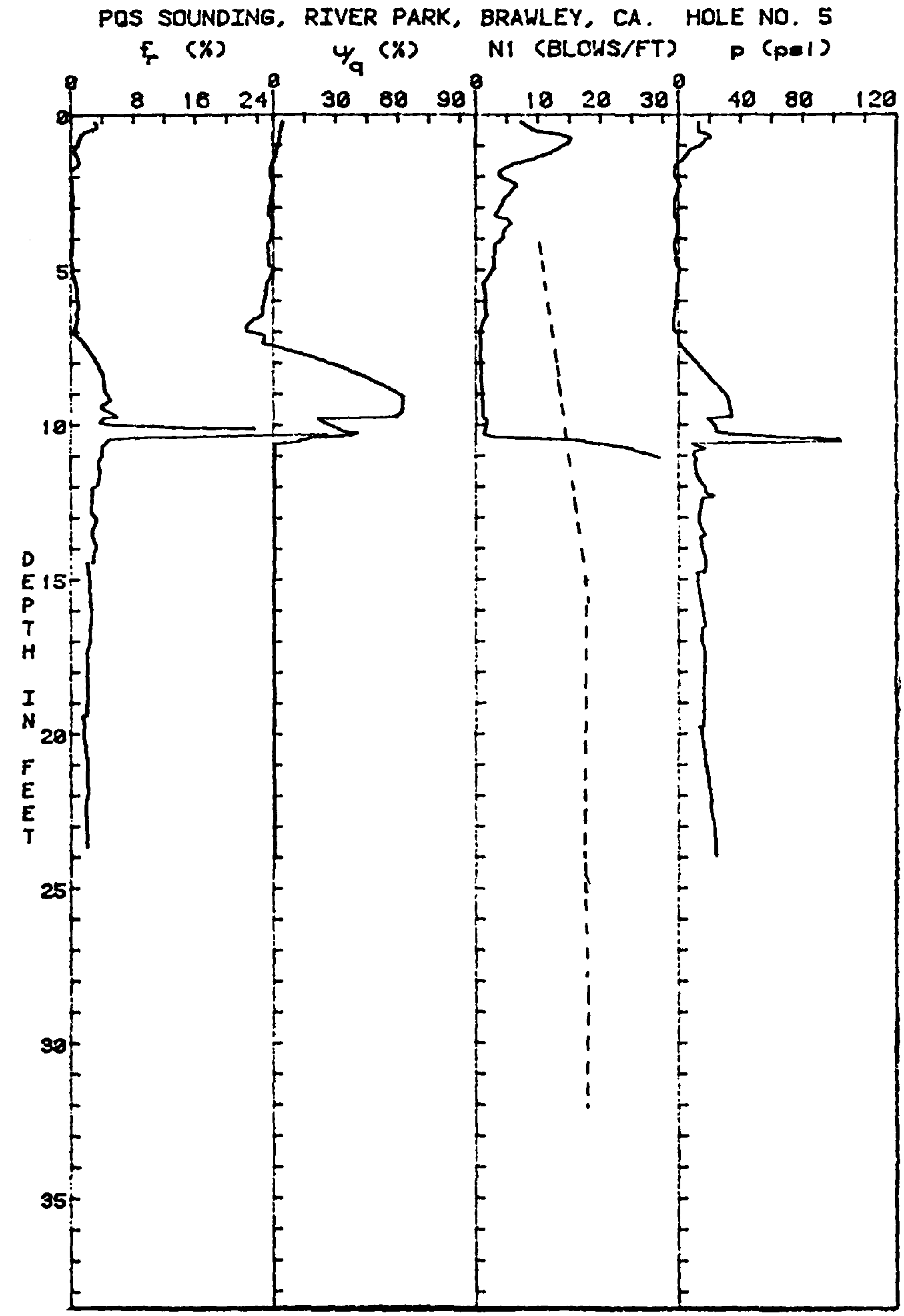
END
DATE
FILMED
2 84
DTIC

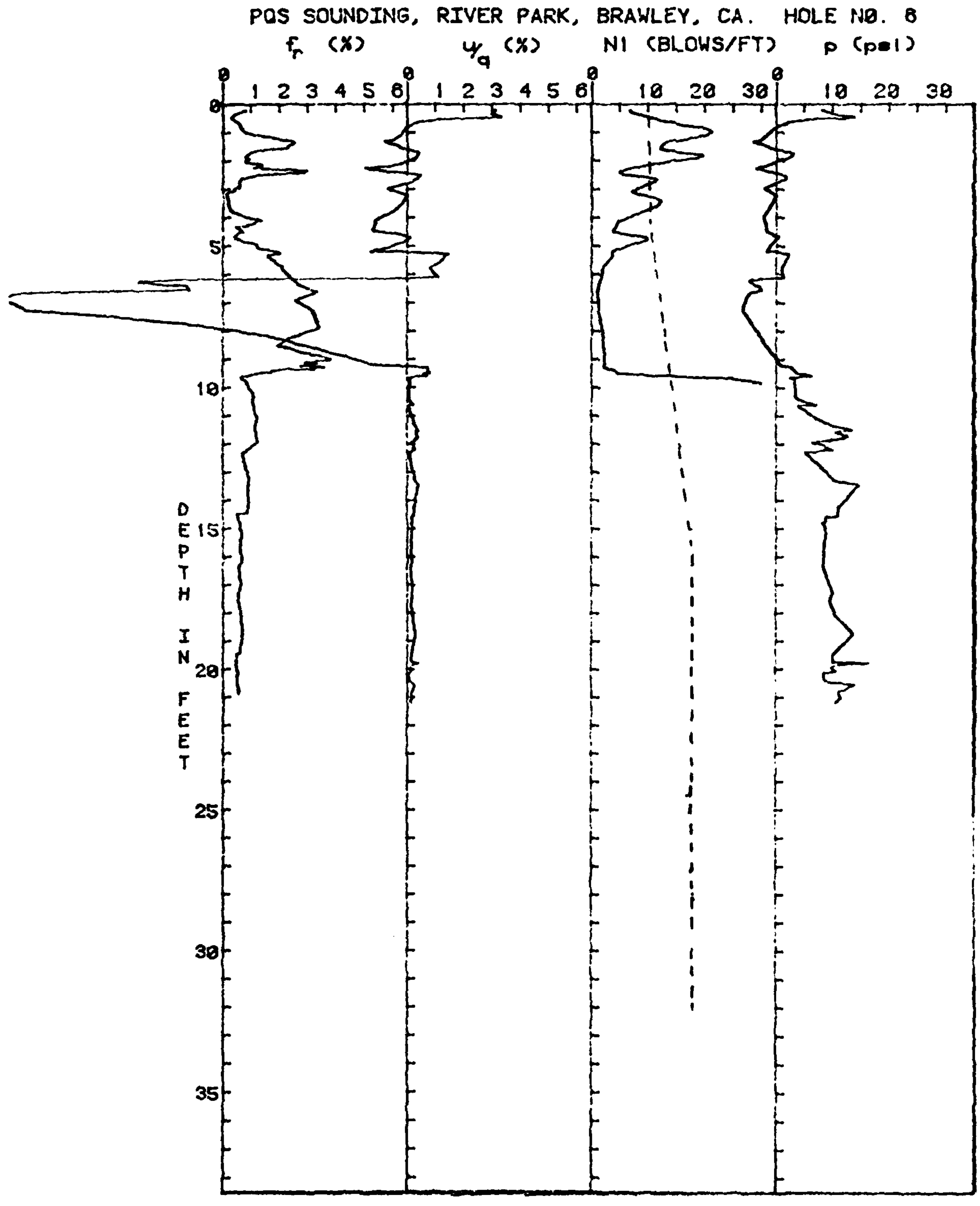


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS 1963-A

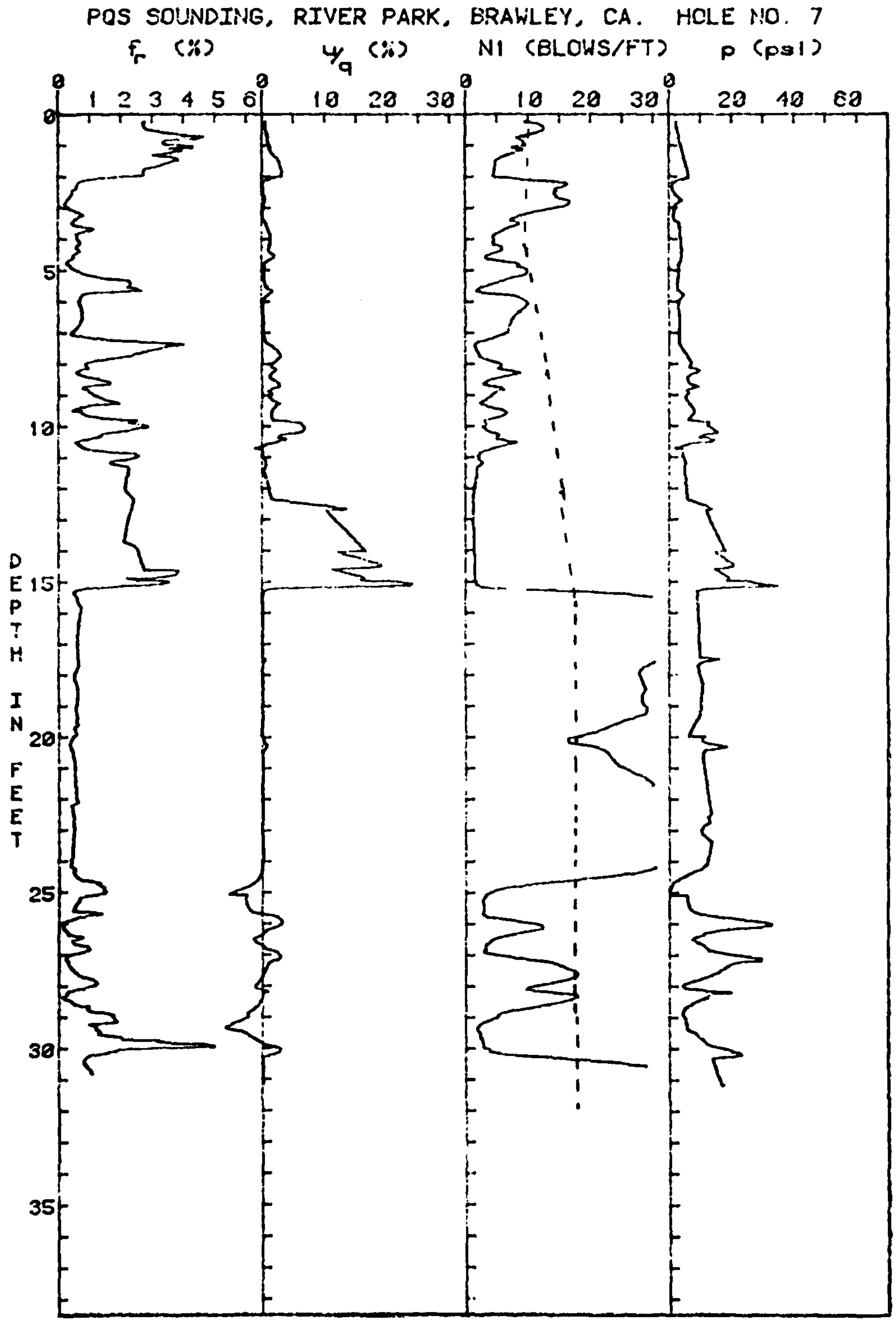

PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 3
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 1 2 3 4 5 6
0 10 20 30
0 10 20 30
0 20 40 60
0
5
10
15
20
25
30
35
DEPTH IN FEET


PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 4
f_r (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 4 8 12
0 1 2 3 4 5 6
0 10 20 30
0 10 20 30
DEPTH IN FEET
0
5
10
15
20
25
30
35


PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 5
fr (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 8 16 24
0 30 60 90
0 10 20 30
0 40 80 120
DEPTH IN FEET
0
5
10
15
20
25
30
35



PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 8
fr (%)
u/q (%)
N1 (BLOWS/FT)
p (psi)
0 1 2 3 4 5 6
0 1 2 3 4 5 6
0 10 20 30
0 10 20 30
DEPTH IN FEET
0
5
10
15
20
25
30
35


PQS SOUNDING, RIVER PARK, BRAWLEY, CA. HOLE NO. 7
f_r (%)
ψ_q (%)
N1 (BLOWS/FT)
p (psi)
0 1 2 3 4 5 6
0 10 20 30
0 10 20 30
0 20 40 60
0
5
10
15
20
25
30
35
DEPTH IN FEET