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S.B.P. John, A.Y. Lee, L.R. Leech, J. Campbell, and J.G. Macfarlane

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Introduction

This report presents the results of a ground water investigation along the semi-depressed portion of the proposed Hoffman Freeway (Route 17) in the City of Richmond (Figure 1).

The work reported herein was requested by Mr. Tom Walsh, of the District 04 Materials Department, on March 16, 1977, at a meeting in Sacramento attended by Mr. Walsh and Messrs. R.A. Forsyth, R.H. Prysock, and S.B.P. John of the Transportation Laboratory, Geotechnical Branch. At that meeting Mr. Walsh outlined the District's design concept for a semi-depressed portion of the proposed freeway.

Authorization to proceed with the work was given by Mr. D.T. Cassinelli of District 04 to Mr. G.A. Hill of the Transportation Laboratory in a memorandum dated March 18, 1977 (1).*

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Ground Water Investigation For The Richmond Semi-Depressed Section

04-CC-17

SEPT. 1978

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STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF CONSTRUCTION
OFFICE OF TRANSPORTATION LABORATORY

04-CC-17, PM 1.3 to 3.9
47th St. to 6th St.
04209-108701

Lab Auth 652907

September 1978

Mr. T. R. Lammers - 04
District Director of Transportation

Attention: Mr. D. T. Cassinelli
District Materials Engineer

Gentlemen:

Submitted for your consideration:

REPORT OF GROUND WATER INVESTIGATION FOR THE PROPOSED

RICHMOND SEMI-DEPRESSED SECTION 04-CC-17

Study made by . . . . Geotechnical Branch

Under the General Direction of . . . . Raymond A. Forsyth
Chief, Geotechnical Branch

Work Supervised by . . . . R. H. Prysock
S. B. P. John

Analysis and Report by . . . . S. B. P. John
A. Y. Lee
L. R. Leech
J. Campbell
J. G. Macfarlane

Very truly yours,

NEAL ANDERSEN
Chief, Office of Transportation Laboratory

SBPJ:lb/cj/bjs
Attachment

Distribution I

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INTRODUCTION

This report presents the results of a ground water investigation along the semi-depressed portion of the proposed Hoffman Freeway (Route 17) in the City of Richmond (Figure 1).

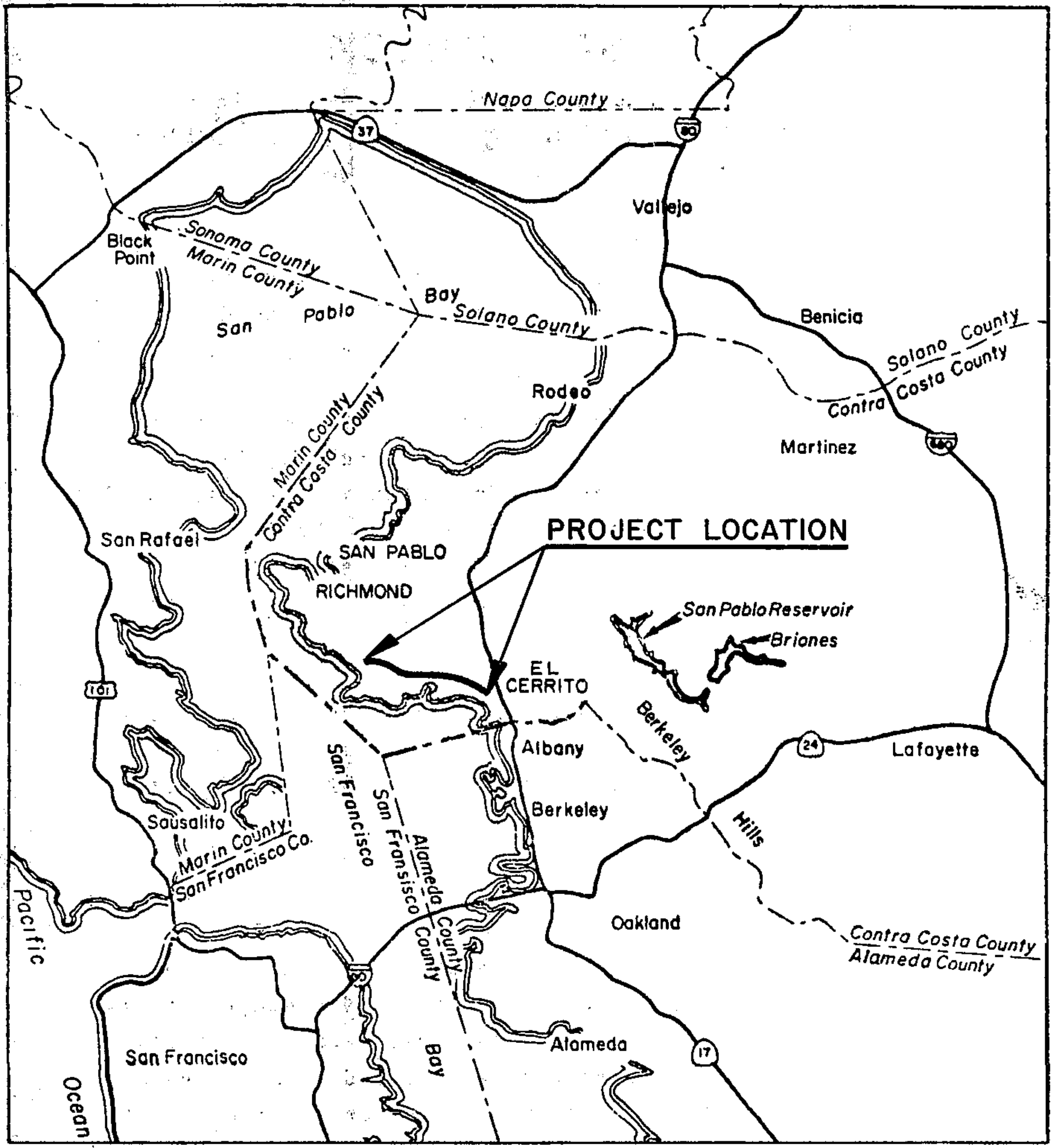
The work reported herein was requested by Mr. Tom Walsh, of the District 04 Materials Department, on March 16, 1977, at a meeting in Sacramento attended by Mr. Walsh and Messrs. R. A. Forsyth, R. H. Prysock, and S. B. P. John of the Transportation Laboratory, Geotechnical Branch. At that meeting Mr. Walsh outlined the District's design concept for a semi-depressed portion of the proposed freeway.

Authorization to proceed with the work was given by Mr. D. T. Cassinelli of District 04 to Mr. G. A. Hill of the Transportation Laboratory in a memorandum dated March 18, 1977 (1).*

Background Information

The design of Hoffman Freeway (Route CC-17) in the City of Richmond has been under study by District 04 since the early 1950's. Original design concepts included a rolling grade, primarily at ground level, rising and descending on approach embankments to structures over the various cross streets. A second design alternate consisted of an elevated roadway profile, supported the entire length (except for short structures over cross streets) by 25-foot high embankments (2, 3).

*Numbers thus shown denote cited references.



SOURCE: REFERENCE 6

VICINITY MAP

04-CC-17 PM 1.3-3.9

47TH ST TO 6TH ST

04209 108701

HOFFMAN FREEWAY, RICHMOND SEMI-DEPRESSED SECTION

DEWATERING STUDY

FIGURE 1

These alternates were received with some public disfavor because of potential noise problems and the physical barrier that would be created by the embankments. Consequently, a third design concept was developed which involved fully depressing a two-mile portion of the freeway to a depth of 25 to 30 feet. Surface streets would be carried by structure over the freeway at, or very near, their original grades.

To assess ground water problems associated with the fully depressed concept, a geotechnical study was initiated in 1970, with most of the work actually being performed in 1971 (4). That investigation included soil sample borings, laboratory tests, and four pump (drawdown) tests. The report concluded that an open cut depressed profile was feasible but that settlement outside the construction limits probably would result due to the removal of large quantities of water from an aquifer below a depth of 30 feet. Futhermore, considerable expense would result from providing the necessary permanent dewatering facilities.

The generally unfavorable findings from the 1971 study resulted in the proposal of a fourth design alternate: an elevated profile, about half of which would be supported by embankment, the other half by viaduct. In effect, this alternate was a modification of the earlier elevated concept. This alternate proved to be uneconomical as well as unpopular to the public; conditions that prompted further consideration by the District.

Since most of the problems associated with fully depressing the freeway were related to ground water at and below a depth of 30 feet, an alternate concept was proposed to semi-depress the freeway which would avoid ground water below 30 feet and reduce the overall ground water problems to a manageable level at reasonable costs.

Objectives of Investigation

Specific objectives of the study described herein were to:

1. Estimate the quantities of ground water that will be removed during construction and after construction.

2. Develop feasible concepts for removing ground water during construction and after construction.

3. Estimate the effects of dewatering, as required by the proposed facility, on:
   a. Ground subsidence outside the construction limits.
   b. Sea water intrusion.

4. Determine present ground water quality.

Scope of Investigation

The scope of the investigation was based on: (1), several field reviews and discussions with District personnel regarding the preliminary semi-depressed section concept; and (2), potential short term (construction phase) and long term (operation phase) ground water problems in relation to the proposed facility. The study included the following major elements:

1. A boring and sampling program to identify aquifers and determine ground water levels.

2. Drawdown tests to determine quantities of water to be pumped, and the area influenced by pumping.

3. A review of all available ground water literature pertinent to the area.

4. A detailed reconnaissance of the area to establish an approximate extent of present ground water use and number of existing wells.

5. Laboratory tests to determine the present quality of ground water in the area.

6. Appropriate analytical work to accomplish the listed objectives of the investigation.

PROJECT AND SITE DESCRIPTION

Project Location

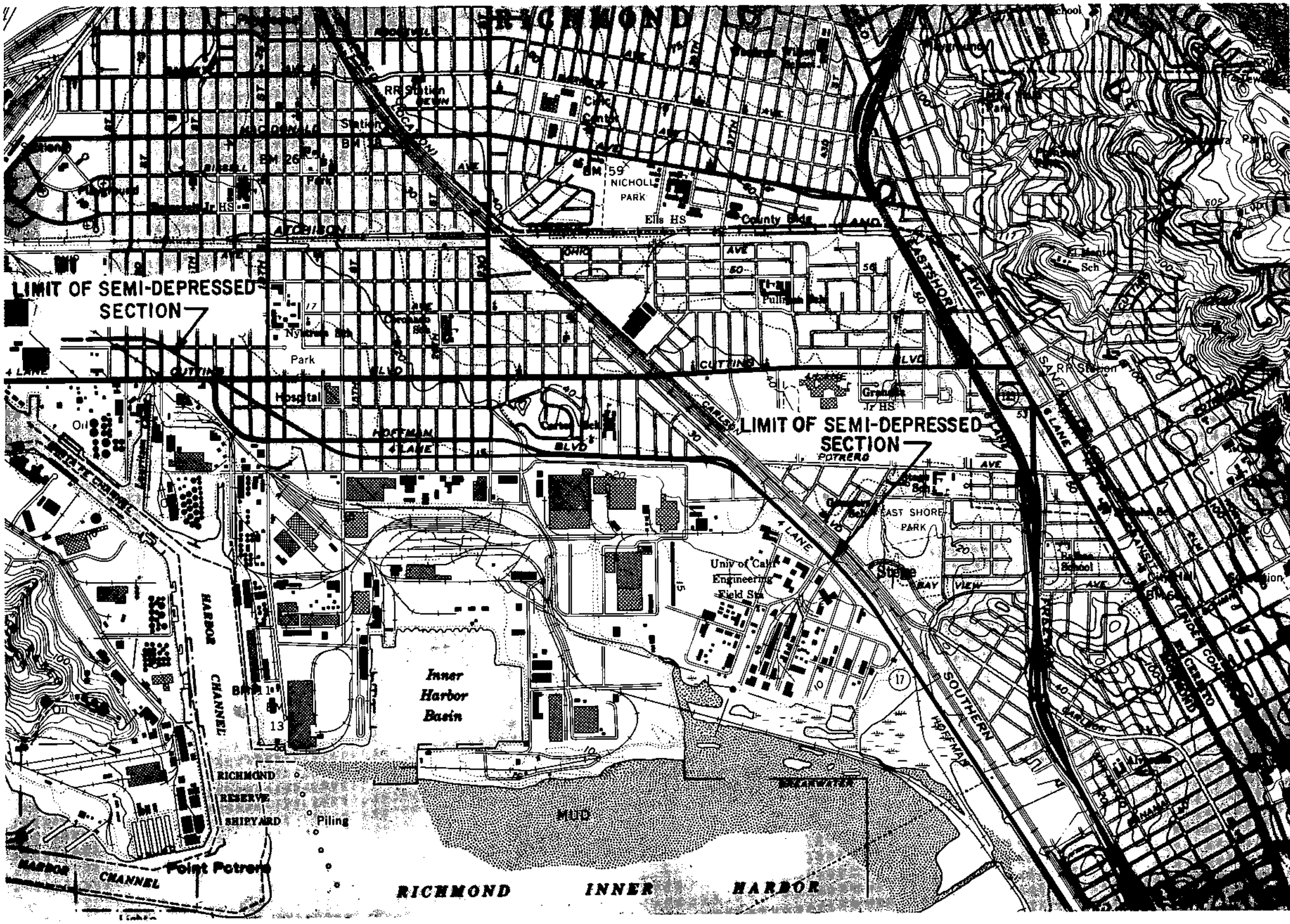
The proposed Route 17 (Hoffman) freeway will traverse the southwestern section of the City of Richmond along an alignment roughly parallel to the existing Route 17 (Figure 2).

This area is part of an alluvium-filled valley lying between San Francisco Bay and the Berkeley Hills to the east. The terrain is gently rolling and ranges from about 8 to 33 feet above mean sea level in the project area. This portion of the city is devoted to light industrial and residential use.

The Hoffman Freeway will cross the southerly end of the valley in an east-west direction, about 1/2 mile north of Richmond Inner Harbor. The entire freeway will extend from El Cerrito to Marine Street in Richmond. The portion between Stations 170 and 276 in Richmond (about 2 miles) will be semi-depressed, with cuts ranging from 10 to 26 feet. Recent boring data indicate that depth to ground water ranges from 5 to 19 feet. Annual rainfall in the immediate area is about 19 inches.

Site Geology

The terrain in the Richmond area consists of a complex, non-homogeneous sequence of sedimentary, igneous, and metamorphic rocks ranging in age from Late Jurassic to Quaternary. The basement rocks for the region consist of the relatively impermeable Jurassic-Cretaceous Age Franciscan Group, composed



SOURCE: REFERENCE 6

HOFFMAN FREEWAY
RICHMOND SEMI-DEPRESSED SECTION
DEWATERING STUDY

SITE LOCATION MAP
FIGURE 2

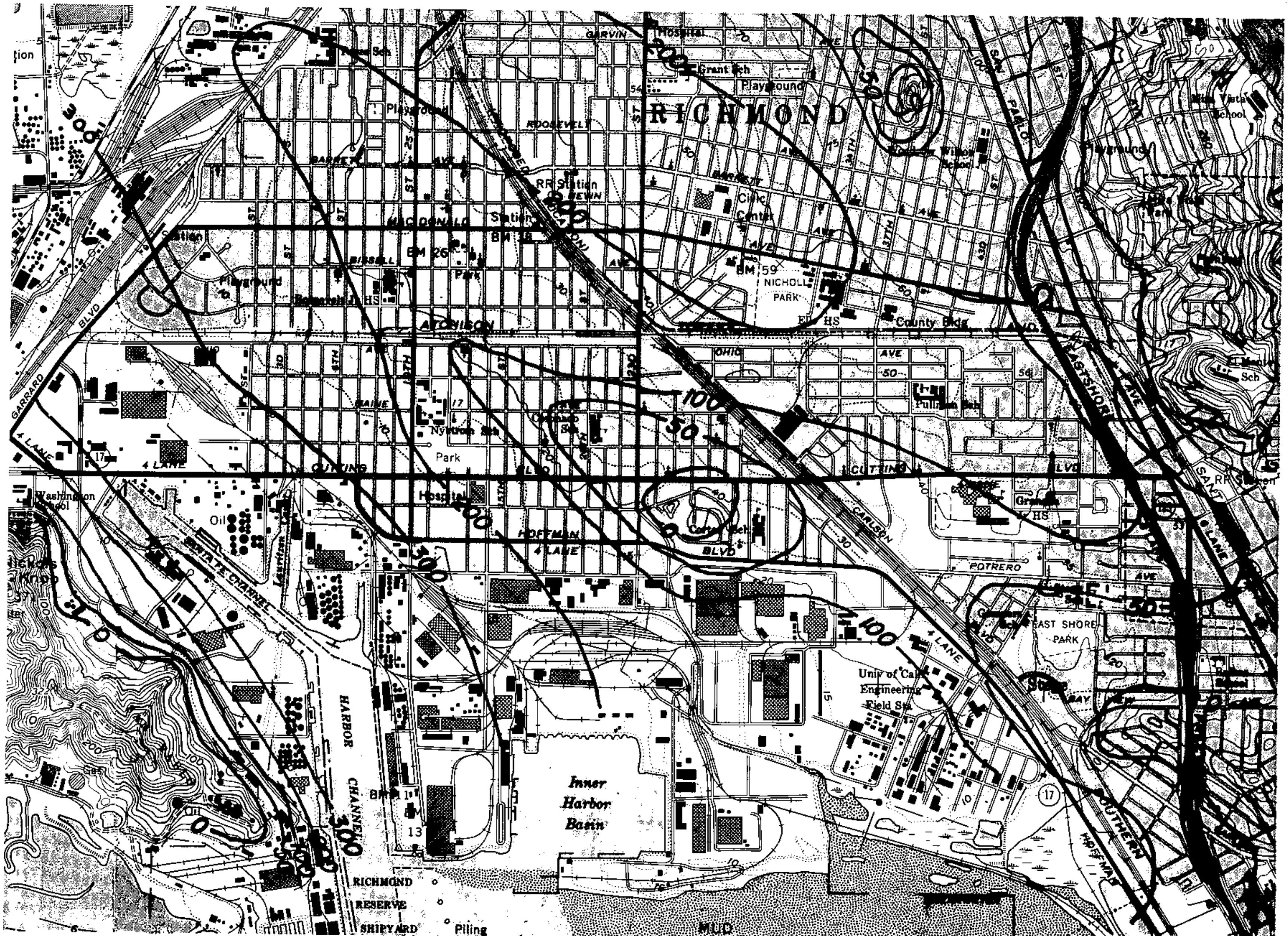
of shales, sandstones, mudstones, serpentines, and greenstones. In the valley the basement is overlain by marine and non-marine sedimentary and volcanic rocks above which lie the Quaternary deposits which consist of alluvium and bay mud, Figure 3.

The alluvium is derived principally from the Contra Costa rocks of the Berkeley Hills to the east. The clay-rich soil that develops in these rocks is largely of the montmorillonite type and is highly expansive in character. The alluvium also contains silts, sands, and gravels. The bay muds are clay and silt mixtures with occasional sand layers. The thickness of the alluvial soils in the project area, or the depth to bedrock, is about 50 feet at the easterly end and some 300 feet at the westerly terminus. Site geology is discussed in more detail under "Subsurface Conditions".

The Semi-Depressed Concept

The semi-depressed profile will be approximately two miles in length between 37th and 6th Streets (Station 170-276). Nominal depth of excavation will vary from 10 to 15 feet. The maximum depth of excavation will be about 26 feet at the easterly end of the project, in the vicinity of 37th Street (Station 180±). This deeper excavation will provide for an undercrossing of the joint Southern Pacific/Santa Fe Railroads. It will also be necessary to raise the railroad grade to accommodate this proposed design.

Normal bottom width of the excavation could be as little as 122 feet, or as much 178 feet, depending upon the geometric design alternate ultimately selected. Nominal additional widths will be required to accommodate interchange ramps. Side slope ratios of 2:1 are planned except for those areas where retaining walls will be employed.



SOURCE: REFERENCE 6

HOFFMAN FREEWAY
SEMI-DEPRESSED SECTION

ALLUVIAL THICKNESS MAP

FIGURE 3

Three interchanges will be provided for all geometric alternates except the High Occupancy Vehicle (HOV) lane alternate. The three planned interchanges will be located at 37th Street, 23rd Street, and Harbor Way/Cutting Blvd. (10th Street). In the event the HOV lane alternate is adopted, a fourth interchange would be located at Marina Way (14th Street).

FIELD INVESTIGATION

The field investigation along the project route followed several lines of inquiry in pursuing the objectives of this study. Work in the field fell generally into the following four categories:

1. A program of boring and sampling to evaluate subsurface soil conditions and establish the location and extent of aquifers, primarily within the uppermost 30 feet of the soils.

2. Drawdown tests were conducted to permit calculation of aquifer constants for the pervious layers and to evaluate recharge potential.

3. The depth to ground water in all open borings along the project was measured to establish present (1978) levels. Fluctuation was assessed on the basis of previouly recorded readings. Future measurements will be scheduled for further comparison.

4. A reconnaissance survey of private and publicly owned water wells within, or proximate to, the project limits was conducted to gather general information with regard to well capacities, water quality, and usage.

Boring and Sampling

During 1977 the District 04 Materials Department made 26 borings along the project to delineate types and geotechnical characteristics of soils within the proposed excavation area. The borings ranged from 30 to 80 feet in depth. Some of the borings were augered (6 inches and 18 inches in diameter) and others provided 2-inch diameter jar and liner samples. Only selective undisturbed sampling was done as the study was oriented primarily to visual identification of soils and location of water-bearing strata.

The information presented by the 1977 series of borings augmented the data obtained from borings made for the 1970-71 study by Districts 04 and 10 and the Office of Structures. Details of all the 48 borings made on the project to date are listed in Table 1 and shown graphically on the boring profiles presented as Appendix A of this report. The locations of the various borings are plotted on Figures 4 and 5.

TABLE 1, DRILLING DATA

Series	Year Drilled	No. of Borings Used	Depth Range	Drill Crew	Type	Sample	Comments
P	1977	15	30-50	Dist 04	Auger (6"&18")	Jar & Sack	Gas Odor in P8
D	1977	11	30-80	Dist 04	2" Sampler	Jar & Tube	Undisturbed samples taken
B	1971	6	70-75	Dist 10	2" Sampler	Jar & Tube	"
P	1970	5	31-35	Dist 04	18" Auger	Sack	
Br	1970	11	60-136	Bridge	1.4" Ø SPT	Jar	

As the complexity of the subsurface soil stratification precluded development of a single generalized soil profile for analytical purposes, it was found expedient to divide the length of the proposed excavation into six areas in order to facilitate this study. The area designations and lineal extent are listed below.

Area	Stationing	Approximate Street Limits
1	170-184	45th St. to So. 8th St.
2	184-205	So. 8th St. to So. 31st St.
3	205-231	So. 31st St. to So. 21st St.
4	231-252	So. 21st St. to So. 14th St.
5	252-273	So. 14th St. to So. 7th St.
6	273-276	So. 7th St. to So. 6th St.

Drawdown Tests

The selection of suitable locations for drawdown tests followed a detailed visual inspection of continuous samples recovered from the borings. The two sites chosen were located at the intersections of 11th Street and 30th Street with Hoffman Boulevard. Since the four pump sites utilized in the 1971 drawdown studies had all been located on the bayward side of the then proposed fully-depressed freeway section, it was decided to locate the two sites for the current study on the landward side of the presently proposed semi-depressed section. These latter locations, identified as PT-1 and PT-2, and the 1971 pump test sites denoted as R-1, R-2, R-3, and R-4, are shown in Figure 6.

The 1971 series of pump tests was oriented to tap aquifers at depths below 30 feet. By contrast, the 1977 pump tests tapped aquifers at, or above a depth of 30 feet.

[illegible]

Preliminary Pump Tests

Preliminary pump tests of short duration were conducted by District 04 personnel at each site to enable estimations of volumes of water that would be encountered during subsequent full-scale pump tests. A pump well and four observation borings were installed at each of the two sites for the preliminary testing.

On December 16, 1977, District 04 personnel used a Proesser 1-HP submersible pump at Site 1, (11th Street and Hoffman Blvd.) for a period of about seven minutes, at an initial discharge rate of 24 gpm until the ground water was drawn down to 21.6 feet below ground surface. The discharge rate was then ajdusted to 3 gpm by partly closing the valve on the discharge line. This rate of discharge held the water level constant at 21.6 feet below the ground surface for the balance of the test, which was terminated after 4 hours.

Rebound, or ground water recovery readings were then taken for a period of 2 hours. This procedure was repeated on December 19, 1977. The equipment was then moved to the 30th Street site, and a similar procedure was carried out at that location on the 20th and 21st of December.

Full Scale Pump Tests

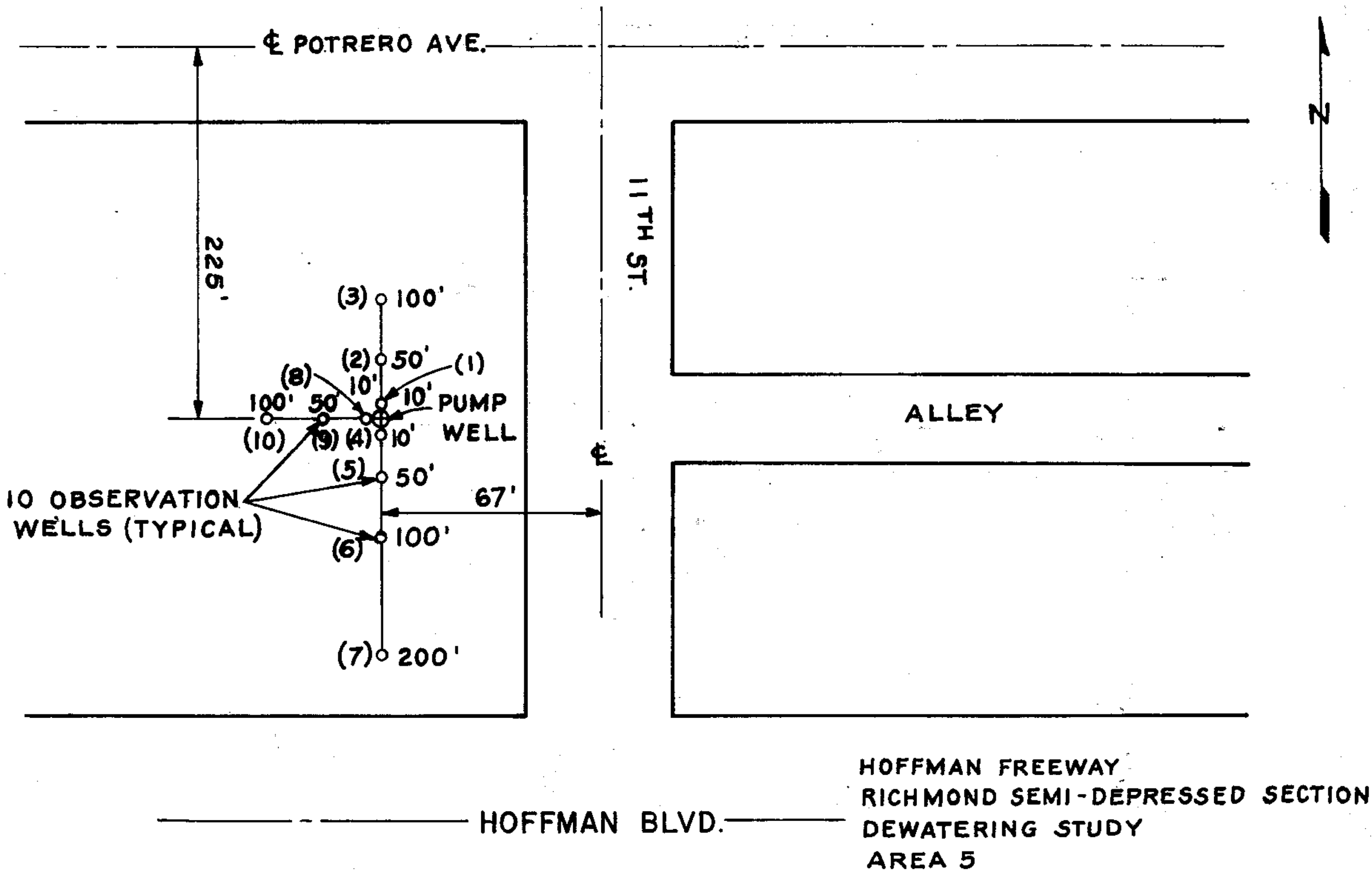
Following evaluation of preliminary drawdowns full scale pumping tests were scheduled. To prepare the sites for these operations District personnel installed additional observation wells as described below.

For Pump Test 1, three arms were laid out and observation wells located at 10-, 50-, and 100-ft. intervals along the north and west arms; and at 10-, 50-, 100-, and 200-ft. spacings on the south arm. The pump test site is shown in Figure 7. The maximum depth of observation wells was 25 feet, whereas the pump well was drilled to 25.5 feet. The pump was set at a depth of 21 feet.

For Pump Test 2, two arms were laid out, with observation wells at 10-, 50-, 100-, and 150-ft. intervals along the north arm, and at 10-, 50-, 100-, and 200-ft. for the east arm. The maximum depth of observation wells was 20 feet and the pump well was drilled to 22.5 feet. This layout is depicted in Figure 8. The pump was set at a depth of 17 feet. In both pump tests the casing used for observation wells consisted of 2-inch diameter PVC pipes, whereas for pump wells the casing was of 8-inch diameter, perforated/slotted steel pipe. All pump wells and observation wells were cased top to bottom.

During the first week of January heavy rains saturated the project area, inundating the ground surface adjacent to some of the observation wells. This surface water infiltrated and in some cases filled the borings. Consequently, it was necessary to dewater the wells to natural ground water levels prior to beginning tests.

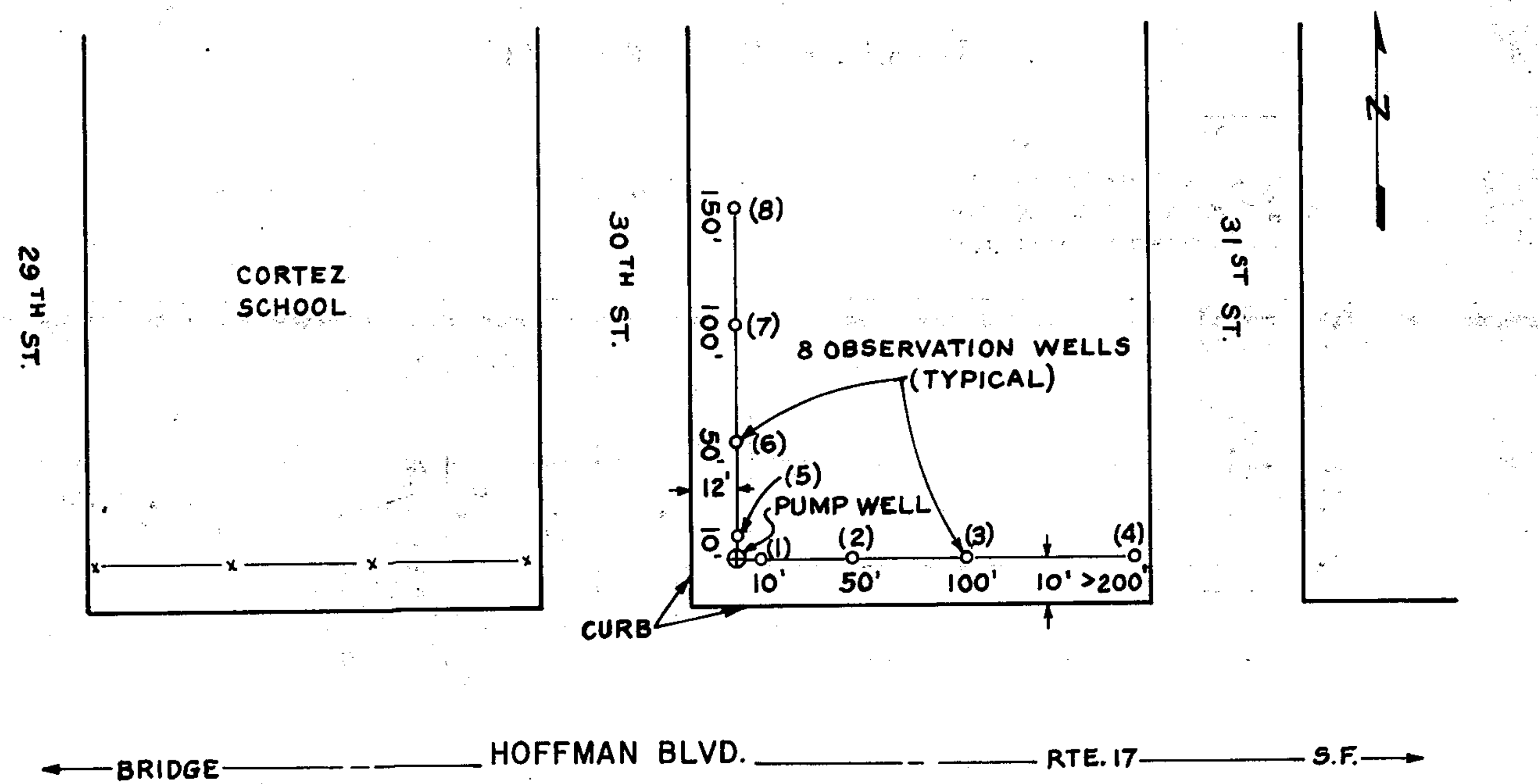
The pump test at the 11th Street site (Pump Test 1) was started at 1030 hours on January 17th, and was terminated at 1100 hours on the 18th when the water level in the pump well reached the desired depth of 21.0 feet below ground surface. Recovery rates were then monitored for four hours, after which the test was concluded.



SITE PLAN, PUMP TEST 1

NOT TO SCALE

FIGURE 7



HOFFMAN FREEWAY
RICHMOND SEMI-DEPRESSED SECTION
DEWATERING STUDY
AREA 2

SITE PLAN, PUMP TEST 2

NOT TO SCALE

FIGURE 8

A wind and rain storm delayed the start of Pump Test 2 at the 30th Street site until January 23rd. The procedure previously described was followed at that location but the duration of the test was considerably shorter. At a discharge rate of 3 gpm the pump well was completely drained in 3-1/2 hours and recovery readings were begun immediately and continued for 4 hours.

Water Level Measurements

Ground water level readings were made in all open borings within the project limits in July 1978, and compared to water level measurements recorded in mid-October of the previous year. These readings are posted on Table 2.

A review of the readings to date on an area-by-area basis permitted the general conclusions shown in Table 3 with respect to fluctuation during the period of observation.

TABLE 3 FLUCTUATION OF G.W.T.

Area	Status
1	Risen
2	Risen
3	No Change
4	No Change
5	No Change
6	No Change

The rise in ground water table may have been due to higher than normal recharge or a reduced demand on the ground water system. Where no change is indicated, a number of factors may have been responsible.

TABLE 2, GROUND WATER LEVELS

Boring No.	Station	Area	Date of Boring	Depths to GWT (feet)		
				On D.O.B.*	10/17/77	7/19/78
D9-77	166+70	1	9/3/77		13.6	12.4
P12-77	166+85	1	8/27/77		14.6	13.9
D17-77	172+35	1	10/11/77		12.85	11.9 (7/31)
P1-70	176+45	1				
P11-77	177+00	1	8/27/77		17.6	16.0
B3-71	177+35	1				
D18-77	178+50	1	10/13/77		16.9	14.4 (7/31/78)
D16-77	181+20	1	10/7/77		14.05	11.9
D11-77	187+50	2	9/20/77		6.55	-
P10-77	187+80	2	8/27/77		14.0	14.0
P9-77	201+00	2	8/27/77	<10.0	7.65	8.4
P4-70	202+70	2				
D10-77	202+90	2	9/8/77	6.3	6.05	-
P8-77	208+00	3	8/26/77	<9.0	9.3	Dry?
B1-71	219+60	3				
P5-70	219+80	3				
P7-77	220+10	3	8/25/77	<15.4	7.9	21.6
P6-77	228+50	3	8/25/77	<16.0	7.95	6.9
D12-77	236+75	4	10/3/77	8.0	8.05	-
P5-77	239+40	4	8/25/77	<22.6	7.65	Dry?
D8-77	239+60	4	9/1/77		8.9	-

*On D.O.B. = on date of boring completion; ground water depth.

TABLE 2, GROUND WATER LEVELS (continued)

Boring No.	Station	Area	Date of Boring	Depths to GWT (feet)		
				On D.O.B.*	10/17/77	7/19/78
P2-70	246+10	4				
P4-77	249+80	4	8/25/77	<6.5	6.1	6.0
D13-77	257+45	5	10/4/77		4.8	-
P3-77	259+00	5	8/24/77	<9.0	6.6	6.7
B4-71	259+50	5				
D14-77	264+20	5	10/5/77		6.2	-
P2-77	272+00	6	8/24/77	<9.0	8.7	8.7
D15-77	272+00	6	10/6/77		8.9	8.7
D7-77	272+20	6	8/30/77		8.2	8.7
P13-77	274+15	6	8/27/77	<7.7	8.1	7.7
P14-77	276+00	6	8/29/77		7.6	-
P15-77	276+00	6	8/29/77	<14.5	7.45	-
P1-77	278+50	-	8/23/77	<7.0	6.4	-
B2-71	287+25	-				
P3-70	294+10	-				

Note: *On D.O.B. = on date of boring completion; ground water depth.

GWT = Ground Water Table

Depths to water are from existing ground surfaces.

Ground Water Reconnaissance Survey

The ground water reconnaissance survey was comprised of field visits, telephone conversations, interviews with officials of both private and public agencies, and a literature search. The purpose was to gather data on existing wells, ground water quality, and other pertinent aspects related to this study.

Personnel of the Transportation Laboratory, Sacramento, conducted this survey. The agencies contacted are listed in Appendix B. Information obtained during the survey is summarized below, along with pertinent tables and figures.

Mr. Alan Neumier, City Engineer, Richmond Public Works Department, was interviewed to inquire about City-owned wells and the surface drainage ditch that crosses Hoffman Boulevard at about Station 180, A-line. Mr. Neumier stated that there are several City-owned wells, used mainly for landscape irrigation. Potable water is provided by the East Bay Municipal Utility District.

A well located at Richmond's Municipal Plunge was originally drilled as an oil well, to a depth of 2,000 feet, and later developed for fresh-water supply. This well is now inoperative and the pump has been removed. Mr. Walter Smith, Richmond P.W.D. Director, recalled that it had been possible to pump this well at a rate of 100 g.p.m. for 24 hours continuously and not lower the water level.

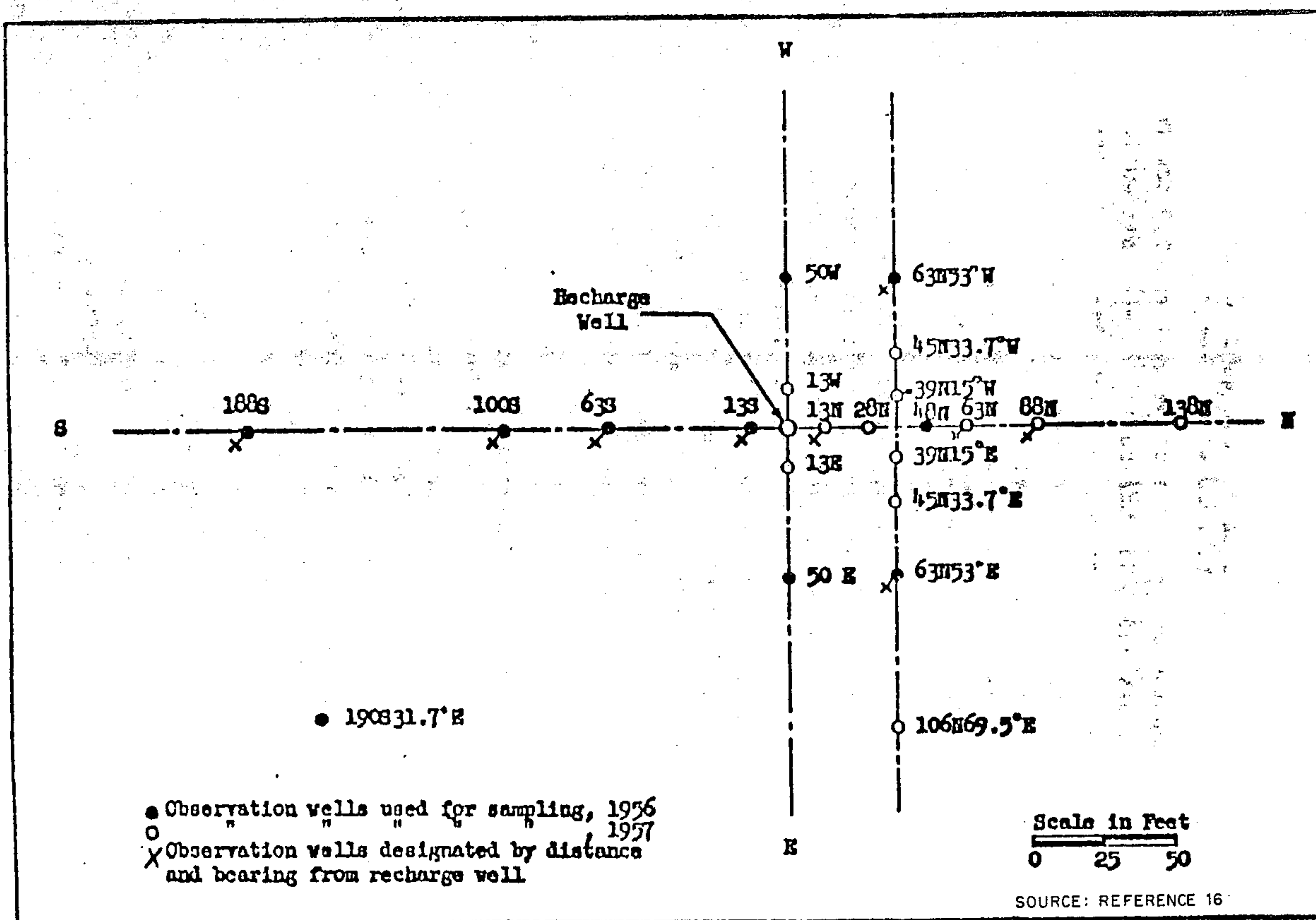
The interviewer was informed that several other City wells drilled more recently are open. However, no specific data have been obtained regarding them as City personnel were not available to assist in locating them at the time of this survey.

Mr. Neumier stated that the surface drainage ditch at Station 189 was improved by contract work in 1971. This open channel has an 8-ft wide bottom and 1:1 side slopes. The 6-ft diameter concrete culvert pipes under Hoffman Boulevard were placed in the mid-50's. The flow line elevation of the pipe is 1.66 ft above sea level at the discharge end near the bay, and about 6.5 ft at its intersection with Seaver Street. The pipe gradient is .003%. The storm water run-off rate from 1580 acres and from the East Shore Freeway is some 745 cfs.

Mr. Bert Lods, Chief of Maintenance, Richmond Unified School District, was contacted regarding school wells in Richmond. He stated that the closest school wells to Hoffman Freeway would be those of the Richmond High School. There are two 180-ft deep wells which furnish water for landscape irrigation. The water is fresh but "very hard," and large quantities are available, although totals have not been estimated. Mr. Lods, a life long resident of the Richmond area, has never heard of any local wells having a salt-water problem.

Mr. John R. Shively, Manager, Technical Services of the Richmond Field Station, University of California (U.C.), and Donald V. Hamblin, Physical Plant Superintendent, were contacted on July 26, concerning drilled wells on U.C. Field Station property. There are three separate well sites on the grounds on an approximate North-South straight line, which, if extended to the Hoffman Freeway alignment, would pass close to the project Boring Number P11-77.

There are single wells at two of the sites. The third site is a "well-field", consisting of 22 wells which have been used in various research programs since 1956 by the Sanitary Engineering Research Laboratory. The well layout is shown in Figure 9. The well-field is comprised of a central or



LAYOUT OF WELL FIELD IN U.C. RICHMOND FIELD STATION

FIGURE 9

re-charge well with 21 observation wells in close proximity. All of these wells are of large diameter and the depths exceed 100 feet. According to the Sanitary Engineering Department, the wells produce large quantities of fresh water from an aquifer of sand and gravel confined between impervious clay layers some 90 feet below ground surface. The aquifer is believed to be about four feet thick.

Details of the two wells at the other sites have not been obtained. No evidence of salt water intrusion has been detected at any of these wells, according to Mr. Shively.

According to Mr. R. C. Chin, of Contra Costa County Health Department, Richmond Division Contra Costa County requires permits before wells are drilled but the City of Richmond does not; consequently, many wells are not recorded with the Health Department. Mr. Chin provided a listing, presented in this report as Appendix C.

Mr. Chin also disclosed that the U.S. General Services Administration had required an extensive soils and ground water study prior to construction of the West Coast Headquarters of the Social Security Administration Office Building in Richmond. Mr. Rodriquez of GSA agreed to submit copies of data relative to the foundation study; however, at this writing the information has not been received.

The Pacific Gas and Electric Company had no drilled water wells on their Richmond properties, according to Mr. R. O. Jensen, Electrical Engineer with the Company.

During this survey the various parties were asked if they had knowledge of, or had experience with, ground water recharge in the Richmond area. The answers to these queries were negative.

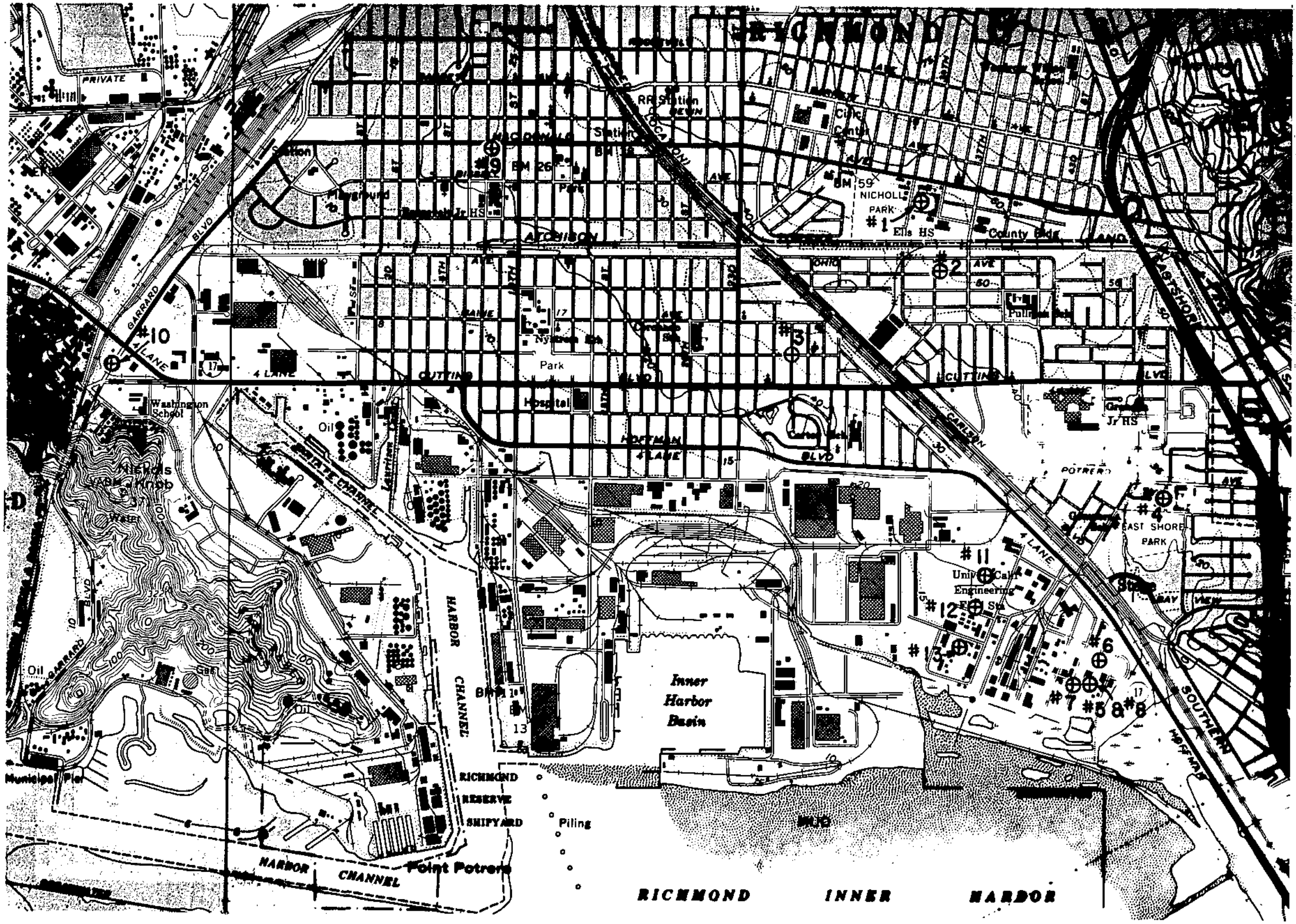
Mr. Charles E. Woodward, Planning Director, City of Richmond, provided information on the South Richmond Shoreline Special Area Plan. This plan identifies areas adjacent to the Hoffman Freeway and defines their present and/or planned uses. He stated that the East Bay Municipal Utility District does not have water supply outlets adjacent to the U.C. Richmond Field Station. He suggested inspection of a well owned by Allied Propane Co. on South 51st Street, near Stauffer Chemical Company Facilities.

Mr. Stan Teaterman, the owner of Allied Propane Company, was not available at the time of the survey. However, his shop foreman showed the well. The well is cased and capped. The steel cap has a 1/2-inch vent plug which can be removed for water level soundings. This well is about 100 feet deep and cased with 6-inch diameter plastic casing. The company uses several hundred gallons of water per day from this well. Water from the well is not used for drinking, although it is potable. The shop foreman indicated that other businesses in the area have their own wells.

Blair Excavators, Inc., located across from Allied Propane, on Sea Port Blvd. owns two wells in the area, one of which was inspected. Mr. Charles Shadwick supplied the following information. A large pit about 15 feet in depth was excavated and a steel tank with dimensions of 4- x 4- x 12-ft was perforated on the sides and laid with the 4- x 4-ft face on the bottom. This tank has a 1000-gallon capacity. A 1/2-HP jet pump was installed to move collected water to a holding tank on the surface. Mr. Shadwick stated that this shallow well water is used primarily for filling 2500-gallon tanks and that it takes about 6 hours to fill each tank. This amounts to about 7 gpm and allegedly results in no drawdown. On the day of inspection the ground water level was at a depth of 5 feet below existing ground surface. This water was tasted and was found to have a brakish taste, possibly due to the presence of a chemical.

TABLE 4, INFORMATION ON PRIVATE WELLS
(Furnished by DWR)

Well No.	Location	Depth of Hole	Date Drilled	Type of Casing	Diam.	Pump Rate	Depth of Drawdown	Time
1	130-33rd	210'	5/31/51	- - -	- - -	- -	- - - -	- -
2	35th & Florida	120'	2/15/73	- - -	- - -	- -	- - - -	- -
3	26th & Virginia	101'	10/11/51	Steel (single)	8"	- -	- - - -	- -
4	S49th & Potrero	120'	12/4/74	- - -	- - -	- -	- - - -	- -
5	S50th & Seaport	105'	5/20/69	Plastic (single)	6"	30 gpm	5'	2 hrs.
6	S50th & Montgomery	100'	9/8/71	Plastic (single)	6"	20 gpm	0	1 hr.
7	49th & Seaport	100'	5/2/74	Plastic (single)	6"	15 gpm	30'	3 hrs.
8	5000 Seaport	95'	1/22/68	Plastic (single)	5"	10 gpm	51'.	1 hr.
9	8th & McDonald	120'	3/23/73	- - -	- - -	- -	- - - -	- -



LOCATION OF WELLS

FIGURE 10

In the vicinity of this site a shallow holding pond was observed on property owned by the Stauffer Chemical Company.

Mr. Richard Zipp, of the Central District Office of the Department of Water Resources, provided information on the location of nine wells within the project area, some of which are cathodic protection wells. This information is summarized in Table 4. The well locations are indicated in Figure 10. A summary of the well driller's logs is also presented in Table 5.

Based on the above information, cathodic protection wells are located in the project area. It should be emphasized that cathodic protection wells do not produce water. According to Section 13711 of the California Water Code, a "cathodic protection well" is defined as:

> "Any artificial excavation in excess of 50 feet constructed by any method for the purpose of installing equipment or facilities for the protection electrically of metallic equipment in contact with the ground, commonly referred to as cathodic protection."

A special effort was made to understand the effects of lowering the ground water table or sea water intrusion on these installations. Mr. Richard F. Stratfull, Retired Caltrans Corrosion Engineer, now a private Consultant; Mr. Alvi Arshad, Corrosion Engineer, Sacramento Municipal Utility District; Mr. Kenneth Jukkala of U.S. Bureau of Reclamation, Sacramento; and Mr. Arnie Westerback, Corrosion Engineer with East Bay Municipal Utility District, Oakland, and Mr. Harry Eyeda, Materials Engineer, U.S. Bureau of Reclamation, Denver, Colorado, were contacted. It is understood that the long term effects of lowering the ground water table may reduce the operating efficiency of these installations while sea water intrusion on the other hand may be beneficial to them. A summary of the findings and further discussion are presented in Appendix D.

TABLE 5. DRILLER'S LOGS OF PRIVATE WELLS
(Furnished by DWR)

Well No.	Depth (ft.)	Soil Description
1	0 - 9	Clay
	9 -16	Gravel
	16-20	Clay
	20-30	Fine gravel
	30-100	Yellow clay
	100-130	Clay and gravel
	130-160	Clay
	160-170	Clay and gravel
	170-195	Clay - very hard
	195-199	Broken rock
	199-210	Hard rock
2	0-60	Brown clay
	60-72	Clay and sand streaks
	72-109	Clay
	109-120	Sand and clay
3	0-50	Yellow sandstone
	50-100	Blue sandstone
	100-101	Hard yellow rock
4	0-3	Bay mud
	3-8	Green sandy clay
	8-35	Brown clay w/sand and gravel layers
	35-80	Serpentine w/hard layers
	80-120	Very hard shale w/quartz layers
5	0-2	Dark clay soil
	2-31	Light brown sandy clay w/gravel stratas at 8-12', 18-20', and 20-31'
	31-105	Light brown clay w/gravel stratas at 40-41', 54-58', 71-73', 76-79' & 89-92'
6	0-16	Brown clay
	16-18	Gravel
	18-22	Brown clay
	22-26	Gravel and sand
	26-53	Gray clay and gravel
	53-57	Gravel and sand
	57-79	Brown clay and gravel
	79-81	Gravel and sand
	81-100	Brown clay and gravel

TABLE 5, DRILLER'S LOGS OF PRIVATE WELLS (continued)
(Furnished by DWR)

Well No.	Depth (ft.)	Soil Description
7	0-4	Top soil
	4-14	White clay
	14-47	Brown clay w/sand & gravel stringers
	47-100	Blue clay
8	0-3	Top soil
	3-10	Brown clay
	10-15	Brown clay w/sand
	15-20	Sand and gravel
	20-26	Yellow clay and gravel
	26-35	Yellow clay
	35-40	Blue clay
	40-45	Brown clay and sand
	45-75	Gray clay and sand
	75-85	Gray clay, sand and gravel
	85-89	Gray clay and sand
	89-95	Brown clay, sand and gravel
9	0-41	Clay
	41-53	Sand
	53-94	Clay
	94-108	Sand and gravel
	108-120	Clay

LABORATORY INVESTIGATION

The laboratory phase of the investigation was kept to a minimum due to the abundance of applicable test data from the 1971 study. Laboratory efforts, therefore, were confined almost entirely to index and classification tests such as grain size analysis, sample unit weight determinations, and moisture contents. Types and number of tests are shown in Table 6, and actual results are listed on the appropriate boring profiles in Appendix A. All tests during the present investigation were conducted by District Materials Personnel.

The most important part of the laboratory investigation was the visual inspection of undisturbed 2-inch diameter samples from 12 borings. These samples represented continuous soil columns from ground surface to the bottoms of borings and provided the opportunity to inspect each soil layer and transition zone. The objective was to assess, subjectively, the permeability of each soil based on particle size, plasticity, and denseness. For each boring, the samples were extruded from the 2- by 4-inch brass tubes and placed in order in core boxes. Each sample in turn was then split longitudinally to expose material relatively unaffected by sampling operations. The inspection was conducted by experienced personnel who assigned permeability ratings of low, moderate, or high, based on the previously mentioned criteria. Results are shown in Table 7.

TABLE 6, TYPES AND NUMBER OF TESTS

Series	Year of Test	Unit Wt.	Moisture Content	Sieve Analysis	Torvane	"R" Value	Consoli-dation	Uncon-fined	Triaxial
P	1977	-	18	18	-	10	-	-	-
D	1977	221	55	49	64	-	-	-	-
B	1971	75	74	74	-	-	35	20	2

TABLE 7, VISUAL IDENTIFICATION OF SOILS

Area No.	Boring No.	Station Station	Depth of Excavation	First Aquifer At* (ft)	First Aquifer Thickness (ft)	First Aquifer Type**	Second Aquifer At* (ft)	Second Aquifer Thickness (ft)	Second Aquifer Type**	Third Aquifer At* (ft)	Third Aquifer Thickness (ft)	Third Aquifer Type**	Fourth Aquifer At* (ft)	Fourth Aquifer Thickness (ft)	Fourth Aquifer Type**
-	D9	166+70	7.0	9	12	H.P.	40	2	L.P.to H.P.	61	8	H.P.to M.P.	-	-	-
1	D17	172+35	14.5	9	8	H.P.	27	2	L.P.to H.P.	47	12	L.P.to H.P.	-	-	-
1	D18	178+50	26.0	11	4	L.P.to H.P.	15	6	L.P.to H.P.	24	4	H.P.	28	3	L.P.to H.P.
1	D16	182+20	23.0	9	7	L.P.to H.P.	43	4	H.P.	-	-	-	-	-	-
2	D11	187+50	14.0	14	2	L.P.	20	1	H.P.	49			-	-	-
2	D10	202+90	9.5	5(?)	4	H.P.	27	8	L.P.to H.P.	42			-	-	-
4	D12	236+75	14.5	6	6	L.P.to H.P.	-	-	-	-	-	-	-	-	-
4	D8	239+60	15.0	30	9	H.P.	66	12	L.P.to M.P.	-	-	-	-	-	-
5	D13	257+45	17.0	12	3	L.P.to H.P.	37	2(?)	H.P.	-	-	-	-	-	-
5	D14	264+20	15.0	16	1	M.P.	18	3	H.P.	38	1(?)	H.P.	-	-	-
5	D15	272+00	11.0	6	6	M.P.to L.P.	16	1	M.P.	16	1	M.P.	-	-	-
5	D7	272+20	11.0	6	6	M.P.to L.P.	36	5	H.P.	36	5	H.P.	48	3	H.P.

**Type
H.P. = High Permeability
M.P. = Moderate Permeability
L.P. = Low Permeability

*At = Below existing ground surface.

SUBSURFACE CONDITIONS

Sources of Information

The subsurface conditions within the project area are extremely complex. Evaluation of these conditions necessitated a thorough review of all available sources of information. A partial list of the sources utilized in this study follows:

1. Boring Logs of holes drilled and sampled by Caltrans (District 04, District 10, and Office of Structures).

2. Visual identification of undisturbed samples.

3. Well driller's logs from the California Department of Water Resources.

4. Various Tri-Cities (Cities of El Cerrito, Richmond and San Pablo) reports embracing seismic elements, geological and geophysical elements, and environmental analysis.

5. Observed aquifer characteristics during pump tests (1971 and 1977).

6. Various maps of U.S. Geological Survey.

7. State Water Quality Control Plan report for the San Francisco Basin.

8. California Department of Water Resources Publications.

9. Information obtained from geotechnical reports of two private soils firms.

This section briefly describes the sub-surface soils; discusses the so-called "Richmond Aquifer", and presents two models to describe the existing subsurface conditions.

Soil Descriptions Based on Caltrans Borings

Area 1, Stations 170-184 (1400 ft)

The 7 sample borings in this section range from 30 to 125 ft in depth. No soil patterns or trends were revealed by this group. Clayey sands with gravel alternate with silty clays and clayey silts in an extemely random manner, both vertically and laterally. The fine grained soils are moist, and range from soft to hard with increase in depth. The granular materials are saturated and dense.

Area 2, Stations 184-205 (2100 ft)

The subsoils are relatively uniform in this sector along the line of proposed excavation. Silty clays predominate from ground surface to depths of 20 to 30 feet. Intermittent minor layers of wet, dense sand and gravel alternate with the clays at greater depths. Boring B7 terminated in weathered rhyolite some 110 feet below ground surface; whereas the rhyolite was found at a depth of only 65 feet in Boring B6. Boring D11 was bottomed in dense, gravelly sand at 80 ft below ground surface.

Area 3, Stations 205-231 (600 ft)

The subsoils within this section to depths of about 30 feet are basically overconsolidated sandy or silty clays interrupted by minor layers of wet, dense sand and gravel containing clay binder. Below 30 ft silty fine sands with some gravel predominate. Boring B10A at Station 214 encountered weathered rhyolite at a depth of 40 feet (Elevation -30).

Area 4, Stations 231-252 (2100 ft)

This section is characterized by massive layers of firm to hard clays and silty clays from ground surface to depths of some 70 feet. Sandy layers here are relatively thin and well separated vertically. In the vicinity of Stations 250-252 stiff silts containing varying increments of sand or clay are predominant.

Area 5, Stations 252-273 (2100 ft)

Within this section all borings show clays, silty clays, and clayey silts with some intermittent dense sand and gravel layers between Elevation -20 and -40. The deepest boring in this area (B-2) was bottomed in hard silty clay at 110 ft below ground surface.

Area 6, Stations 273-276 (300 ft)

The soils of this short section are principally overconsolidated clay, interbedded with lenses of fine sand and some gravel. The clays contain shale fragments, particularly in the surface areas. The clays are firm to hard in consistency. Boring B1 logged sandy or clayey silt to a depth of 20 feet below ground surface; however silts were not observed in significant quantities in other borings within this section.

Detailed descriptions of the soils and other pertinent boring data are shown on the individual boring profiles, Appendix A.

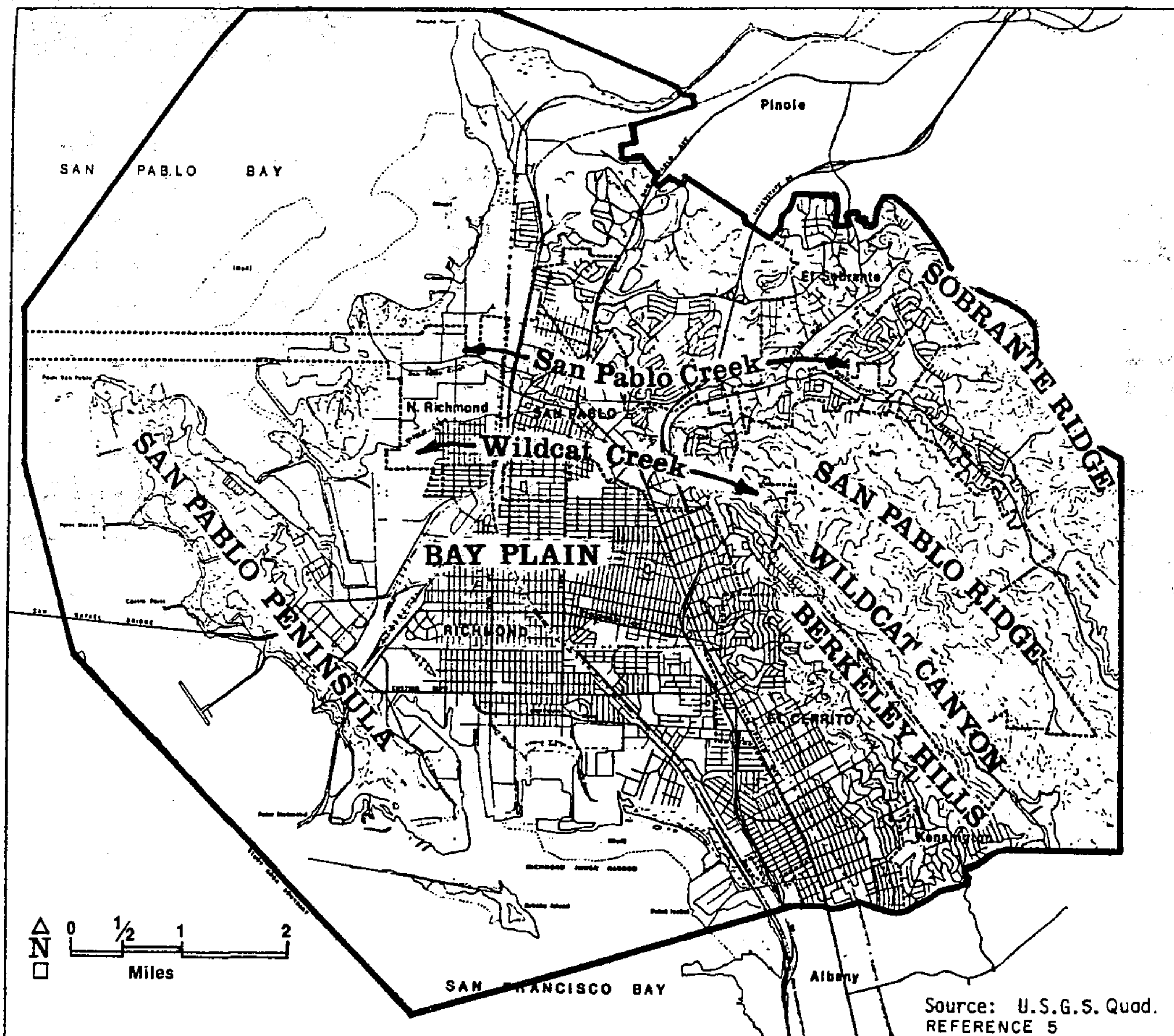
Subsurface Information

In this subsection the geomorphological setting of this project is discussed and the results of a literature survey to confirm or disprove the existence of the so-called "Richmond Aquifer" are presented. Finally, the first two of the four models will be presented to describe the present subsurface conditions. The other two models, which deal with sea water intrusion, will be presented in a later section.

Geomorphological Setting

As shown in Figure 11, the project area lies in a large flat area called the Bay Plain and is flanked by four southeast-northwest trending ridges. The following quote from (5) describes the topography adequately.

> "The most westerly of the ridges is the San Pablo Peninsula, which is separated from the other ridges by the Bay Plain. The Bay Plain is characterized by a long shoreline, part marshy and part hilly, and by extensive flatlands where the population of Richmond, San Pablo, and El Cerrito is concentrated.
>
> "The other three ridges, the Berkeley Hills, San Pablo Ridge and Sobrante Ridge, are the westernmost extension of the Briones Hills, a complex of hills and valleys noted for its rugged topography and unstabel slopes. El Cerrito climbs the more gentle southwest-facing slopes of the Berkeley Hills, while the steep northeast-facing slopes decend rapidly to Wildcat Creek. Between the Berkeley Hills and San Pablo Ridge is Wildcat Canyon, a



TOPOGRAPHY OF AREA SURROUNDING RICHMOND

FIGURE 11

rugged, wild area, owned in large part by the East Bay Regional Park District. On the north side of the San Pablo Ridge, the valley of San Pablo Creek separates San Pablo Ridge from Sobrante Ridge. The community of El Sobrante is located largely in this valley."

The geology of this area in general is explained by a quote from (6) as follows:

"GENERAL STATEMENT

The terrane in the Tri-Cities study area consists of a complex, inhomogeneous sequence of sedimentary, igneous, and metamorphic rocks ranging in age from Late Jurassic to Quaternary. "Basement" is composed of a complex assemblage of Franciscan rocks. Stratigraphically above these materials lies a section of Tertiary marine and nonmarine sedimentary and volcanic rocks. The marine section of Eocene to Miocene age is represented by a rhythmically bedded sequence of sandstone and shale with occasional interbeds of pyroclastic volcanic material. The Pliocene nonmarine section is composed of loosely consolidated sedimentary rocks as well as a small amount of basaltic volcanic rocks. The Quaternary is represented by bay mud and various alluvial deposits.

FRANCISCAN TERRANE

With relatively minor exceptions, the western ridge line of the Berkeley Hills marks the easternmost outcrops of Franciscan rocks in the study area. The rock types common to the Franciscan are serpentinite, greenstone, graywacke, chert, shale, sandstone, and glaucophane schist. In their fresh, unweathered, unsheared states, most of these rocks are dense, hard, and resistant and underlie areas that will be relatively stable during earthquake shaking. However, where intensively sheared and/or weathered, these rocks disintegrate and form much less stable zones. As much as possible, the Slope Stability map reflects these variations.

MIOCENE AND EOCENE MARINE ROCKS

Outcrops of the marine sedimentary section are very limited within the study area. The Miocene and Eocene section is represented by a rhythmically bedded sequence of well-indurated sandstone and shale. These rocks probably underlie the Contra Costa Group of Pliocene age in many places, however, they appear to be absent west of the Wildcat Fault. In some places, the marine sedimentary rocks have slid extensively while adjacent areas of the same formations have remained stable.

CONTRA COSTA GROUP

Rocks of the Contra Costa Group crop out east and north of the western Berkeley Hills ridgeline. Within the study area, the Contra Costa Group is composed of friable, poorly-indurated conglomerate, sandstone, and siltstone, all of which contain significant amounts of clay (the clays are largely of the montmorillonite type [Radbruch and Case, 1967] and are therefore, expansive). Individual beds are lenticular.

The rocks of the Contra Costa Group and the overlying soils are particularly susceptible to creep and landsliding. The rocks are fairly incompetent (Radbruch and Case, 1967), friable, loosely cemented or compacted, and extremely heterogeneous. The clay-rich soil that develops on these rocks actively moves downhill through a pronounced shrinkage and swelling response to desiccation and saturation, respectively. In many areas, the soil develops extensive sets of cracks which may be 2 or 3 inches wide, 1 to 3 feet deep, and tens of feet long. The intensity of landsliding within the Contra Costa Group may also vary with grain-size, composition, natural slope, or other factors.

QUATERNARY SEDIMENTS

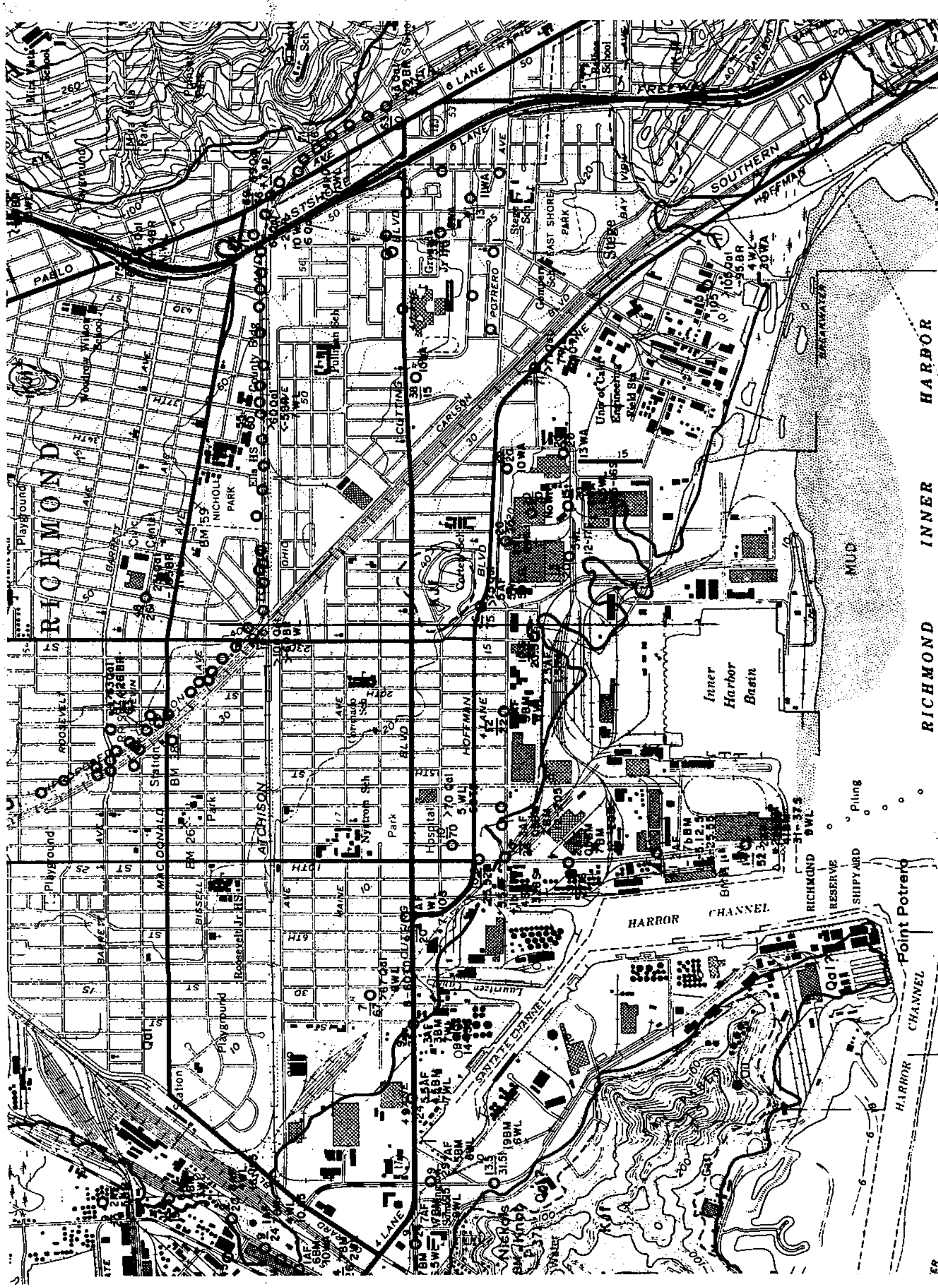
Alluvium and bay mud dominate the low-relief areas of the three cities, the valley of San Pablo Creek, and the nearshore bay deposits. Bore hole data indicate that the alluvium is composed of clay, silt, sand, and gravel, and the bay mud is composed chiefly of a clay and silt mixture with occasional sand layers."

The major geologic feature in the western Contra Costa area is the northwest-trending Hayward fault zone. This 44-mile long zone varies in width from a few feet in the vicinity of San Pablo Bay to over a thousand feet or more. It presents a major topographic feature along the base of the Berkeley Hills in this area. The fault enters San Pablo Bay approximately one mile south of Pinole Point(7).

The broad alluvium plain which emerges south and west of the Hayward fault slopes gently towards San Pablo Bay. As mentioned above, this alluvial deposit is composed of random layers and thin zones of clay, silt, sand, and gravel. These sediments range from less than a foot to 400 feet in thickness. Due to the variable nature of this alluvium and placement of artificial fill, wide variations in soil conditions are present(7).

Information was obtained from Bishop et al(6), as to the depth of alluvium in the project area. The locations of the various bore holes from which this information is obtained are shown in Figure 12. The alluvium thickness contours are presented in Figure 3. In the vicinity of the project the thickness of alluvium varies from about 2 feet to more than 300 feet. Other boring data extracted from Bishop, et al(6) are summarized in Table 8.

The original shorelines of a portion of San Francisco Bay and the San Pablo Bay are shown in Figure 13. The hatched area shows man-made land overlying San Francisco and San Pablo Bay muds. The entire project area is outside this hatched area except the westerly end, which is very close to the inner harbor channels.



SUBSURFACE INFORMATION

FIGURE 12

SOURCE: REFERENCE 6

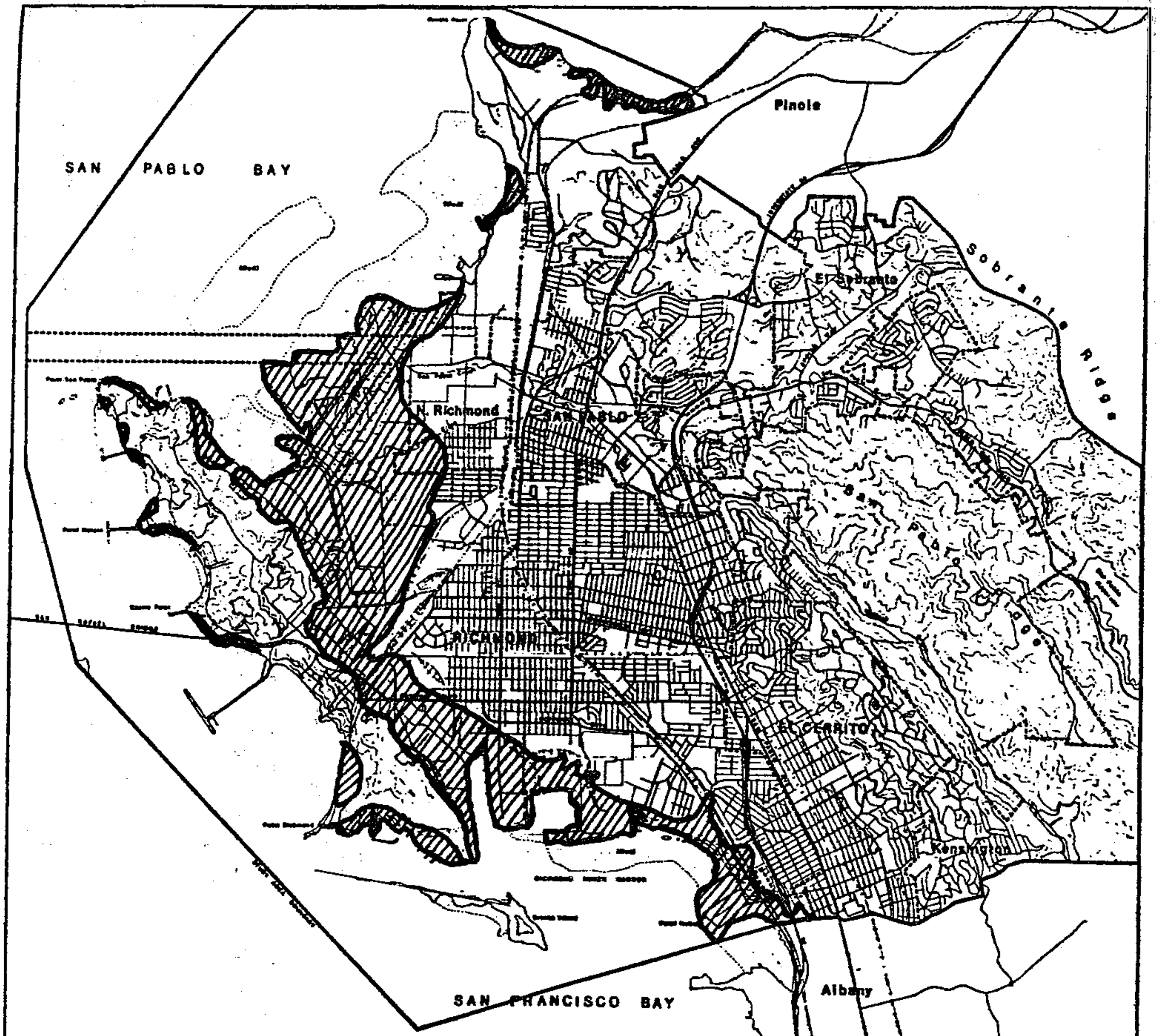


FIGURE 13 MAN-MADE LAND OVERLYING SAN FRANCISCO AND SAN PABLO BAY MUDS

SOURCE: REFERENCE 5

TABLE 8 BORING DATA (FROM TRI-CITIES REPORT)

No. of Hole	Ground Elev.	Water Level	Boring Depth	Thickness of Alluvium
1	7'	6'	67'	>67'
2	10'	5'	70'	>70'
3	15'	5'	75'	>75'
4	31'	7'	77'	>77'
5	10'	4'	105'	>105'
6	9'	6'	29'	-
7	13.5'	6'	31.5'	-
8	9'	7'	24'	-
9	9.6'	-	2'	-
10	12'	6'	22'	-
11	13'	5'	20'	-
12	15'	14.5'	20'	-

Note: Artificial fill in vicinity of Richmond Harbor is 2.5 to 5 ft in thickness.

Richmond Aquifer

During the literature search for a definition of the "Richmond Aquifer" it was found that apparent confusion exists as to its identity. In the following passages are quotes from various sources, with comments. In the next subsection names are suggested for the ground water basin and the various aquifers (at different depths) within this basin.

Robert L. Nevin, of Environmental Assessment Engineering, in an environmental impact report(8) on a Western Contra Costa Sanitary Landfill Project, calls the Bay Plain (Figure 11) the Richmond Basin. Even though this particular project is far northwest of the proposed Hoffman Freeway, the general topographical description of the basin is still applicable. The following is quoted from that report.

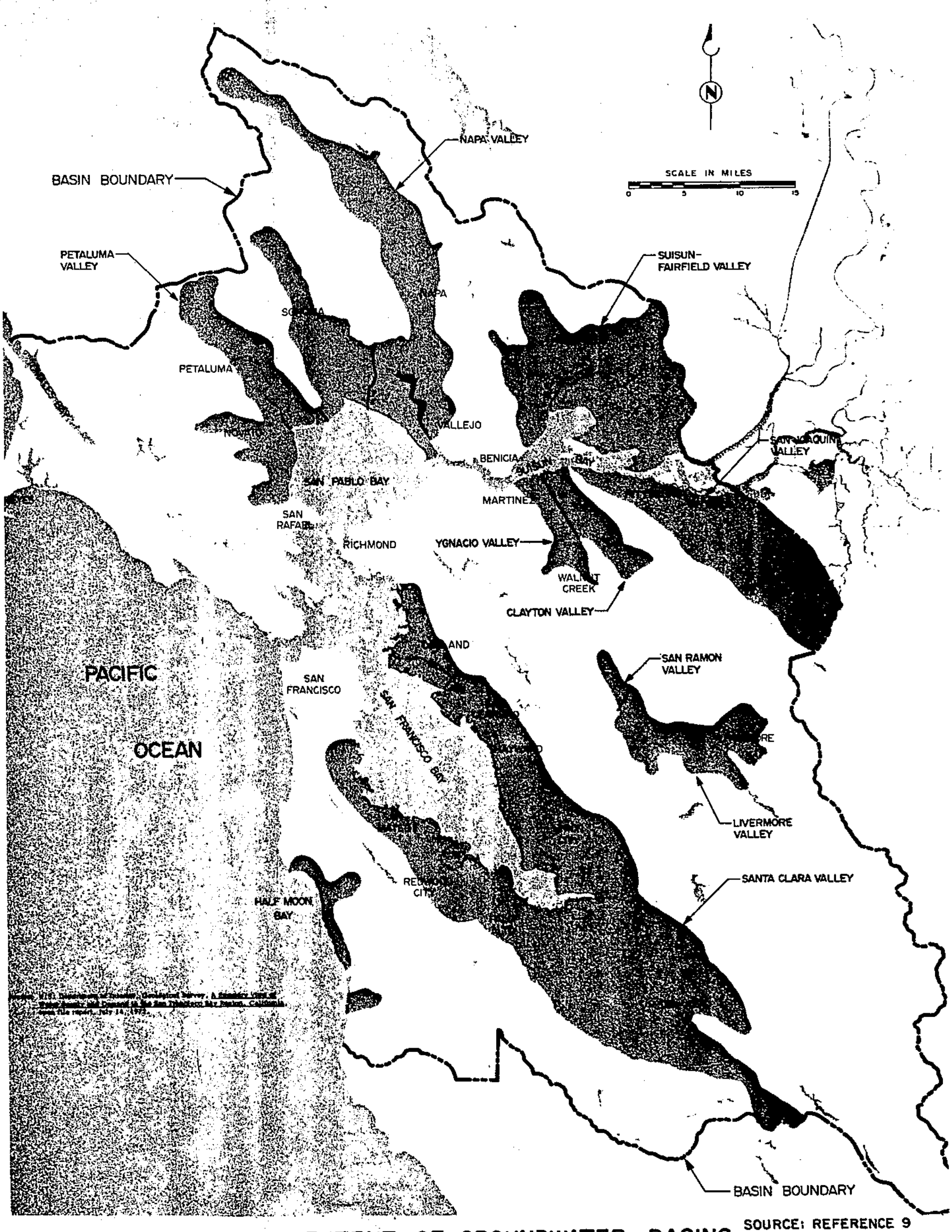
"The groundwater basin encompassing the Project Site, herein referred to as the Richmond Basin, occupies an area bounded on the north and east by the Hayward fault, marked by the Berkeley Hills, and on the south and west by the San Pablo fault, which lies along the north edge of Point San Pablo Peninsula. To the northwest, the Basin emerges with bay mud deposits and/or sediments from sources across the Bay. Consolidated bedrock underlies the Basin sediments at depths averaging perhaps 300 feet.

"The Richmond Groundwater Basin is made up of consolidated sediments of clay, sand, and gravel, which dip generally westward toward the Bay in this area. Clay predominates in the Basin, making up as much as 90 percent or more of the alluvial section in most areas for which data are available. It occurs typically in large thicknesses and is virtually impermeable. Sand and gravel layers, deposited mainly by San Pablo and Wildcat Creeks, are sparse and highly variable in occurrence and cannot be individually traced for any distance. Generally only a few feet in thickness, they are found mostly below depths of 100 feet or more. Such deeper pervious beds constitute the only productive aquifers within the area.

"Semi-consolidated and unconsolidated clay and mud underlie San Francisco Bay and extend beneath the entire project area to maximum depths of about 150 feet. Thin stringers of sand occur erratically within these materials, but are discontinuous over any distance and have no direct connection with sediments of the Richmond Groundwater Basin."

Very little geological mapping of the ground water basin has been done in the past. In fact, the "Tentative Water Quality Control Plan Report for San Francisco Bay Basin, Part 2"(9), published by the State Water Resources Control Board presents a figure (Figure 14) that does not show any ground water basin of importance. The following paragraphs are quoted from this source(9):

"In the Central and North Bay areas groundwater aquifers are of limited scope and importance. One comparatively large aquifer on the east side of the Bay is composed of old alluvial deposits overlaid with more recent fine-grained alluvial deposits and bay sediments. This formation outcrops along the western edge of the Berkeley Hills and extends westward beneath the Bay where it is covered by 25 to 100 feet of bay mud.



SOURCE: REFERENCE 9

AERIAL EXTENT OF GROUNDWATER BASINS

FIGURE 14

"The principal aquifers adjacent to San Pablo Bay underlie the alluvial plains of the Petaluma, Sonoma, and Napa Valleys. In the Petaluma Valley the principal groundwater body occurs in the old alluvial deposits which underlie the valley as well as the recent alluvial and mud deposits existing near the Bay. The aquifer has a depth which varies from 75 to 150 feet on the valley floor and 75 to 250 feet near the Bay."

The report "Environmental Analysis of Western Contra Costa County", a part of a series of reports to the Cities of El Cerrito, Richmond and San Pablo (hereafter referred to as Tri-Cities)(10) concludes as follows:

"The study area has no areas of groundwater, or "aquifers", large enough for use in irrigation or as a permanent municipal water supply (Webster, 1972). Map 5-4 [Figure 15 of this report] shows what is presently known about the groundwater in the Tri-Cities Area; its precise extent and capacity have not been investigated."

The above quote tends to confirm that studies have not been performed to delineate thoroughly the extent of this ground water basin or determine its geohydrological and geotechnical characteristics.

The location of ground water in this basin is summarized by two sources. The first source is the Tri-Cities Report on "The Open Space and Conservation Element of the General Plan"(12), quoted as follows:

"Groundwater is found in the flat part of the study area, chiefly near North Richmond and Western San Pablo. The groundwater is a limited resource and could yield at most perhaps one million gallons of water per day. However, the groundwater basin is very shallow and is extremely susceptible to pollution and overuse, which could cause sea water intrusion and subsidence of the land. Liquid disposal of wastes and deep dredging could also damage the supply. Thus, the local groundwater supply is useful chiefly as an emergency supply in case of disaster or water shortage and for various supplemental uses, such as irrigation."

The second source is Webster(11 and 13), and consists of two maps of the San Francisco Bay Region. The first shows areas where certain chemicals such as Boron, etc., are present; and the second shows probable well yields in the ground water basin.

Webster(11) has presented generalized information regarding the probable maximum yields of wells in the San Francisco Bay Region. He divided the entire region into four areas by symbols A, B, C and D. An enlargement of this particular ground water basin is presented in Figure 15.

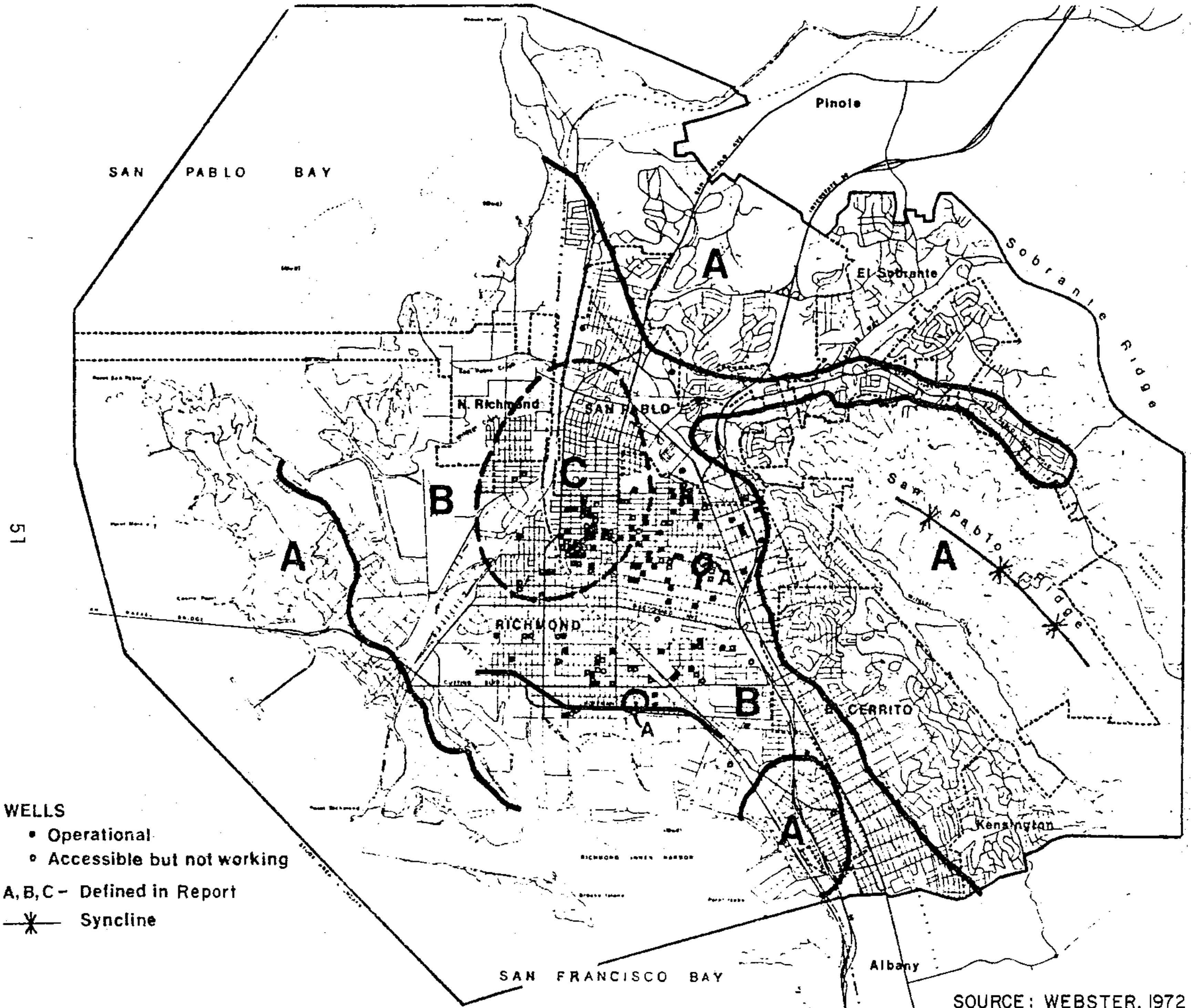
The explanation of symbols A, B, and C summarized in Table 9. Quoting Webster(11):

> "Fresh-water-bearing deposits that occur beneath the bays of the region beneath salt- and brackish-water deposits along the margins of the bays are not evaluated."

Also he indicates that boundaries of very poor control are indicated by dashed lines and that more detailed mapping at a larger scale would be warranted in many areas. Looking at Figure 15, the Hoffman Freeway is located in Area B with a small circular area near Cortez School having delineation A.

The wells in Area (B) will have maximum yields of 5 to 50 gpm with a 68 percent chance, and of 1 to 100 gpm with a 95 percent chance. This was verified during the two recent Caltrans pump tests (1977) in which the pump rates were between 2 and 3 gpm. The Summary Water Resources Report for Tri-Cities(14) summarizes the findings of Webster(11) in the following words:

> "What significant groundwater is available in the study area is located in the areas marked "B" and "C" on the map. The area marked "B" is sufficient for single-family use, but inadequate to marginal for light industry. The area marked "C" is the area of greatest groundwater



SOURCE: WEBSTER, 1972
Reference 11

FIGURE 15. LOCATION OF GROUND WAT

supply and will yield enough for light industry, but inadequate to marginal for irrigation, heavy industry, or a municipal supply.

TABLE 9 RANGES IN THE PROBABLE MAXIMUM YIELD OF WELLS

Map Symbol	Adequacy of yield (at 68-percent level of chance)	68-percent chance that maximum yields will range from (gpm)	95-percent chance that maximum yields will range from (gpm)
A	Marginal to adequate for stock or single family domestic use.	0.5 to 5	0.1 to 10
B	Adequate for stock or single family domestic use, but inadequate to marginal for light industrial use.	5 to 50	1 to 100
C	Adequate for light industry, but inadequate to marginal for irrigation, heavy industry, and municipal uses.	50 to 500	10 to 1,000

Source: Reference 11.

"Those areas mapped "A" produce only enough water to be marginally adequate for a single-family home or for cattle. Consequently, wells drilled here may go dry or yield much less water in the summer and autumn.

"There are some springs in the Wildcat Canyon-San Pablo Ridge Area. The sandstones and gravels in this area dip to form a "syncline", where water may accumulate . . . . However, the East Bay Municipal Utility District has no records of wells in this area. The groundwater here may be adequate for Park District or other minimal uses, however".

There were numerous wells in operation in the entire ground water basin in the early 1900's. The following paragraphs from (10) bring out the history of these wells:

"At one time over 300 wells were operating in areas B and C for domestic and agricultural purposes. (RPD, 1971). In fact, the settlement of western Contra Costa County was made possible by the existence of groundwater. However, the supply proved inadequate by 1918, when there was no water in local reservoirs for over five months, and the entire supply had to be drawn from wells which were producing a decreasing yield of low-quality water. After seventeen private water companies failed, the East Bay Municipal Utility District was formed in 1923. (EBMUD, 1972). EBMUD capped most of the wells in the area when it began operation.

"In 1970 the County Health Department began a study of Richmond wells to establish the depth of the wells, the quality of the water, and the location of operating wells. The study discovered 97 operating wells and 59 not working but accessible in case of emergency. .... Three are used for drinking water; most, for lawn irrigation. In general, the wells are drawing water from 25 to 180 feet deep. (CCCHD). This is lower than the level where water is first encountered when boring a hole because wells are usually constructed to draw water from a certain depth. (Bishop, DMG).

"The chief use of the groundwater today is as an emergency supply of water is case of earthquake or other diaster. In an earthquake, the flat part of the study area could be cut off from the water supply in San Pablo Reservoir. In that case the groundwater supply could be crucial."

Appendix C of this report presents a summary of private wells in the vicinity of the Hoffman Freeway. Of 91 private wells identified during the survey (Appendix C), only one was being used for drinking purposes. In the vicinity of the University of California's Richmond Field Station, a number of private wells are in use for purposes other than drinking.

At the Richmond plunge, near the project west end, a 200-ft well (originally intended as an oil well and drilled to 2,000 feet) exists. It is reportedly capable of delivering at least 100 gpm with less than 30 ft of drawdown. The aquifer is at least 90 ft below ground surface at this well. Near Martin Luther King Park, fresh water is supplied by a well. Near the project east end, in the U.C. Richmond Field Station, there is a well-defined, confined 4-ft thick aquifer at a depth of 90 ft, capable of abundant supply of water.

The delineation of the depths and thicknesses of the various aquifers within this ground water basin are described variously in the literature. Aquifers are found not only at shallow depths mentioned earlier, but also at greater depths as quoted(15) below.

> "The majority of aquifer material, which lies at greater greater depths, is overlain by thick and tight clay zones which serve as aquicludes to confine these aquifers under artesian pressure. This deep aquifer zone is encountered at depths of 80 or 100 feet several miles to the east of the landfill area, but deepens to below 180 feet in the project vicinity. This zone has not been fully penetrated by most wells and appears to vary greatly in makeup and thickness. Except when pulled down by localized heavy pumping, deep wells exhibit a water level (piezometric surface) much above that of the actual producing aquifers. This condition exists beneath all the lower lands in the West Richmond-San Pablo area and for an undetermined distance eastward. Other miscellaneous thin aquifers at shallower depths are also overlain by substantial thickness of impermeable clays and probably have their own individual pressure heads.
>
> "Very shallow sand and gravel beds carry meager quantities of water which are unconfined and water table conditions prevail. Most of these occurrences are perched by underlying clays; this is particularly the case for sand lenses within bay mud deposits."

The basin capacity has been roughly estimated by the U.S. Geological Survey. Quoting from Reference (10):

> "The U.S. Geological Survey has estimated that the entire groundwater basin from San Lorenzo to Richmond could safely yield approximately 4.5 million gallons per day of

water (Rantz, 1972). The Richmond area appears to constitute perhaps one-fourth of this, or to yield slightly over one million gallons per day. This is a rough guess at best, however."

Reference (14) presents the following conclusions:

"The U.S. Geological Survey has estimated that the Richmond basin could safely yield approximately 1.0 million gallons per day of water.

"However, both the Utility District and a geologist from the State Division of Mines and Geology feel that any pumping from such a small, shallow aquifer might cause serious problems.

"First, salt water might invade the fresh water.

"Second, pumping groundwater might cause the land to subside, causing serious property damage."

Diverse opinions expressed in these quotes illustrate the need for further study on basin capacity. Based on the two more recent Caltrans pump tests (1977) and ground water analysis with regard to the Hoffman Freeway, the best estimate (considered conservative) of pumpage during construction is between 2.5 mgd and 3.5 mgd. The long term pumpage during the operational phase is estimated at the upper limit of 1.4 mgd.

Recharge, at least in the area of the Hoffman Freeway, occurs very slowly. Four quotes are presented below to summarize the findings on how the ground water basin is recharged.

Reference (14) states....

"No detailed studies have been made of the groundwater recharge in the Tri-Cities area. However, in the Tri-Cities area, most soils are clay or clay loams - the types generally least permeable to water. Thus it is very likely that the groundwater basin is recharged only very slowly, and that much time will be required to replace any water taken out."

Reference (12) states....

"To maintain the groundwater supply, the basin must be recharged each year from "groundwater recharge areas". No studies have been completed to discover specific areas of groundwater recharge in the Tri-Cities area. However, recharge usually occurs in stream channels and in floodplains."

Reference (10) states....

"Recharge to groundwater occurs by direct infiltration of rainfall, underflow, and seepage along stream channels. In the hills, early season precipitation is almost completely retained as soil moisture, and infiltration to groundwater must await later storms that exceed the capacity of the soil to absorb moisture. (Livingston and Blayney, 1971)."

Reference (15) states....

"Replenishment of basin groundwater is mainly from percolation of stream flows in high areas considerably east of the landfill site where aquifers are not capped by impermeable clays and can receive surface water seepage. From such recharge areas, groundwater moves as the ground slopes toward the Bay."

The water from shallow aquifers can be expected to be brackish. From deeper aquifers fresh water can be obtained. Based on the basin capacity and water quality it is interesting to note that the following conclusions were arrived at by the authors of the various Tri-Cities Reports(14):

"The County Health Department tested the water in the wells it found operating. The water was found to be suitable for drinking if chlorinated, with no evidence of sea water intrusion. The supply is easily polluted, however. The presence of fresh water as close to the ocean as Point Isabel also indicates that the groundwater is retaining sufficient pressure to resist sea water intrusion.

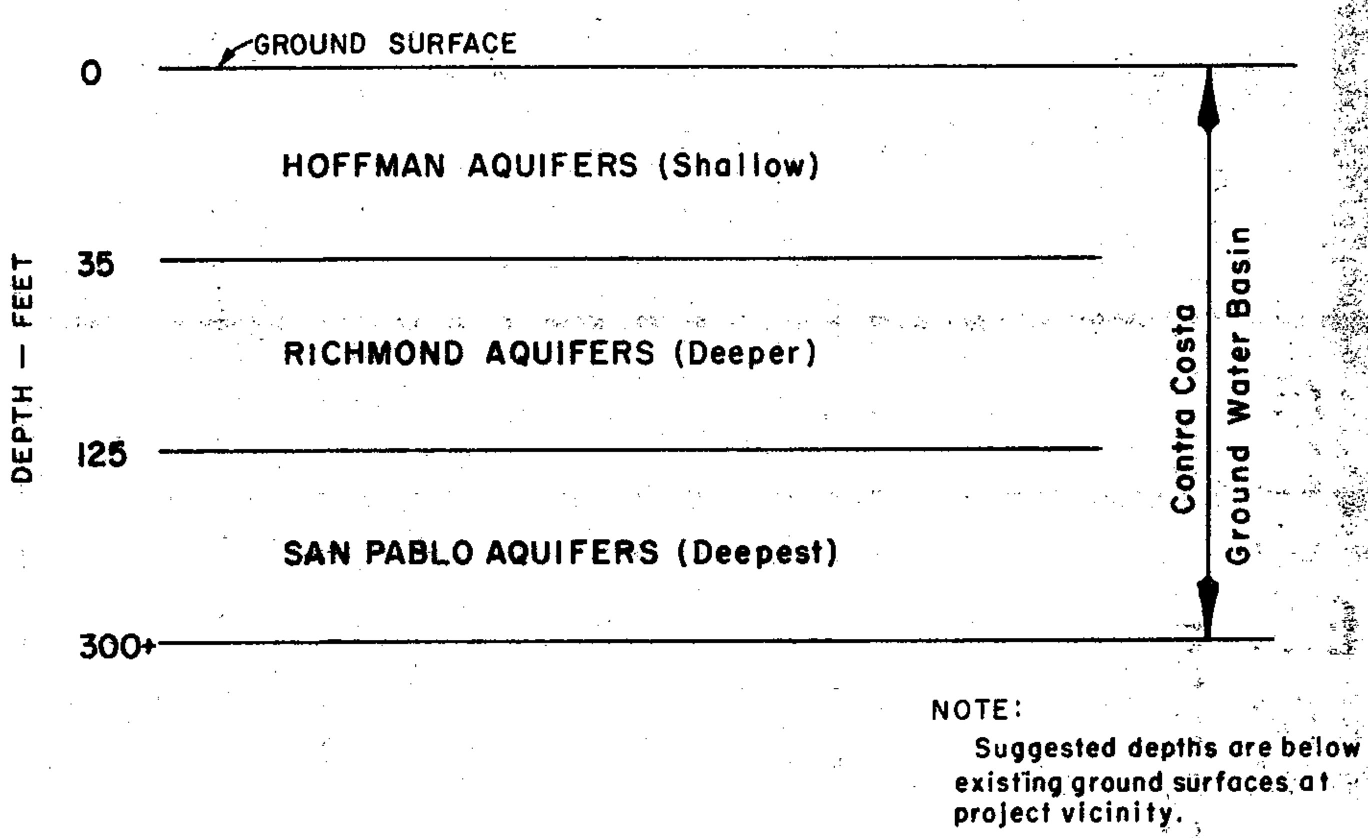
"Because the area's groundwater has not been a crucially important resource, very few plans or policies have been made regarding it. Richmond's Coastline Plan found that the aquifer would provide a useful emergency water supply and that damage to the aquifer could result from deep dredging and underground disposal of liquid wastes. The Plan has proposed that: (1) the excessive withdrawal of underground supplies be prevented; (2) the underground disposal of liquid wastes be prohibited; and (3) any development be rejected that would degrade or deplete the underground water supply."

Suggested Names of Aquifers and Basins

The presence of a series of aquifers at shallow depths (0 to 35 feet) have been definitely confirmed during this investigation. The name Hoffman Aquifer<u>s</u> (in plural and not singular) is suggested for this family. From the field review of existing wells, boring records, and personal interviews the existence of a series of aquifers between depths of 35 and 125 feet has been established. The name Richmond Aquifer<u>s</u> (in plural and not singular) is suggested for this series. From all available geological records, the maximum thickness of alluvium is 300 feet or more, not only in the project vicinity, but also in the entire basin; and the name San Pablo Aquifer<u>s</u> (in plural and not singular) is suggested. The above suggested names are presented in model form in Figure 16. The entire ground water basin discussed above is recommended to be referred to as the Contra Costa Ground Water Basin, Figure 17. This area is referred to as Bay Plain in Figure 11.

<u>Proposed Models of Subsuface Conditions</u>

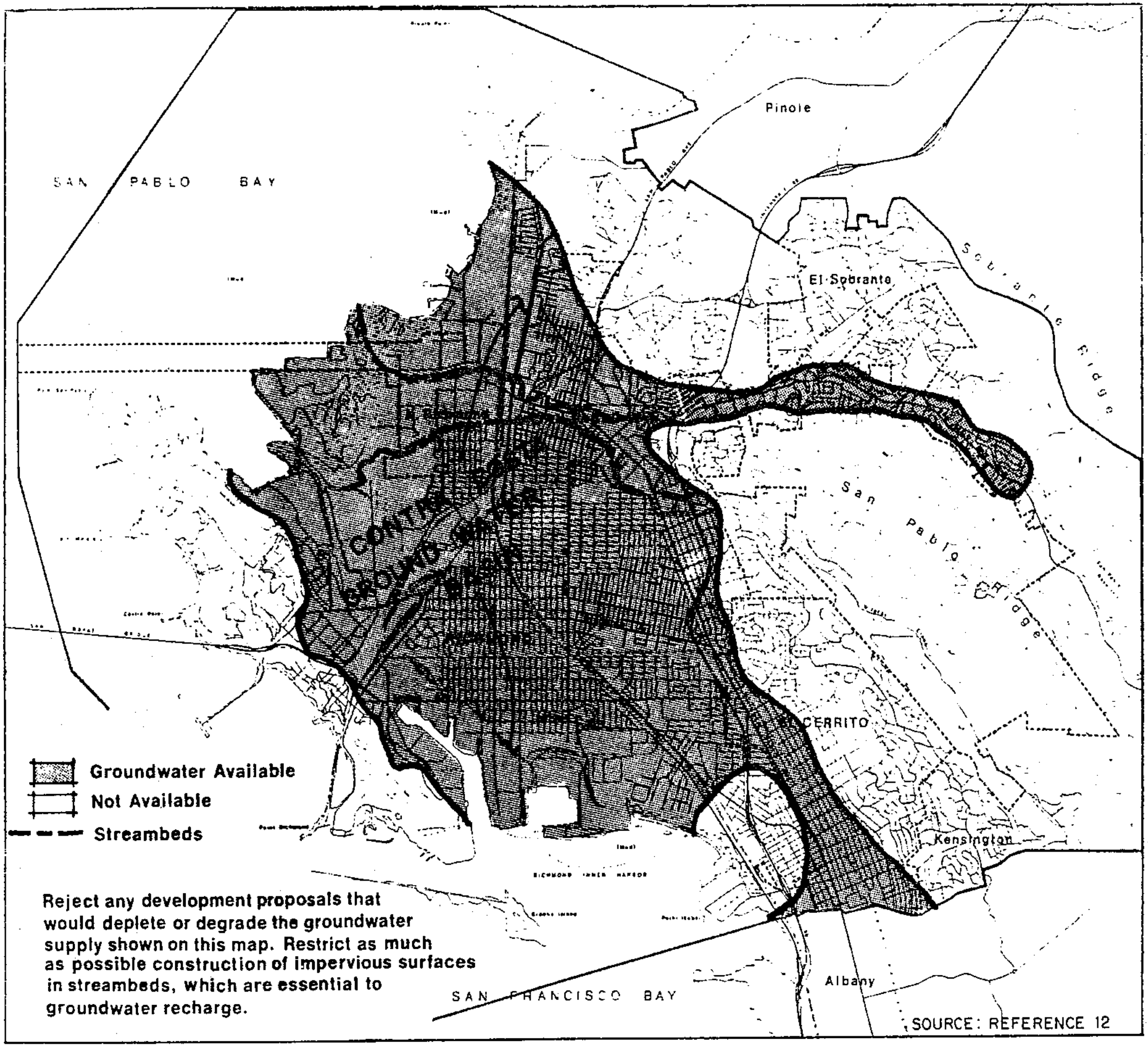
In the absence of a generalized soil profile, four models are proposed in this report to represent the subsurface conditions



SUGGESTED NAMES OF AQUIFERS AND BASIN

NO SCALE

FIGURE 16



SUGGESTED NAME OF GROUND WATER BASIN

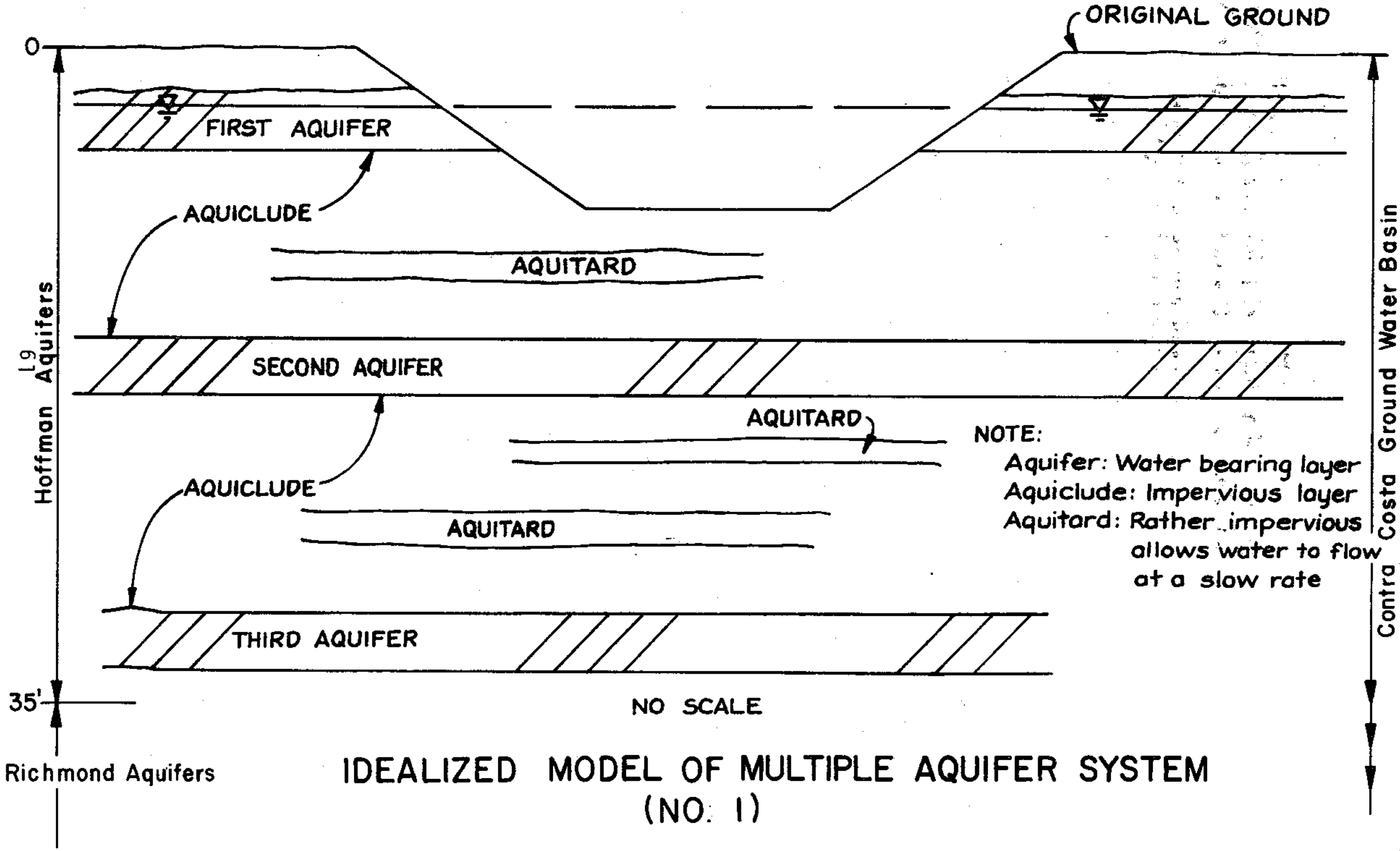
FIGURE 17

along the entire stretch of the semi-depressed section. Two of these models will be presented here. The third and fourth models will be described under the section on Sea Water Intrusion.

Model 1: Idealized Model of Multiple Aquifer System

The deeper aquifers occurring 35 to 60 feet below the existing ground levels were tested in situ by pump tests for aquifer characteristics in 1971. In 1977 the shallower aquifers, those to depths of about 35 feet below existing ground level, were tested in situ by pump tests. Visual examination of the undisturbed samples taken during 1977 indicated alternating layers of sand-gravel lenses and silty-clayey layers (see Table 7).

Based on the pump test data and visual review Model 1 is proposed, as shown in Figure 18. A series of aquifers, ranging from a few inches to a few feet in thickness, and consisting of sand-gravel lenses are proposed. These aquifers are formations capable of storing, transmitting, and yielding quantities of ground water. They are separated spatially from each other by clayey layers which have been termed aquicludes. Aquicludes completely exclude movement of ground water. Aquifers have permeabilities ranging from low to very high; whereas the permeability of aquicludes ranges from very low to almost nil. The model visualizes thin to thick lenses of aquitards embedded within these aquicludes. Aquitards may consist of silty sands, clayey sands, or other combinations of silt, peat, sand, and clay. They are rather impervious; i.e., they allow ground water flow at only a very slow rate. The model soil layers are depicted horizontally to facilitate conception of subsurface conditions. In the field, however,



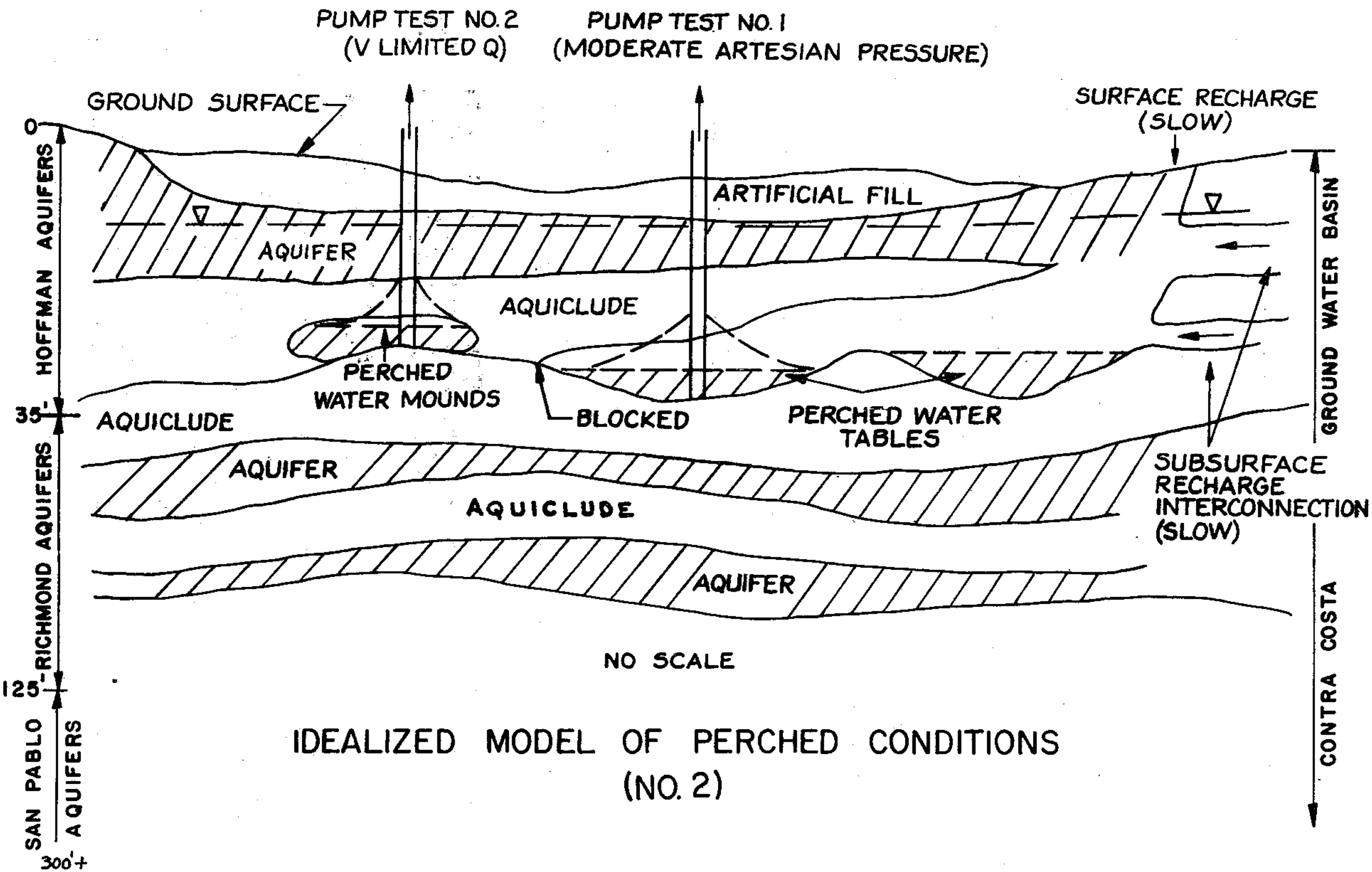
IDEALIZED MODEL OF MULTIPLE AQUIFER SYSTEM
(NO. 1)

FIGURE 18

these layers were found to be distorted, contorted, and twisted in widely varying combinations. Construction activities, soil erosion, earthquakes, uplift of ocean floor, and other geologic activities are a few of the many reasons that can be postulated for these conditions.

Model 2: Idealized Model of Perched Conditions

Whereas Model 1 dealt with the physical makeup of the aquifers, Model 2, described below, is concerned with their elasticity. Model 2 as shown in Figure 19, visualizes the presence of perched water tables and perched water mounds. Perched water tables can be replenished by surface recharge sources or supplied by interconnection to other underground aquifers. The pump test revealed slow recharge is occurring at shallower depths in these perched water tables. These may be under moderate artesian pressure.



IDEALIZED MODEL OF PERCHED CONDITIONS
(NO. 2)

FIGURE 19

DRAWDOWN ANALYSIS

The drawdown analysis for this project entailed a number of water-related procedures, some of which were complicated by the highly variable nature of the sub-soils along the line of the proposed excavation. To facilitate the analysis and enhance the accuracy of the estimates resultant thereof, each of the six areas into which the project was divided was evaluated as a separate entity. The elements of the analysis included the following:

°Development of a cross section typifying each area.

°Determination of the geohydrologic parameters: transmissibility factors and storage coefficients.

°Determination of permeabilities of the sub-soil strata.

°Estimation of pumpage during construction (short term).

°Estimation of pumpage following construction (long term).

Selection of Typical Cross Sections

As previously described, the 12,000-ft. length of the proposed semi-depressed section was divided into 6 areas based on criteria developed from the field and laboratory investigations. To implement the drawdown analysis, the soil/aquifer data from each area was reviewed in detail and a boring was selected from the area that most nearly represented the general subsurface conditions within the area. The profile of the selected boring was then used to develop a typical cross section representation of the area. Cross section locations, station limits, and representative soil borings are listed in Table 10. The sections are shown graphically in Figures 20 through 25.

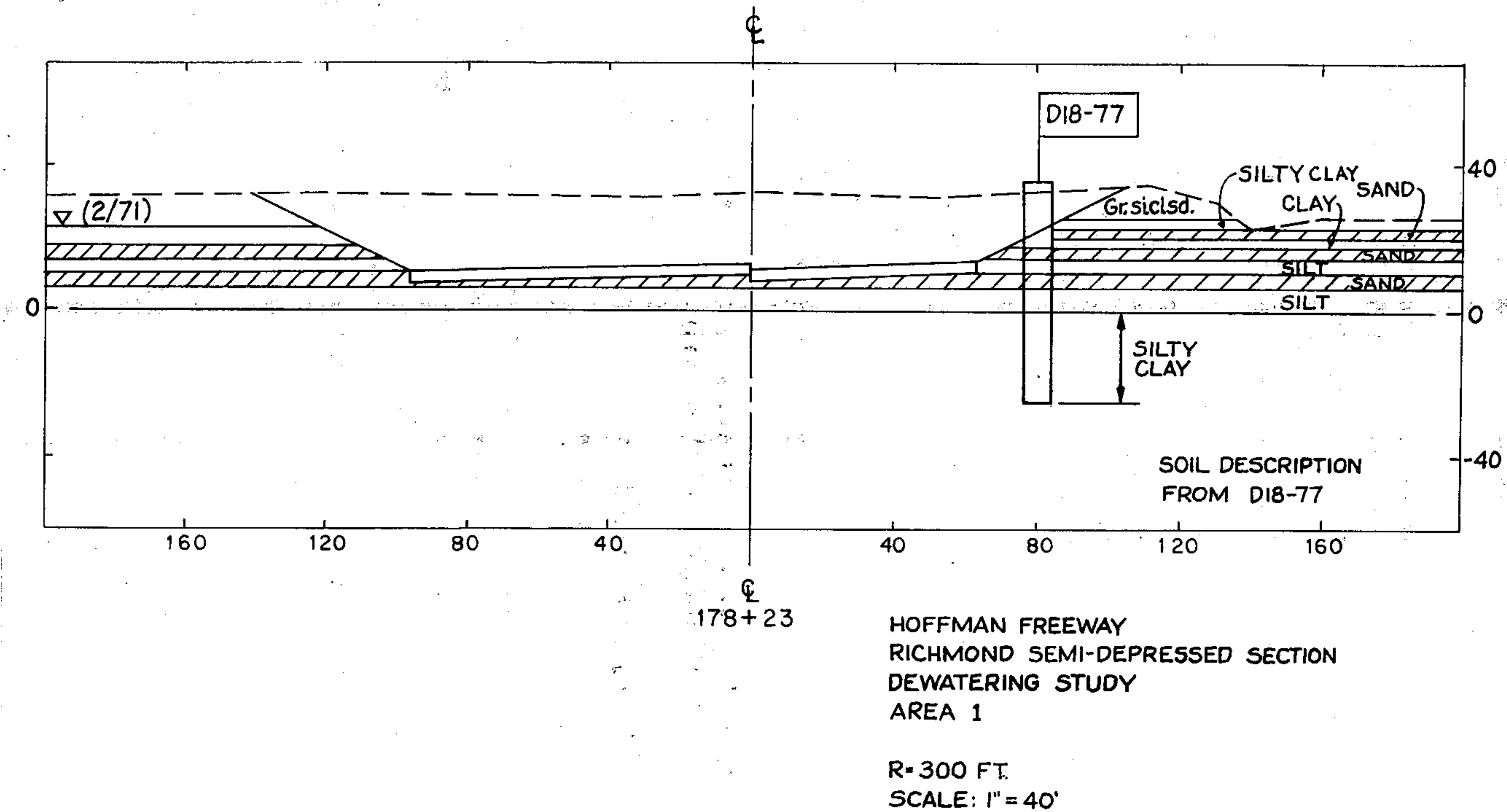
TABLE 10 TYPICAL CROSS SECTIONS AND AREAS REPRESENTED

Area No.	Area Limits	Typical Cross Section Location	Representative Soil Boring
1	Sta. 170 to 184	178+23	D18-77
2	184 to 205	187+60	P10-77
3	205 to 231	220+05	B1-71
4	231 to 252	236+90	D8-77
5	252 to 273	257+62	D13-77
6	273 to 276	272+00	D7-77

Determination of Geohydrological Parameters of the Aquifer

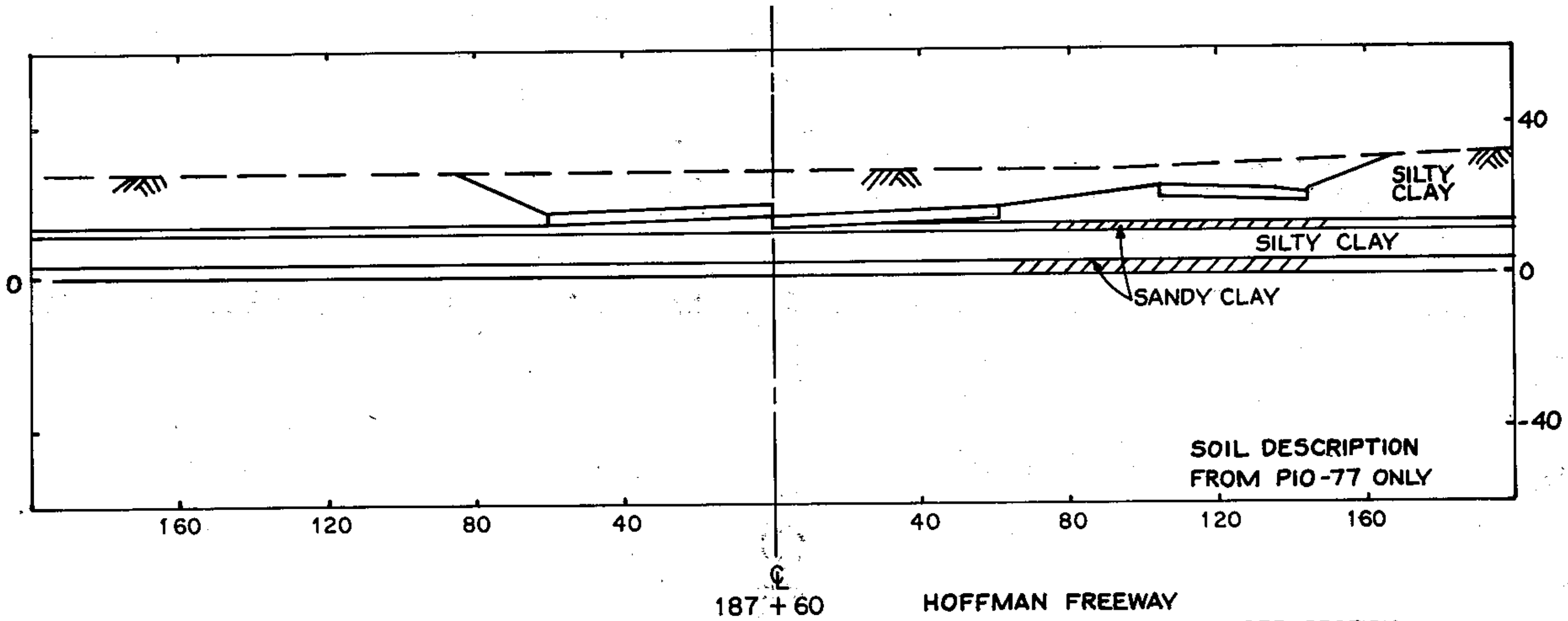
During January 17-23, 1978, full scale pump tests were conducted at the two selected sites. Pump Test 1 was conducted near 11th Street and Hoffman Blvd. Pump Test 2 was performed at Hoffman Blvd. and 30th Street. The well layouts at these sites are shown on Figures 6 through 8.

A pump test program is usually conducted for a continuous period of 72 hours or more. However, both pump tests performed in 1977 were of shorter duration. For Pump Test 1, the pumping operation was discontinued after 23 1/2 hours when the ground water level reached equilibrium. For Pump Test 2, the well was pumped dry at the end of 3 1/2 hours, at which time the pumping operation was terminated.



TYPICAL CROSS-SECTION FOR STA. 170 TO 184

FIGURE 20

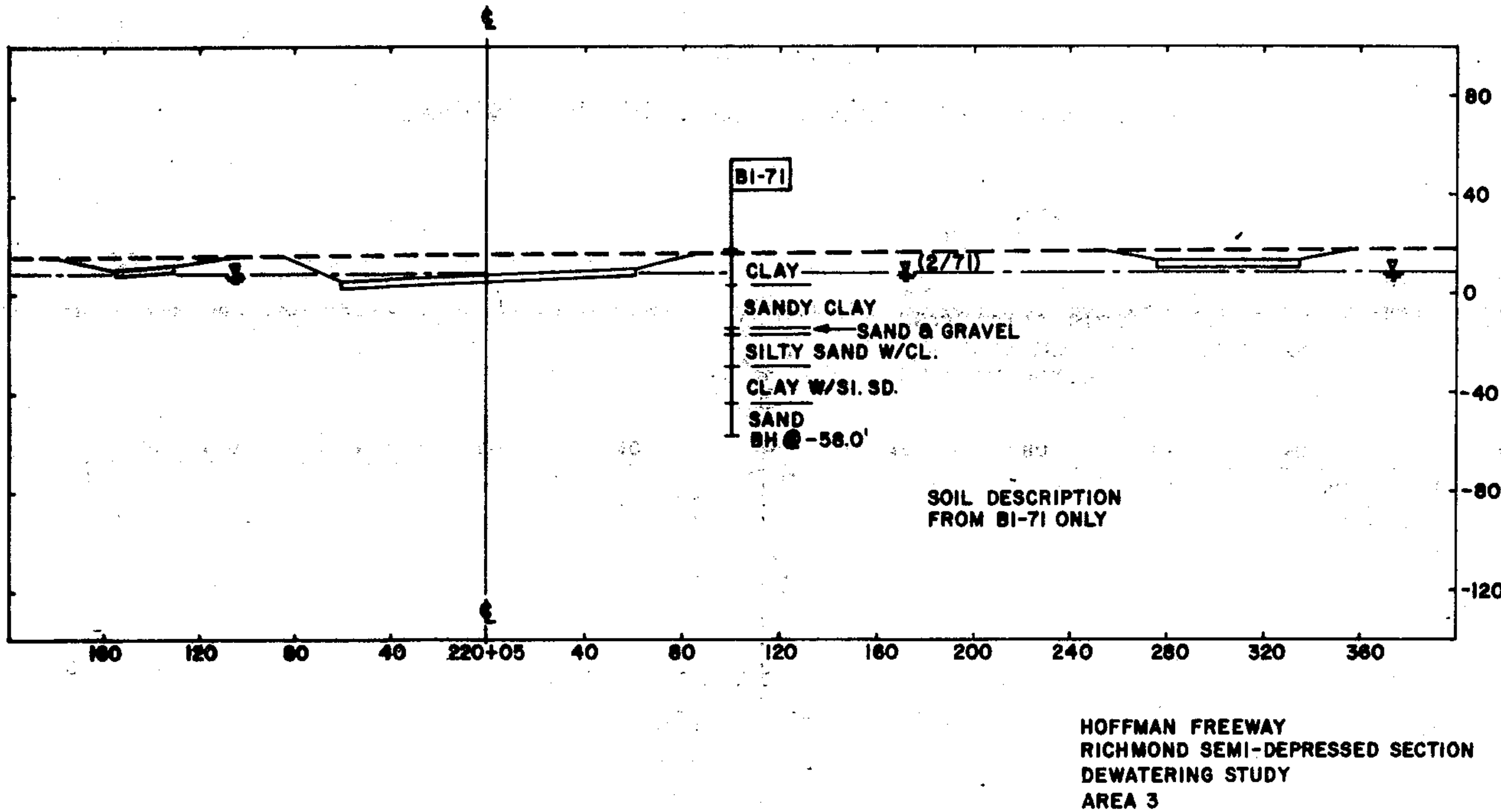


HOFFMAN FREEWAY
RICHMOND SEMI-DEPRESSED SECTION
DEWATERING STUDY
AREA 2

R = 300 FT
SCALE: 1" = 40'

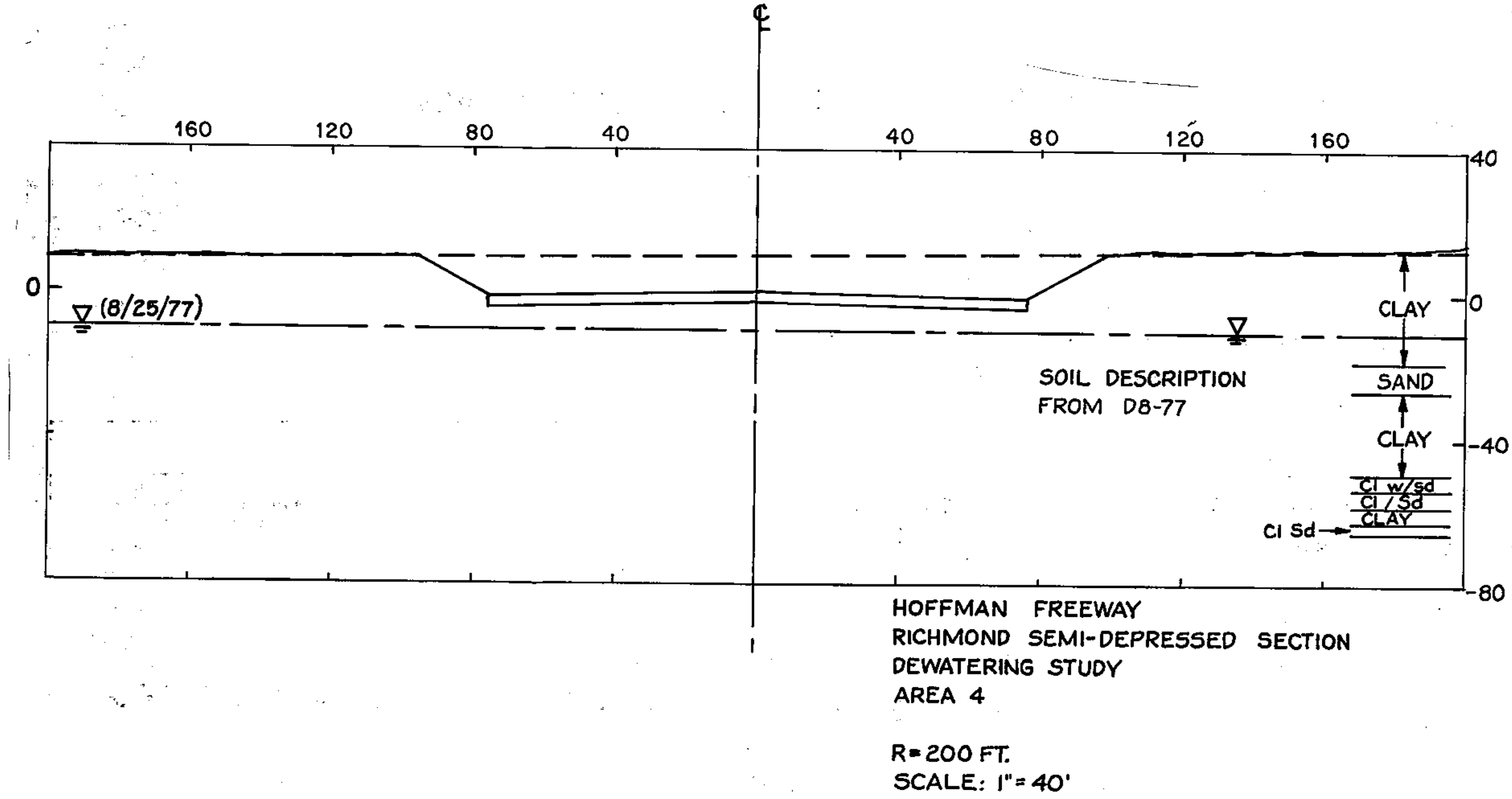
TYPICAL CROSS SECTION FOR STA. 184 TO 205

FIGURE 21



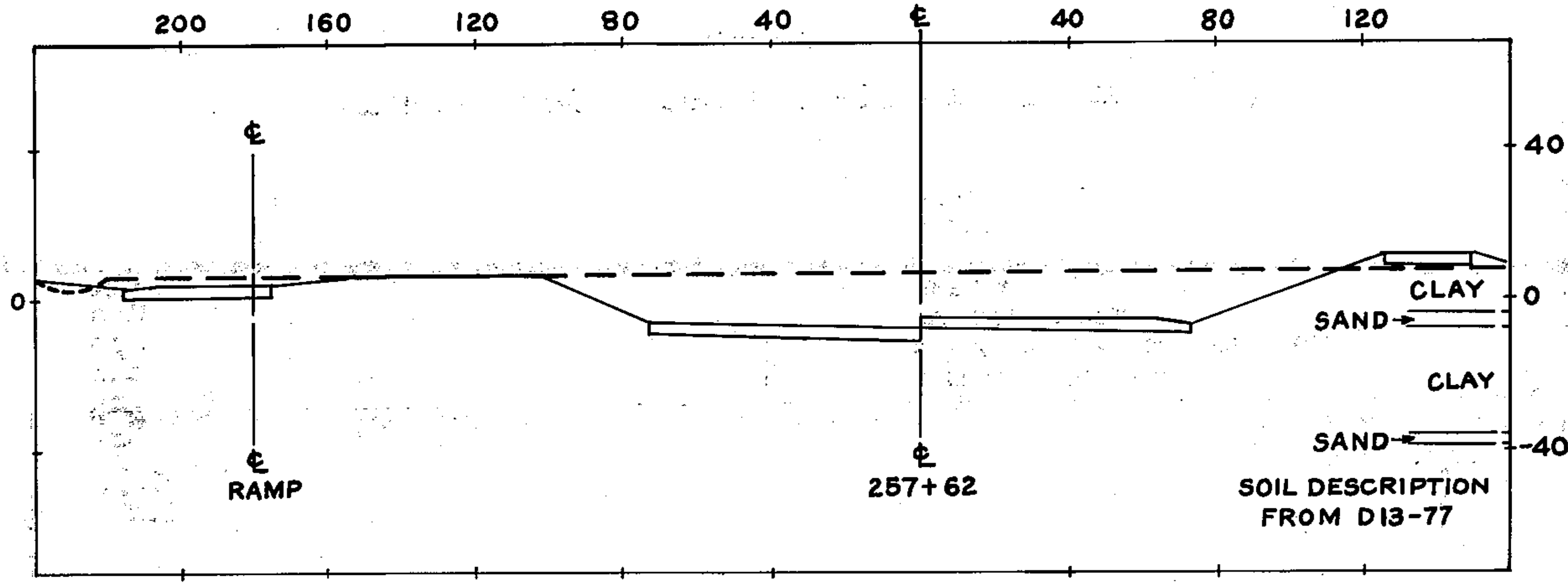
TYPICAL CROSS SECTION FOR STA. 205 TO 231

FIGURE 22



TYPICAL CROSS-SECTION FOR STA. 231 TO 252

FIGURE 23

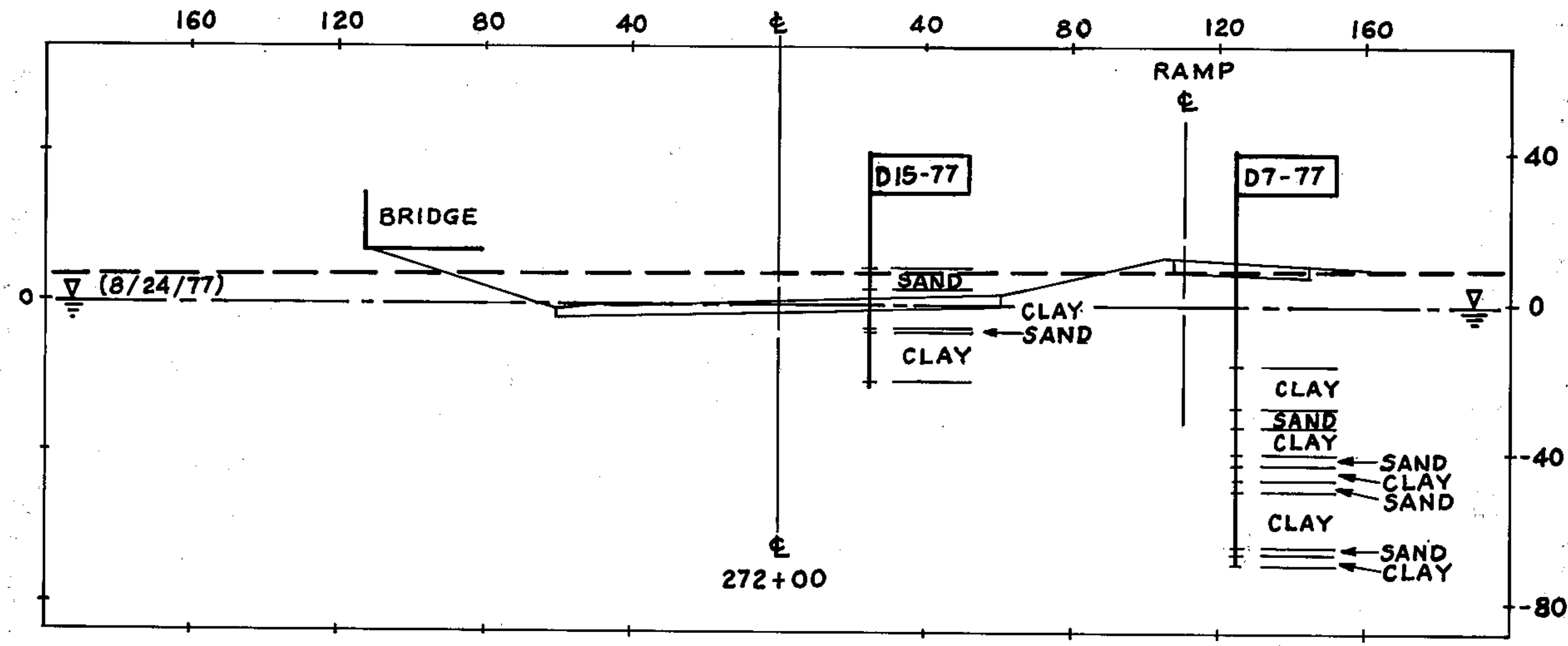


HOFFMAN FREEWAY
RICHMOND SEMI-DEPRESSED SECTION
DEWATERING STUDY
AREA 5

R = 300 FT.
SCALE : 1" = 40'

TYPICAL CROSS-SECTION FOR STA. 252 TO 273

FIGURE 24



HOFFMAN FREEWAY
RICHMOND SEMI-DEPRESSED SECTION
DEWATERING STUDY
AREA 6

R = 200 FT.
SCALE: 1" = 40'

TYPICAL CROSS SECTION FOR STA. 273 TO 276

FIGURE 25

Using the somewhat limited data obtained from Pump Test 1, geohydrological parameters of the aquifer were calculated using the following methods:

a. Theis non-equilibrium method
b. Jacob modified method
c. Recovery method
d. Residual drawdown data method
e. Cone of depression method
f. Thiem's equation

The results obtained from these six methods are summarized in Tables 11 to 13 and discussed separately.

Theis and Jacob Modified Methods

Table 11 is a tabulation of the ground water transmissibility factors and aquifer storage coefficients obtained by the Theis and Jacob methods. These aquifer constants differ markedly from one well to another. However, the values obtained by the two methods compared well with each other.

The transmissibility factors calculated using Theis' method range from 448 gpd per ft to 5247 gpd per ft, for an average of 1568 gpd per ft. The factors obtained with Jacob's modified method range from 396 gpd per ft to 4950 gpd per ft, with an average value of 1615 gpd per ft.

The storage coefficients are somewhat more consistent among the wells. The values range from 1.08×10^{-3} to 6.96×10^{-3} with the Theis equation, and 1.09×10^{-3} to 6.65×10^{-3} using Jacob's modified method. Average values of 3.88×10^{-3} and 3.56×10^{-3} are derived by Theis and Jacob methods, respectively.

TABLE 11, AQUIFER CONSTANTS THEIS AND JACOB METHODS

Well No.	Coeff. of Transmissibility gpd/ft		Storage Coefficient	
	Theis	Jacob	Theis	Jacob
1	463	396	6.75×10^{-3}	6.65×10^{-3}
2	2292	2140	5.28×10^{-3}	5.39×10^{-3}
3	5247	4950	1.64×10^{-3}	2.04×10^{-3}
4	448	480	5.90×10^{-3}	4.03×10^{-3}
5	793	701	3.02×10^{-3}	2.61×10^{-3}
6	806	1320	6.69×10^{-3}	3.84×10^{-3}
7	3163	3046	1.34×10^{-3}	1.58×10^{-3}
8	578	480	4.79×10^{-3}	6.48×10^{-3}
9	916	1056	2.05×10^{-3}	1.90×10^{-3}
10	944	1584	1.08×10^{-3}	1.09×10^{-3}

TABLE 12, AQUIFER CONSTANTS RECOVERY DATA AND RESIDUAL DRAWDOWN METHODS

Well No.	Coeff. of Transmissibility gpd/ft		Storage Coefficient	
	Theis	Jacob	Theis	Jacob
1	312	272	6.46×10^{-3}	-
2	2031	1886	7.95×10^{-3}	-
3	3435	2400	7.83×10^{-3}	-
4	194	172	1.16×10^{-4}	-
5	689	495	3.9×10^{-3}	-
6	2400	2141	1.39×10^{-4}	-
7	7920	7200	5.94×10^{-4}	-
8	388	305	2.01×10^{-4}	-
9	727	720	3.44×10^{-3}	-
10	1584	1015	9.88×10^{-4}	-

TABLE 13 AQUIFER CONSTANTS - CONE OF DEPRESSION METHOD

Well No.		Coeff. of Transmissibility gpd/ft	Storage Coeff.
North Arm	1 2 3	391	7.37×10^{-4}
South Arm	4 5 6 7	434	4.52×10^{-4}
West Arm	8 9	576	3.75×10^{-4}

Recovery and Residual Drawdown Methods

The overall results calculated by using recovery data and residual drawdown data are presented in Table 12. There were generally higher values in transmissibility factors calculated by recovery data, as compared with those derived from the residual drawdown data. The average factors are 1968 gpd per ft with recovery data, and 1663 gpd per ft with residual drawdown data. The storage coefficients as determined by recovery data ranged between 1.16×10^{-4} and 7.95×10^{-3}.

Cone of Depression Method

Table 13 summarises all the aquifer constants obtained from three arms of the observation wells using the cone of depression method. As can be seen, these aquifer constants have relatively low values compared with those obtained by the other methods. The average transmissibility factor is 467 gpd per ft and the average storage coefficient is 5.21×10^{-4}.

Thiem's Equation

Using Thiem's equation, permeability values were calculated. Since the transmissibility factor is the rate at which water will flow through a vertical strip of one-foot wide aquifer extending through the entire water-bearing stratum (H) at a hydraulic gradient (i) of 1.00, the coefficient of permeability (k) is related to the transmissibility factor (T). This relationship can be expressed as:

$$T = kH$$

With an assumed 9-foot water-bearing stratum (H) and the calculated permeability (k) for various pairs of wells, transmissibility factors were determined, as shown in Table 14.

TABLE 14 TRANSMISSIBILITY FACTORS USING THIEM'S EQUATION

Well No.	Transmissibility (T) gpd/ft.	Well No.	Transmissibility (T) gpd/ft.
1-2	254	5-6	412
1-3	340	5-7	664
2-3	1591	6-7	1472
4-5	297	8-9	463
4-6	336	8-10	518
4-7	409	9-10	715

Estimation of Geotechnical Parameter

The geotechnical parameter, namely, in situ permeability of the subsoil strata, was determined using (a), the six methods described earlier; and (b), data from recovery tests performed by District 04.

Using Six Methods

Based upon the constants determined by the first five of the six methods mentioned earlier; and assuming a 9-foot water-bearing formation, in situ coefficients of permeability were calculated, as shown in Table 15.

TABLE 15 ESTIMATE OF IN SITU PERMEABILITY VALUES FROM PUMP TEST DATA

Well No.	Coefficients of Permeability, ft/day (1) Theis'	(2) Jacob's	(3) Cone of Depression	(4) Recovery Data	(5) Residual Drawdown
1	6.8	5.9		4.7	4.1
2	34.1	31.8	5.9	30.2	28.1
3	77.9	73.5		51.1	35.7
4	6.7	7.1		2.9	2.5
5	11.8	10.4	6.4	10.3	7.4
6	12.0	19.7		35.7	31.8
7	46.9	45.2		117.7	107.0
8	8.6	7.1		5.7	4.5
9	13.6	15.6	8.6	10.8	10.7
10	14.0	23.5		23.5	15.1
Avg.	17.4	23.9	7	29.3	24.7

Additional calculations related to permeability were performed using the Thiem equation. In order to determine permeability from various pumped wells the following assumptions were made: (a) a pumping rate of 4330 gpd to effect a steady-state condition; and (b) the pumping well penetrates a 9-foot water-bearing formation.

Drawdowns in any two observation wells at different distances from the pump well were determined. From these data, the coefficients of permeability were estimated and are summarized in Table 16.

TABLE 16 COEFFICIENTS OF PERMEABILITY DETERMINED BY THIEM'S EQUATION

Well Number	Permeability (k) fpd	Well Number	Permeability (k) fpd
1-2	4	5-6	6
1-3	5	5-7	10
2-3	24	6-7	22
4-5	4	8-9	7
4-6	5	8-10	8
4-7	6	9-10	11

Using Recovery Tests

During September 1977, recovery tests using 18-inch diameter bored holes were conducted by District 04 Materials personnel to determined recovery rates of ground water for various locations along the proposed depressed section. Coefficients of permeability were then determined from the recovery test data. The results are presented in Table 17.

TABLE 17 COEFFICIENTS OF PERMEABILITY CALCULATED FROM RECOVERY TESTS

Area	Boring No.	Boring Location	Permeability (k), fpd
1	--	--	*8.0
2	P10	Sta 187+80	8.3 to 4.2
3	P8, P7	Sta 208+00	8.0 to 2.0
4	P5	Sta 239+40	5.0
	P4	Sta 249+80	6.8
5	P3	Sta 259+00	14.2, *8.0
6	P2	Sta 272+00	4.8
	P13	Sta 274+15	8.5
	P14, P15	Sta 276+00	4.0

*Assumed values used in analysis.

These estimated permeability values ranged from 2.0 fpd to 14.2 fpd, with an average value of 6.6 fpd. These values are generally lower than the values derived from pump test data. However, the average value is consistent with the values obtained by the cone of depression method based on the pump test data.

Discussion of Aquifer Characteristics

Thickness and Location

A generalized soil profile could not be constructed to delineate soil layering along this project because of the complexity of the soil strata involved. A complex multiple-aquifer system exists within the shallower horizons relevant to this study. Two idealized models, considered representative of this system, are presented in Figures 18 and 19. At least three definable aquifers exist in localized areas within depths relevant to this project. Their thicknesses and positions relative to each other, however, vary along the line of proposed excavation.

Geohydrologic Parameters

The average values for transmissibility factors and storage coefficients as calculated by various methods are presented in Table 18.

TABLE 18 GEOHYDROLOGIC PARAMETERS

Method	Average Values of T gpd/ft	Average Values of S
Theis non equilibrium	1568	0.0038
Jacob modified	1615	0.0036
Recovery	1968	0.0032
Residual drawdown	1663	- - -
Cone of depression	467	0.00052
Thiem's equation	623	- - -

Geotechnical Parameter

The aquifers have permeability characteristics ranging from low (4 fpd) to very high (118 fpd), Table 12.

Careful review of the information provided by all borings made in the project area since 1970 indicates that there are a large number of perched water tables with moderately limited storage of drainable water or perched mounds of water with extremely limited storage capacities (Figure 19).

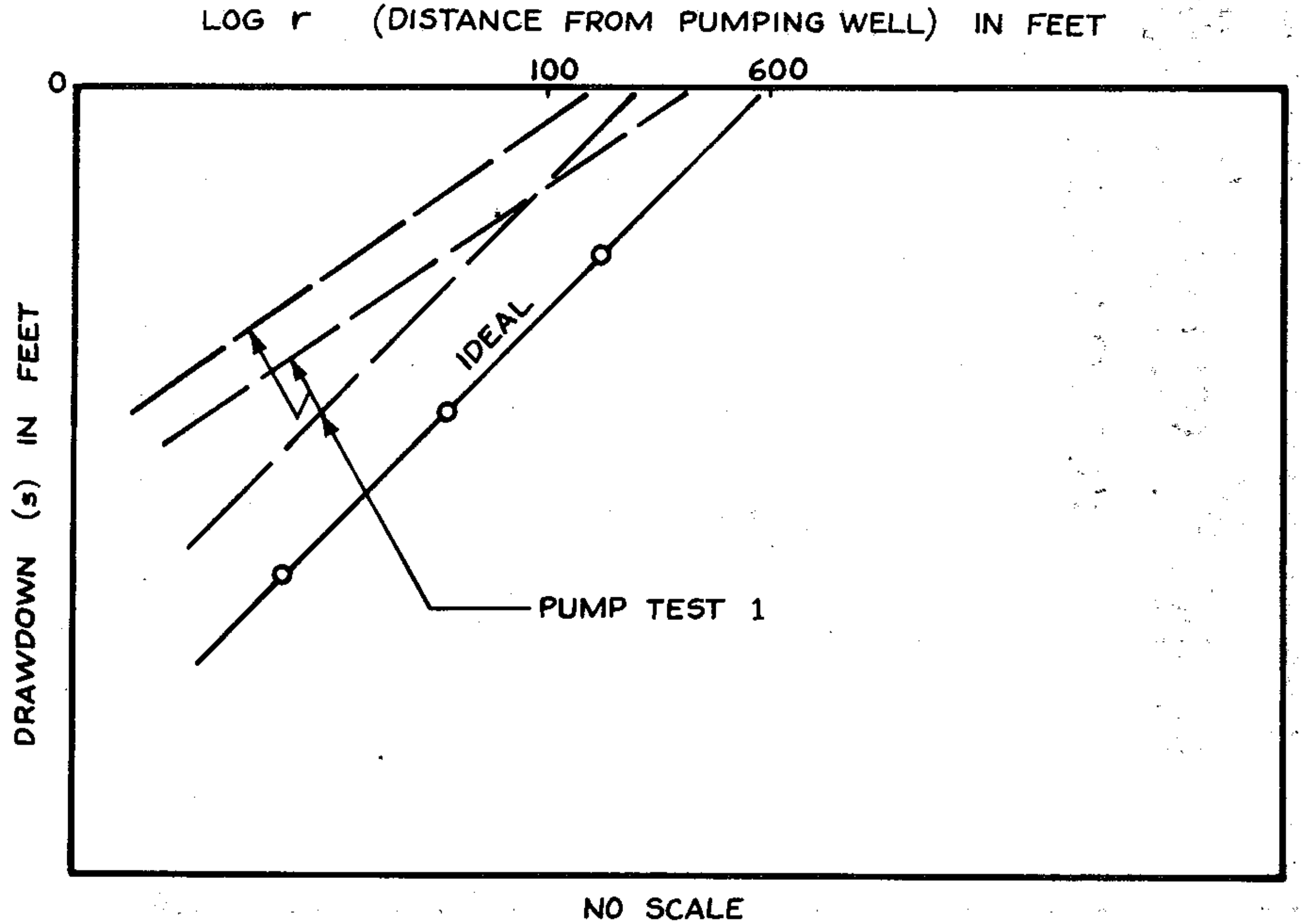
The transmissibility factor (T) indicates the quantity of water that will move through a given formation in a given period of time. These values ranged widely, from a low of 194 gpd/ft (Table 12) to a high of 5247 gpd/ft (Table 11). If a formation has a transmissibility of less than 1000 gpd/ft, it can supply only enough water for a domestic well, or similar usage. If the transmissibility factor is of the order of 10,000 or more, yield can be adequate for industrial, municipal, or irrigation purposes. The coefficient of storage indicates how much water can be moved by pumping, or draining. At Richmond these values also ranged widely, from a low of 0.000116 to a high of 0.00795 (Table 12). Such values indicate the existence of artesian conditions. Although artesian conditions are indicated by theoretical considerations, water table conditions were observed in the field. It is, therefore, concluded that both artesian and water table conditions exist in the project area. Artesian conditions indicate the existence of confined aquifers and water table conditions indicate the presence of unconfined aquifers.

Recharging Ability

During the analysis of Pump Test 1 data, certain discernible patterns were observed. The plot of drawdown (s) versus logarithm of distance from pumping well (r) yielded patterns as shown in Figure 26. For an ideal condition, i.e., where incoming and outgoing flow quantities are equal, the ideal line will pass through 600 feet. The data plotted, however, with three lines for the three arms, pass through the range 150 to 350 feet. This indicates that recharge was taking place during pumping operations. The plot, drawdown (s) versus logarithm of time of pumping (t), produced patterns shown in Figure 27. This also indicates that the water bearing formation is being recharged. The plot shows residual drawdown (s') versus logarithm of (t/t'), where t' is the time since pumping was stopped, and t is the time of pumping. The data from Pump Test 1 consistently passed through log t/t' = 2 (about), as shown in Figure 28. The indication is that recharge was taking place at a rather slow rate. During this study it was not possible to identify sources of recharge. It is believed that some of the aquifers may be interconnected.

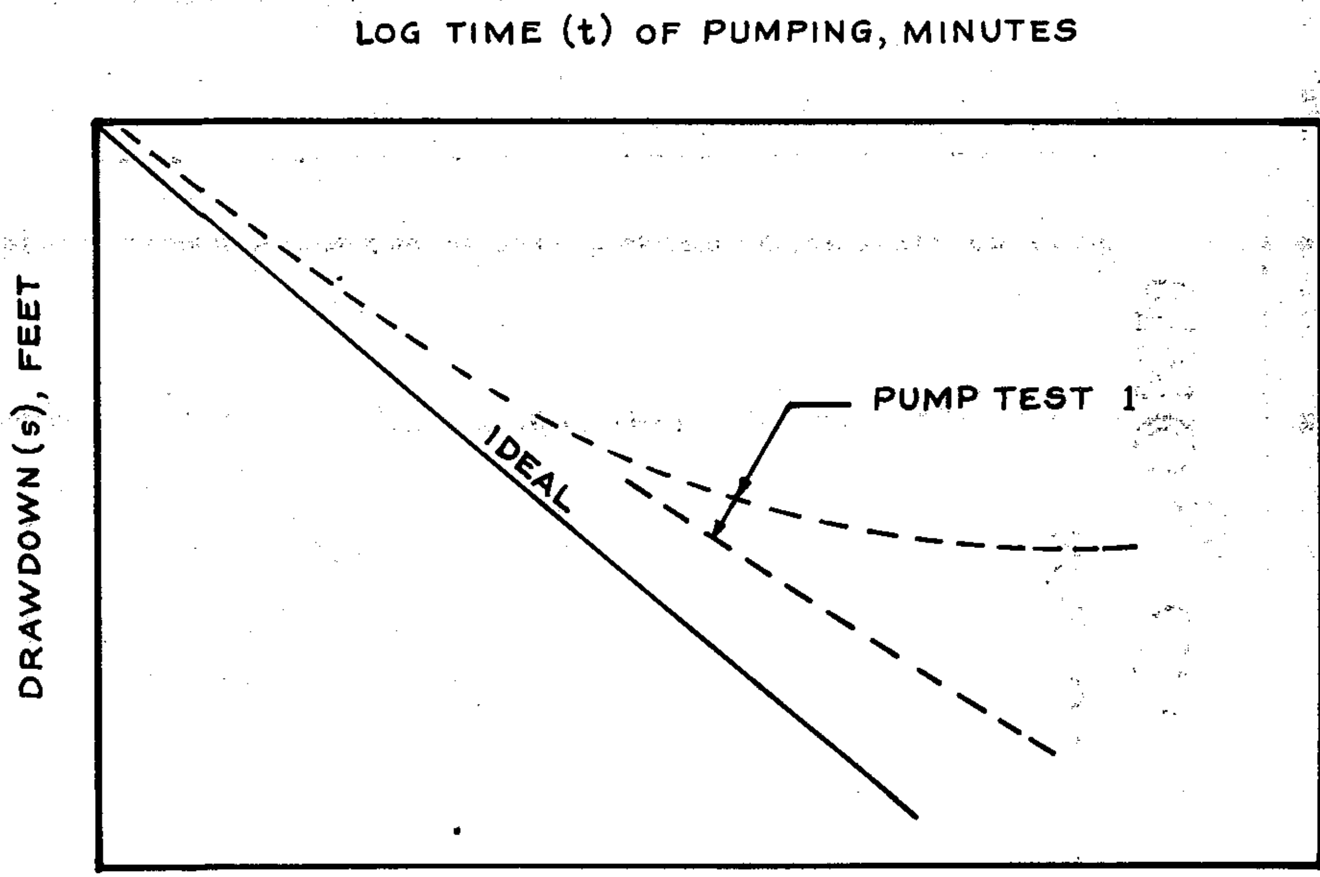
Estimate of Total Pumpage During Construction

The total pumpage was calculated assuming two rows of deep wells as shown in Figure 29. The spacing of these wells was 50 ft. The pumping rate in gallons per minute was based on the pumping rates used in the two most recent pump tests and the desired average values of drawdown in each of the six areas previously described. These values are tabulated in Table 19. From this table it is estimated that a maximum pumpage of 3.1×10^6 gpd needs to be handled during construction to keep the water tables depressed to desired levels.



AQUIFER CHARACTERISTICS BY s VS LOG r PLOTS

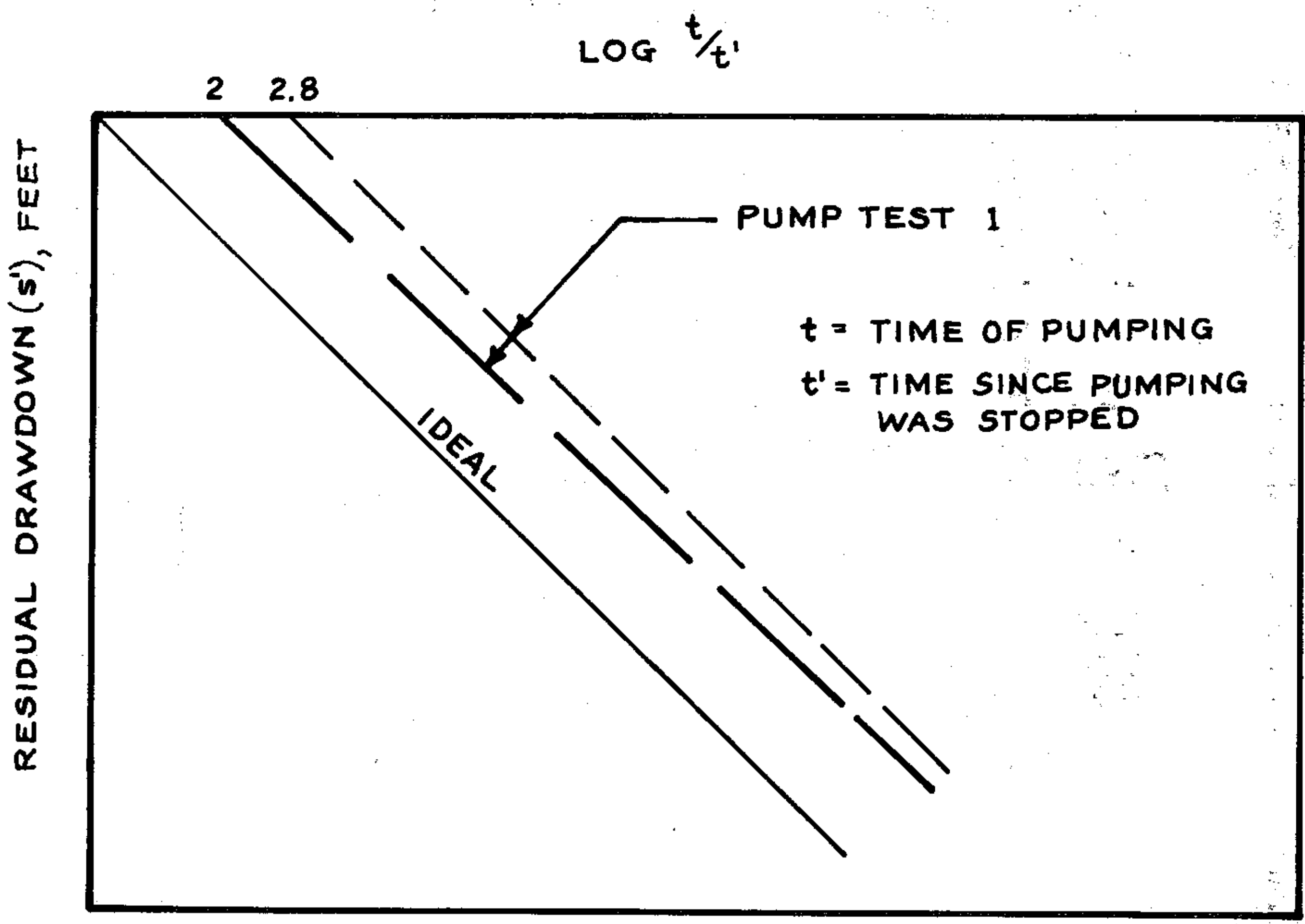
FIGURE 26



AQUIFER CHARACTERISTICS BY s VS LOG t PLOTS

NO SCALE

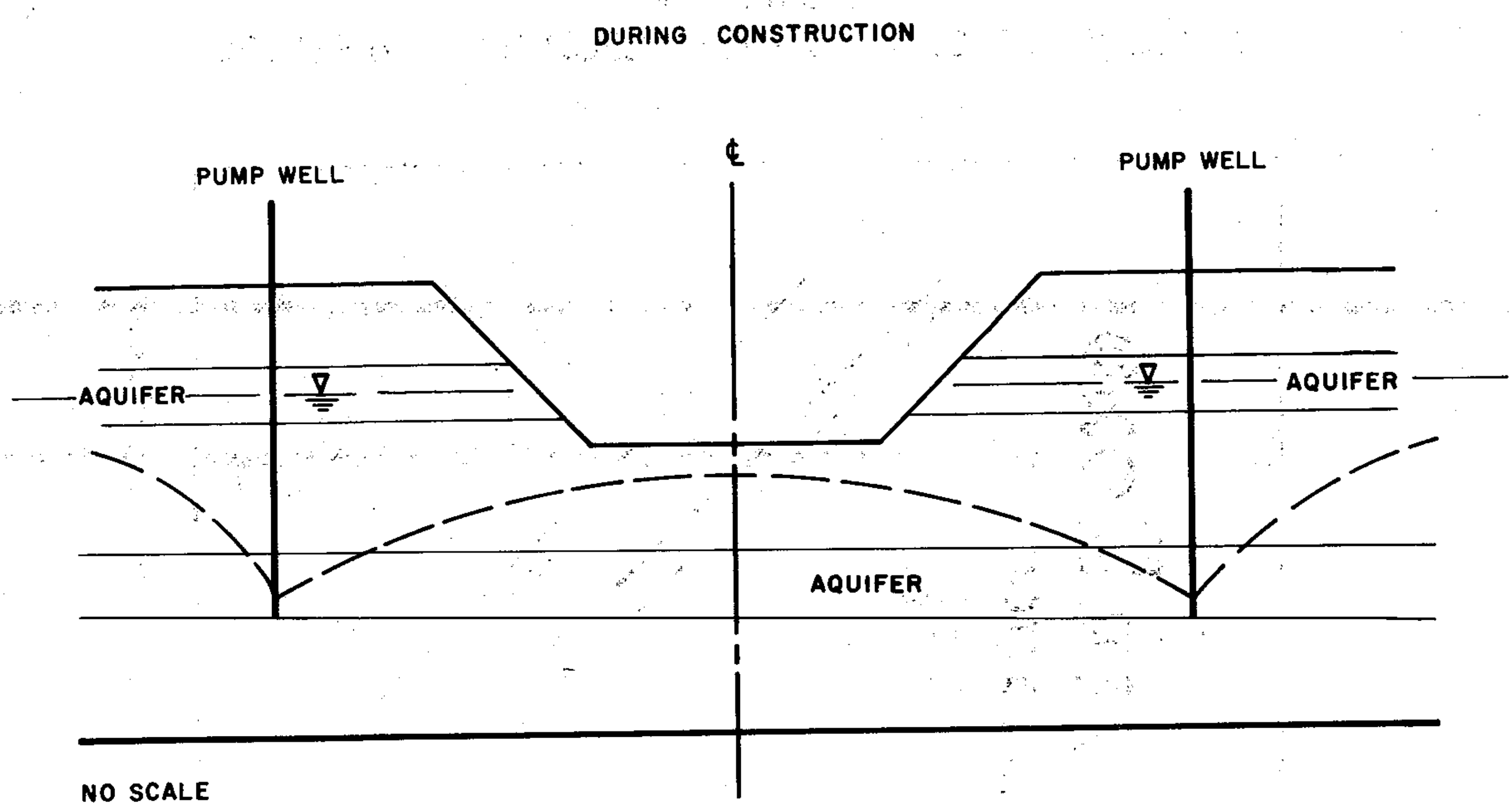
FIGURE 27



AQUIFER CHARACTERISTICS BY s' VS LOG t/t' PLOTS

NO SCALE

FIGURE 28



HOFFMAN FREEWAY
SEMI-DEPRESSED SECTION

ESTIMATE OF TOTAL PUMPAGE

FIGURE 29

TABLE 19, PUMPAGE PER DAY DURING CONSTRUCTION

Area	Rows	Length (ft)	Spacing (ft)	No. of Wells	Pumping Rate gpm	Pumpage gpd
1	2	1400	50	58	6	501,120
2	2	2100	50	86	6	743,200
3	2, 1	2600(?)	50	106	4 3	610,560
4	2	2100	50	86	5	525,000
5	2	1800	50	74	5	532,800
6	2	600	50	26	4	149,760

Total Pumpage = 3.1×10^6 gpd

TABLE 20, TOTAL DRAINAGE QUANTITY AFTER CONSTRUCTION
- DUE TO GRAVITY SEEPAGE

Area	k fpd	q_1/LF gpd (gravity seepage)	L (ft)	Q_1 gpd
1	8	23	1400	32,200
2	4	17	2100	35,700
3	2	4	2600	10,400 19,400*
4	5	26	2100	54,600
5	7	20	1800	36,000
6	6	15	600	9,000

(For one side) Total Q_1 = 177,900 gpd
or 186,900 gpd

*Includes side street or on ramps.

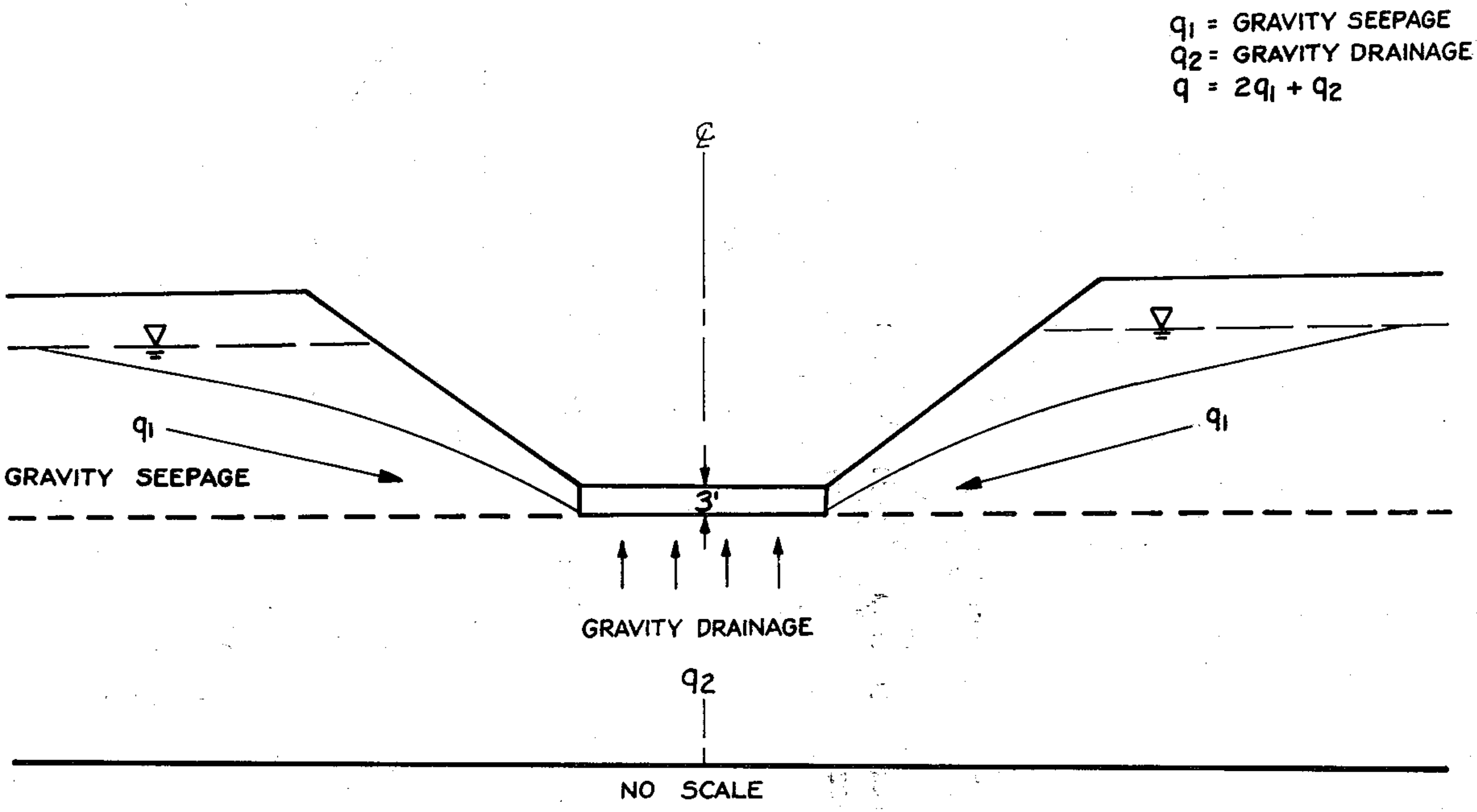
Estimate of Total Pumpage After Construction

The theory of gravity drainage was used to calculate the quantity of water to be removed. In these calculations deep wells or well points were not considered as a solution because of the long term expenses involved. In this theory the total quantity to be drained by any system is considered to consist of gravity seepage (q_1) and gravity drainage (q_2) as shown in Figure 30. Referring to Tables 20 through 22, from the known values of q_1 and q_2, the values of q_g (the overall quantity due to drainage per square foot of permeable subsurface drainage area in gallons per day) were derived. From these values of q_g the total quantity to be drained by gravity is estimated at a maximum value of about 1.4×10^6gpd.

Discussion on Total Pumpage

During construction it may be necessary to remove from 2.5×10^6 gallons per day to 3.5×10^6 gallons per day. These estimates are based on Pump Test 1, which was made in Area 5. The project construction period is estimated at between 2 years and 2-1/2 years. Hence, during this period the total pumpage in some reaches of the project could be substantially lower than at the start of construction. Also, it should be remembered that the actual pumpage is a function of depth of penetration of wells; number of aquicludes, sand lenses, or aquifers encountered; and extent and/or continuity of these water bearing formations.

After construction the total pumpage would be substantially reduced to about 1.4 million gallons per day if gravity drainage is adopted (Tables 20, 21 and 22).



RICHMOND SEMI-DEPRESSED SECTION
TYPICAL CROSS SECTION

AFTER CONSTRUCTION
THEORY OF GRAVITY DRAINAGE

FIGURE 30

TABLE 21, TOTAL DRAINAGE QUANTITY AFTER CONSTRUCTION - DUE TO GRAVITY DRAINAGE

Area	L (ft)	W (ft)	Area (sq ft)	Gravity Drainage		
				q_2 gpd/Lf	q_g gpd/ft^2	Q_2 gpd
1	1400	160	224,000	39	0.50	112,000
2	2100	122 40	256,200	75	1.25 0.50(est.)	320,250 42,000
3	2600	122 58	317,200	22	0.44 0.22(est.)	126,880 30,160
4	2100	146	306,600	45	0.6	183,960
5	1800	146	262,800	30	0.4	105,120
6	600	122	73,200	103	1.7	124,440

Total 1,044,810 gpd
or
972,650 gpd

TABLE 22, TOTAL DRAINAGE QUANTITY AFTER CONSTRUCTION (Gravity Seepage and Gravity Drainage)

Gravity Seepage (2 Q_1)	355,800 gpd	373,800 gpd
Gravity Drainage (Q_2)	972,650 gpd	1,044,810 gpd
Total Quantity	1.33×10^6 gpd or	1.42×10^6 gpd

METHODS OF DEWATERING

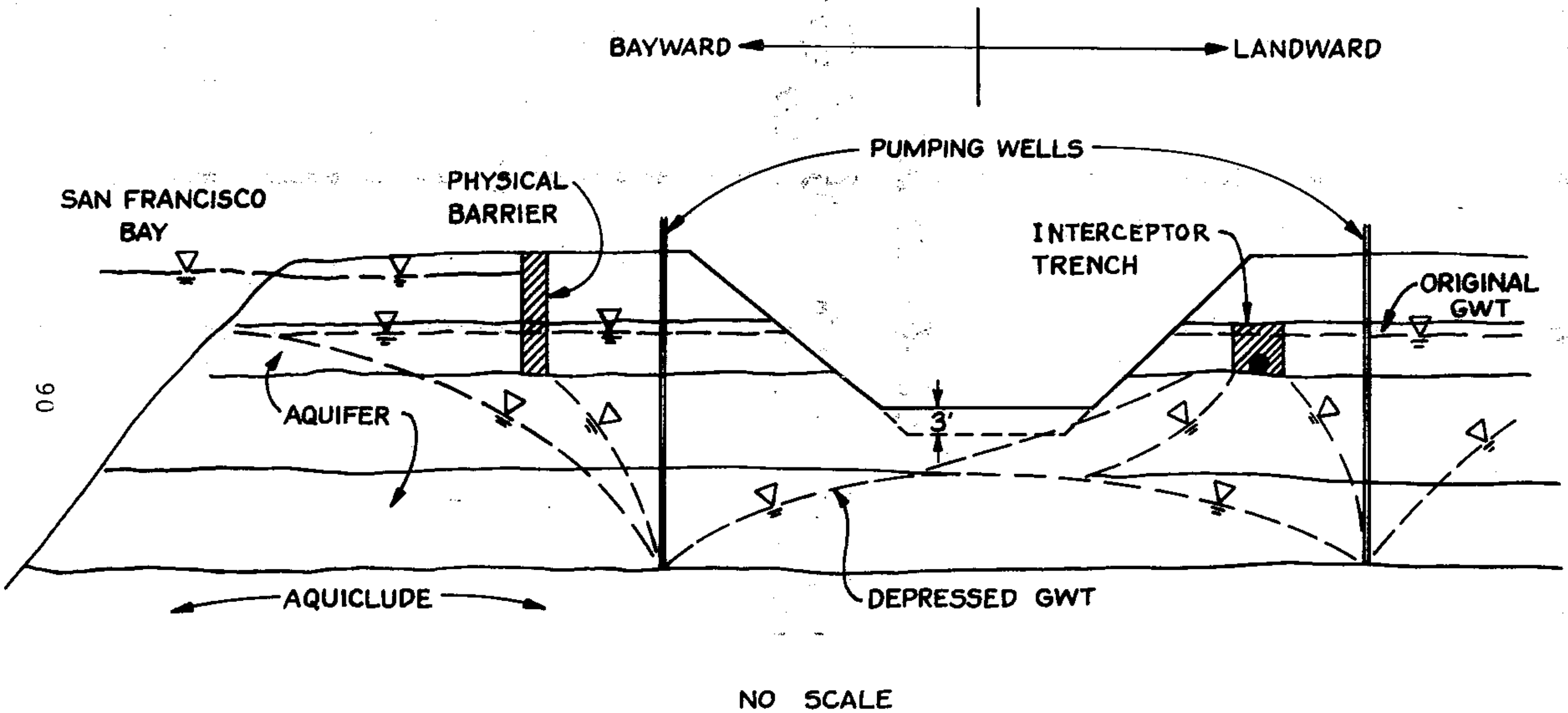
The models here proposed for dewatering the semi-depressed section consider 'during construction' and 'after construction' conditions for the project. The concepts embodied in the models can, in application, be employed singly, or in combination, as required.

During Construction

Explanation of Model

The various elements of a comprehensive dewatering system are depicted in the during construction model diagram, shown as Figure 31. It consists of three principal components: deep wells, interceptor trenches, and physical barriers.

The total pumpage during construction is based on installation of a row of deep wells along each side of the semi-depressed section. The depths of these wells should be decided based on the soil profile at each location. A general guideline can be stated as follows: The bottoms of these deep wells should be at least 10 feet below the lowest point in any cross section. This would allow at least 5 feet of water standing in the well after it is drawn down 5 feet below the lowest point in any cross section. This 5 feet of standing water is necessary for submerging the pump as well as to allow for any silting. If the criterion of 10 feet below the lowest point could not be followed in the field because of entering a deeper aquifer, then much shallower depths



METHODS OF DEWATERING DURING CONSTRUCTION - MODEL

FIGURE 31

have to be chosen for these wells and corresponding increases in pumping rates may have to be adopted to keep the ground water table depressed to desirable levels.

The second component of this model consists of the interceptor trenches. These should be excavated to the bottom of the "first aquifer". A nominal width of 2 ft is suggested for these trenches. Their depths will vary, as the trench bottoms should conform to the elevation of the base of the "first" aquifer, as determined during excavation. It is anticipated that this will average about 10 ft below ground surface. These interceptor trenches may be back-filled wij suitable sand-gravel material enclosed within filter fabric and should contain perforated drain pipes laid along the bottom of the trench.

The third component is the physical barrier, which can consist of a narrow (2 ft wide) deep trench, cut to the bottom of the "first aquifer" and backfilled with a slurry of drilling mud and water.

Alternate Methods

Four alternative methods deemed suitable for during construction dewatering are proposed here in outline.

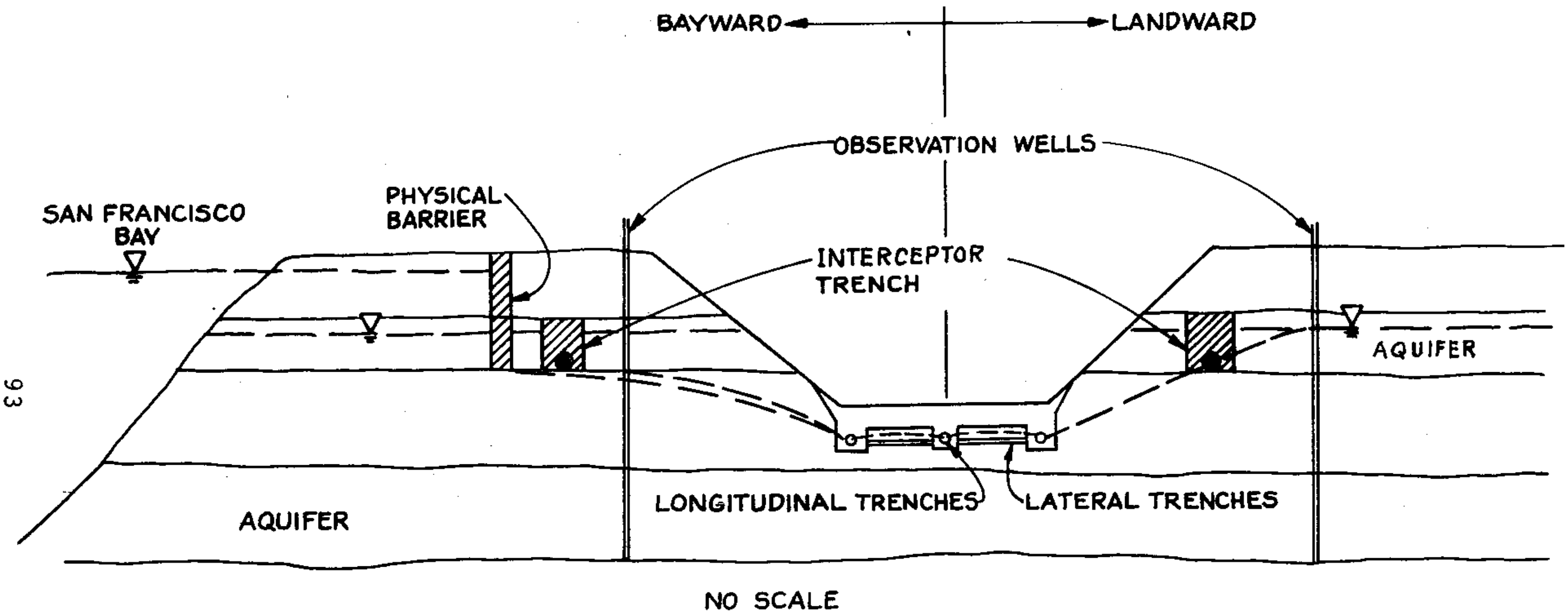
The first alternate that may be considered is simply a system of deep wells alone. This would entail two rows, one on either side of the freeway, spaced at 100-foot intervals. The depths, as earlier stated, would have to be decided during construction. In general, the feasibility of this method is questionable as evidenced by the short duration of Pump Test 2. However, at either end of this project, where there is a possibility of hitting two or more mini-aquifiers, this method might prove effective.

A second alternate could consist of combining an interceptor trench on the inland side (excavated as mentioned above) and a physical barrier on the bayward side. These trenches could be installed as the primary excavation proceeds. The physical barrier can be effected through the use of a bentonite mud slurry. The interceptor trench would divert ground water from the inland side while the physical barrier on the bayward side can prevent any gravity seepage into the excavation. Gravity drainage accumulations can be collected and removed by means of sumps and pumps.

A third alternate would utilize two interceptor trenches, one on either side of the freeway (both being placed in the "first aquifer"), plus a physical barrier on the bayward side. The placement of an interceptor trench on the bayward side between the physical barrier and the freeway would assure effective removal of seepage in that sector. This latter arrangement is recommended for the project as a prime viable method.

Gravity drainage from the bottom of the excavation can be handled by a system of sumps and pumps. The physical barrier may be required to extend from Station 202 to 272.

A fourth alternate is a combination of Alternates 1 and 3. The use of deep wells is recommended if the quantities of water to be removed will exceed the capacity of the interceptor trenches. The areas where this alternate could be used to advantage would be at the ends of the depressed section. The length of such areas would have to be decided during construction as excavation proceeds.



METHODS OF DEWATERING AFTER CONSTRUCTION - MODEL

FIGURE 32

After Construction

Explanation of Model

The various concepts involved in the after construction condition are presented in Figure 32. They consist of five main components: namely, observation wells, interceptor trenches, physical barrier, longitudinal and lateral trenches, and collection stations.

The selection of drainage components on a long term basis (operational phase) would naturally hinge on the dewatering components adopted during the construction phase. The related concepts will be commented on at this time.

Suggested Components

The adoption of observation wells as part of the long term operational phase is highly recommended. These wells could be used for periodic monitoring of water elevations, and for sampling to determine if any sea water intrusion is occurring. If properly installed they might also be used to augment drinking water supplies in the event of a natural disaster (earthquake), or damage to existing potable water conduits. These wells would be drilled to specified depths at specified locations. If deep wells are used as part of the dewatering system during construction they can also function as observation wells. The second and third components (interceptor trenches and physical barrier) have been discussed earlier.

In the section of this report related to sea water intrusion the long term effects of adopting a physical barrier on the bayward side are discussed. While the physical barrier on the bayward side will be able to prevent gravity seepage, gravity drainage from below the bottom of the semi-depressed section will still flow into the fourth component, the longitudinal and lateral trenches. The fourth component may be referred to as a "sub-surface drainage system". It is recommended that filter fabric be used as a blanket over the entire surface of the exposed trenches and pavement areas. The longitudinal and lateral trenches should contain perforated pipes to carry the drainage water.

The fifth component involves placement of collection stations for drained water. Two locations are recommended for collection stations: one in the vicinity of Station 225, and the other near Station 262. The sea water intrusion studies (discussed elsewhere in this report) indicate that a portion of the semi-depressed section (east end) might still have fresh water which can be collected at Station 225; the contaminated water can be collected at Station 262.

Comments on Pumpage

The total pumpage during construction will vary, depending upon the method of dewatering adopted. The volume of water to be removed during construction as determined by theoretical calculations should be considered as an upper limit. The actual total pumpage quantities are expected to be much less than estimated.

In the after construction condition also, the total pumpage based on the calculated estimates is considered the upper limit. Here, again, the components adopted for the long term drainage system will influence volume. Potential uses of drainage water based on its present condition will be discussed in the section, Ground Water Quality.

SETTLEMENT ANALYSIS

In this section a summary of the findings of the previous report (1971) is reviewed initially. After stating the basis of the present study a detailed presentation of settlement estimates for each area is given. This section concludes with comments on settlement/subsidence effects due to long term removal of water from the project area.

Findings of 1971 Study

Based on a large number of consolidation tests conducted for the 1971 study(4) detailed settlement analyses were performed and the conclusions of the 1971 report pertinent to the present ground water investigation are quoted below:

> "Most soil strata between Stations 179 and 230 are comprised of significant amounts of sand. Some shallow and deep clayey layers exist and Bridge Department soil borings in the immediate area show layers of stiff gray clay and silty clay underlying dense sand and gravel. The sand and gravel layers contain some clay binder.
>
> "Analysis of consolidation test data from sample borings B-1 and B-3 indicate that about 2-1/2 inches of settlement can be anticipated under a drawdown of 24 feet. The settlement rate in the clay will be relatively slow. Residual settlement can be expected in the clay strata following construction for a period of one and one-half to two years. The amounts involved are probably in the order of 1-1/2 to 2 inches. We believe these amounts are tolerable. Settlements of 2.5 inches, 2.0 inches, 1.6 inches, 1.4 inches, 0.75 inch and 0.5 inch were calculated for distances of 0, 50, 100, 150, 200, and 250 feet from the pumping station, respectively. The maximum differential settlement is expected to be approximately one-half inch within any 50-foot interval; but if extensive dewatering occurs, differential settlement will be much less."

"The soils between Stations 230 and 290 are predominantly fine-grained. Although a limited amount of peat was noted in the upper 13- to 15-foot clayey layer the soil is firm to stiff and only slightly compressible. Our laboratory test data show that the ultimate settlement at the pump due to 22 feet of drawdown will be in the neighborhood of 2-1/2 inches. Differential settlement will be less than 1/2 inch in a 50-foot interval. The settlement will be mostly complete during construction and no special treatment is recommended. Since very little ground water recharge is expected from the bay side of the excavation, a general settlement of 1.5 to 2.0 inches is expected throughout the industrial complex southwest of the excavation. It is recommended that a survey be conducted to see if this will interfere with production in any way."

Note: It should be noted that the idealized soil profile constructed for the 1971 study, and upon which the above quoted conclusions are based, was developed from a limited number of borings and differs markedly from the model profiles developed during this investigation, which are considered more representative of actual subsurface conditions.

Settlement Estimates

The estimates for settlement presented in this report are based on data developed during the 1971 study. For that study, laboratory consolidation tests were performed on samples from four borings (B-1 through B-4). In view of the limited amount of information available with regard to soil compressibility along the length of the project a considerable degree of interpolation has been required in the settlement evaluation process. For this reason the settlement predictions presented here should be considered as indications only.

For the purpose of estimating settlement that will occur as a result of ground water drawdown during construction a hypothetical ground water profile was assumed based on pump test data. Settlement was then calculated for each of six specified sub-areas along the proposed depressed section. A summary of these calculations is tabulated on Table 23.

TABLE 23, SUMMARY OF SETTLEMENT ESTIMATES

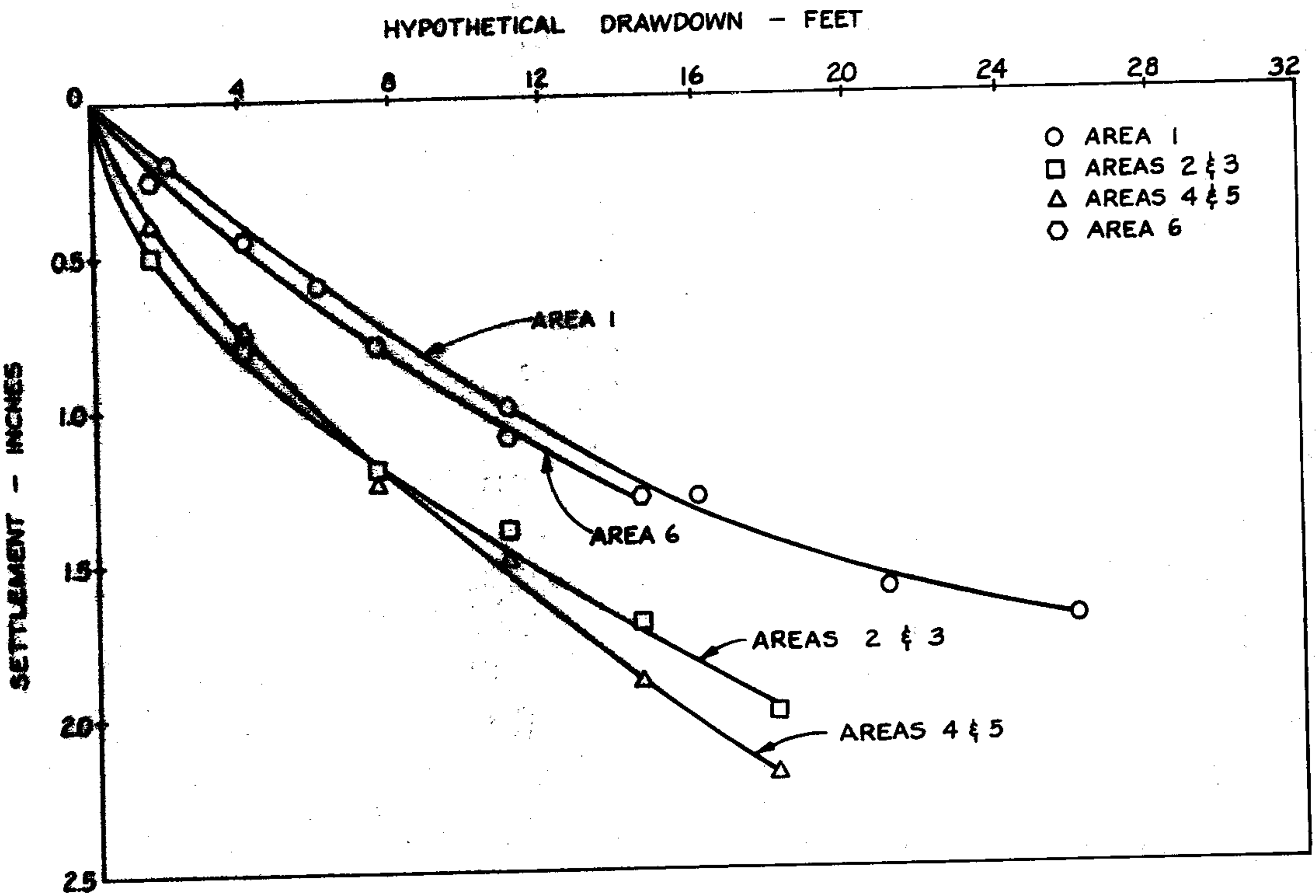
Area (Station)	Draw-down (ft)	Settle-ment (in.)	Area (Station)	Draw-down (ft)	Settle-ment (in.)
1 (Sta 170 to Sta 184)	26	1.7	4 (Sta 231 to Sta 252)	18	2.2
	21	1.6		14.5	1.9
	16	1.2		11	1.5
	11	1.0		7.5	1.25
	6	0.6		4	0.75
	2	0.2		1.5	0.4
2 (Sta 184 to Sta 205)	18	2.0	5 (Sta 252 to Sta 270)	18	2.2
	14.5	1.7		14.5	1.9
	11	1.4		11	1.5
	7.5	1.2		7.5	1.25
	4	0.8		4	0.75
	1.5	0.5		1.5	0.4
3 (Sta 205 to Sta 231)	18	2.0	6 (Sta 270 to Sta 276)	14.5	1.3
	14.5	1.7		11	1.1
	11	1.4		7.5	0.8
	7.5	1.2		4	0.45
	4	0.8		1.5	0.25
	1.5	0.5			

Curves representing settlement versus drawdown were developed and plotted in Figure 33. Evaluations of settlement relative to each of the six areas follows:

Area I (Station 170 to 184)

Based on the hypothetical ground water profile, the depths of drawdown for this area at distances of 0, 50, 100, 150, 200 and 250 ft from pumped well were estimated to be 26, 21, 16,

SETTLEMENT ESTIMATES



SETTLEMENT VS DRAWDOWN

FIGURE 33 A

11, 6 and 2 feet, respectively. Analyses based on the consolidation test data of Boring B-3 indicate that ultimate settlements of 1.7, 1.6, 1.25, 1.0, 0.6, and 0.2 inches will occur for distances of 0, 50, 100, 150, 200, and 250 ft from pumping station, respectively. The rate of settlement will be relatively slow. Residual settlements can be anticipated over a period of 1-1/2 to 2 years.

Area II (Station 184 to 205) and Area III (Station 205 to 231)

The subsoils in these areas are predominantly fine grained. At some locations in the vicinity of Borings P6 and B-9, the clays contain peat or organic fractions. Laboratory test data from soil samples taken from Boring B-1 indicate that the ultimate settlement will be in the order of 2 inches. The amounts involved are estimated to range from 2 to 0.5 inches. Differential settlement should be less than 0.2 inch within a 50 ft interval.

Area IV (Station 231 to 252), and Area V (Station 252 to 270)

The current analysis for these two areas is based entirely on consolidation data derived from a single boring; Boring B-4. Settlements of 2.25, 1.9, 1.5, 1.25, 0.75, and 0.4 inches were calculated for distances of 0, 50, 100, 150, 200, and 250 feet from the pumping well, respectively. The calculated settlements are based on assumed depths of the ground water drawdown ranging from 18, 14.5, 11, 7.5, 4, and 1.5 feet. The predicted settlement will be essentially completed during construction.

Area VI (Station 270 to 276)

Soils in the vicinity of Boring No. B-2 (located 100 feet left of Station 287+25), if subjected to dewatering of 14.5, 1, 7.5, 4, and 1.5 feet would result in settlements of 1.3, 11, 0.8, 0.45, and 0.25 inches, respectively. These values should be considered as conservative since it is believed that the depths of drawdown actually required in this area will be less than the depths considered in this evaluation. Should this prove to be the case, the settlements will be less than stated. The rate of settlement will be relatively rapid and only minor residual settlement is anticipated following construction.

Drawdown Estimates

The drawdown at any particular distance from the pump well is necessary in order to estimate settlement at that point. With this in mind a family of drawdown curves has been developed (Figure 33B). These drawdown curves are valid only for the following conditions

Area No.	Pump Rate (gpm)	Maximum Drawdown at Pump Well (ft)	Maximum Distance of Pump Well From Hinge Point (ft)
1	5	26	15
2 thru 5	3.5	18	15
6	3	14.5	15

Should the pumping rates be altered from the rates listed the plotted curves (Figure 33B) will be invalid and a new set of curves must be developed.

DRAWDOWN ESTIMATES

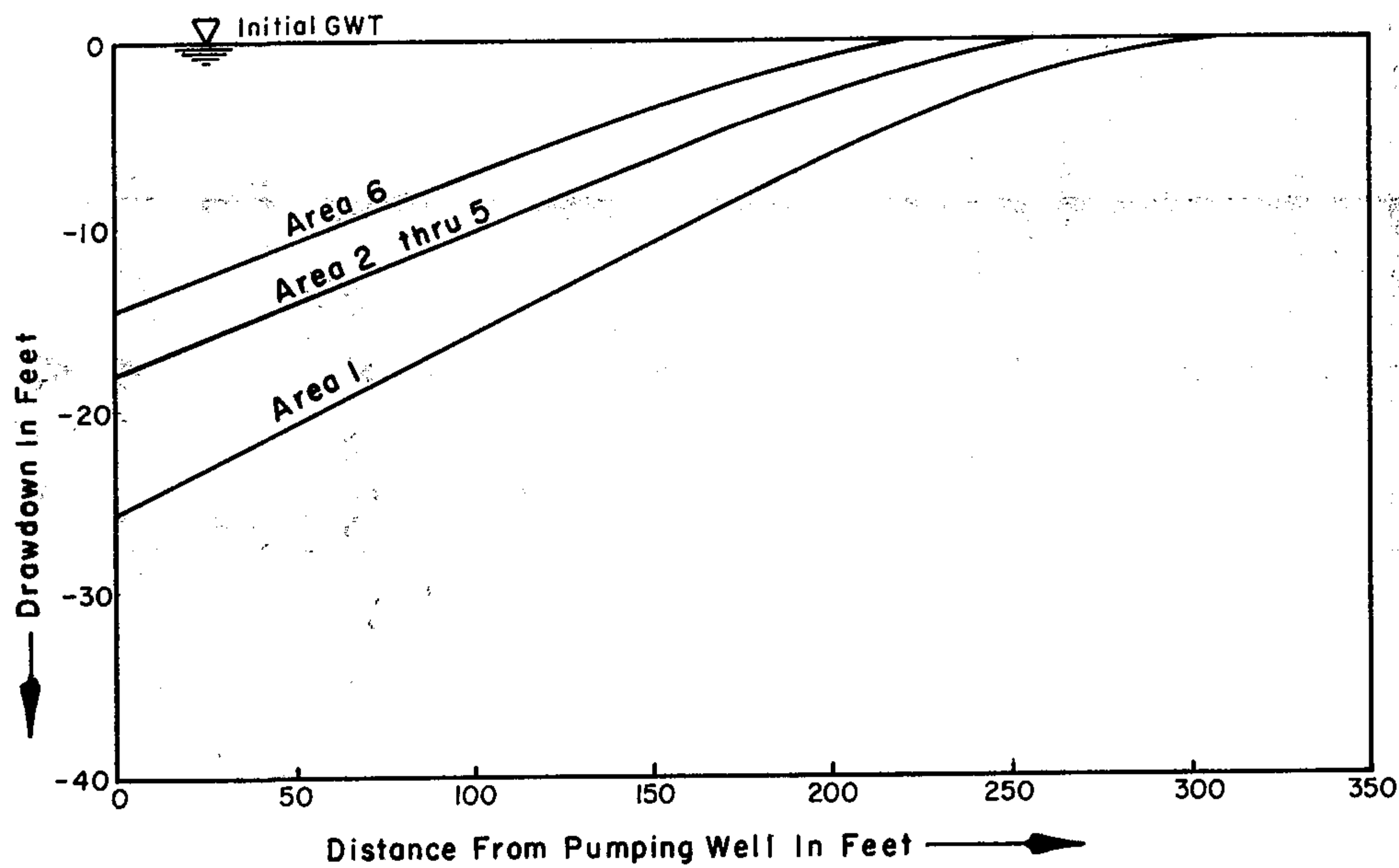


Figure 33-B, DRAWDOWN VS DISTANCE

To estimate maximum settlement at any given distance from the pump well, the following procedure is followed. First, with the distance of this point from the pump well known, the drawdown (in feet) should be estimated from Figure 33B. From this drawdown value the settlement (in inches) can then be determined by use of Figure 33A.

Settlement/Subsidence Effects

Comparing the settlement estimates of the 1971 study with the current study, the magnitudes of ultimate settlement due to the adoption of semi-depressed section are observed to be lower, in general. Hence the differential settlements in a 50 ft interval would be much less. The ultimate magnitudes of settlement will depend upon the choice of the dewatering components adopted both during and after construction.

As previously explained, the depths of deep wells are yet to be decided. The settlement values are estimated at 2.2 inches for a drawdown of 26 feet and 0.2 inch for a drawdown of 1.5 feet.

The continued pumping or removal of ground water from the project area may cause subsidence over areas extending to a maximum of 300 feet beyond the right-of-way. At this distance, subsidence of areas would not exceed 0.2 inch.

During construction the presence of San Francisco Bay Mud layers in excess of 3 feet in thickness should be noted and their extent determined. The settlement estimates presented in this report may require modification if such layers are detected during excavation.

As a precautionary measure it is recommended that prior to the start of dewatering operations a condition survey be conducted of all the major structures adjacent to the excavation area. This condition survey should include adequate detailed photographs, and the placement of suitable bench marks for monitoring settlement during operations. The condition survey should be repeated at periodic intervals during and after the project.

SEA WATER INTRUSION

Sea water intrusion is of considerable importance along the coast of California. Various coastal ground water basins and the aquifers in them are vulnerable to contamination and subsequent degradation in quality by sea water encroachment. In this section, this problem, as relevant to the semi-depressed section, is discussed under four separate parts as enumerated below:

1. Ground water sampling and chemical testing conducted in 1971, 1977, and 1978 are summarized with some conclusions.

2. Analyses conducted to determine the gradients and direction of ground water flow both during and after construction are discussed. The disposition of the sea water-fresh water interface using the Ghyben-Herzberg approximation is determined along the project alignment. Fresh water contours were developed to depict present ground water conditions.

3. Two further models (Models 3 and 4) are presented in idealized form both for existing conditions and to illustrate the long term effects of adopting a physical barrier on the bayside.

4. Potential protective measures are discussed with a few recommended for this project.

Finally, the effect of semi-depressed freeway construction on sea water intrusion is summarized.

Ground Water Sampling and Testing

The possibility of ground water contamination due to sea water intrusion was first assessed during the 1970-71 depressed section investigation. Ground water samples were extracted at intervals during pump tests in 1971 and chloride contents determined. The results of the tests are listed in Table 24.

TABLE 24 GROUND WATER CHLORIDE CONTENT (from tests in 1971)

Test No.	Sta	Chloride ppm
R-1	290	810
R-2	260	190
R-3	220	140
R-4	180	100

Although Tests R-1 through R-4 indicated very low chloride contents, well within acceptable limits for fresh water aquifers, Test R-1 with 810 ppm chloride indicated slight sea water contamination in the vicinity of Station 290, (Figure 6). It can be observed from Table 24 that the chloride content increases progressively from 100 ppm, at R-4, to 810 ppm, at R-1. Even though R-1 is outside the limits of the present study, by straight-line interpolation the chloride content can be expected to be about 500 ppm in the vicinity of Station 275±. Boss and Squires commented on this in the following quotation from their report of September 1971(4).

> "The chloride content analyses of pump water at the various locations indicate fresh water aquifers except at the extreme northwest location. This area is near a dredged ship channel

> where the impermeable liner of bay mud has probably been perforated and is allowing salt water intrusion into the area. Lowering the fresh water-water table and hydrostatic head in this area will undoubtedly increase the chances for salt water intrusion into a larger area. However, no quantitative results can be presented at this time. The relative imperviousness, and the fact that the depressed section is daylighting in this area would tend to minimize the possibility of the salt water."

The water samples taken in 1971 presumably represented ground water levels pertinent to the then contemplated fully depressed section. However, in 1977 with a semi-depressed section under consideration the ground water condition at shallower depths became more important. In September 1977 a more comprehensive sampling of ground water was performed at the time of the two previously described pumping tests. Eleven samples were taken and tested to determine resistivity and pH values as well as chloride and sulfate content. The chloride contents ranged from 64 to 142 mg/l, somewhat lower than those of the 1971 series. The conductivity (E.C.) values varied from 600 micromhos/cm to 1400 micromhos/cm with nine of the eleven samples having values equal to, or greater than, 1000 micromhos/cm. The results of the 1977 test series are tabulated in Table 25.

A third series of ground water samples was collected in July 1978 and tested for chloride and sulfate content and various chemicals. This group was obtained from 9 borings at depths ranging from 12 to 17 feet below ground surface. The test results are listed in Table 26. The chloride content and sulfate content showed a slight increase in two borings (P-9 and P-10) since the 1977 study. However, in Borings P-7, P-6, P-4, P-2, P-11 and P-13 these constituents remained constant, or decreased slightly. The ratio of chlorides

TABLE 25, SEA WATER INTRUSION STUDY - CHEMICAL TESTS
Sampled 9-8-77

Area	Station	Boring	Sample	Conductivity micro mho/cm	pH	Chlorides mg/1	Sulfates mg/1
	116+85 175'Lt	P-12	28W	1200	7.3	96	120
2	187+80 290'Rt	P-10	26W	1400	7.2	64	33
2	201+00 165'Rt	P-9	25W	900	7.7	96	130
3	208+00 180'Rt	P-8	22W	1000	6.9	91	37
3	220+10 50'Rt	P-7	20W	1300	7.0	60	71
3	228+50 175'Rt	P-6	18W	1400	7.2	92	69
4	239+40 200'Rt	P-5	17W	1400	7.3	118	25
4	249+80 10'Rt	P-4	16W	1300	6.6	128	82
5	259+00 20'Lt	P-3	14W	600	7.3	119	680
5	272+00 115'Rt	P-2	10W	1100	7.4	142	75
	278+50 50'Rt	P-1	2W	1300	7.2	80	57

TABLE 26. SEA WATER INTRUSION STUDY

(Date of Sampling: July 18-19, 1978)

					Field Measurements - Phase I				Laboratory Test Data - Phase I							
Area	Station	Boring No.	Depth of Sampling (ft)	Sample No.	Temp. °C	Specific Conductance μ mhos	pH	Dissolved Oxygen	Specific Conductance μ mhos	pH @ 25°C	Chloride (Cl) mg/l	Bicarbonate (HCO_3) mg/l	Sulfates (SO_4) mg/l	Nitrates (NO_3) mg/l	Carbonate (CO_3) mg/l	$\frac{Cl}{CO_3+HCO_3}$
1	177+00	P-11	17	7-19-1	17.0	668	6.7	2.2	680	7.2	60	185	65	19	0	.32
1	181+20	D-16	16	7-19-2	17.2	726	6.8	2.2	740	7.3	65	180	93	28	0	.36
2	187+80	P-10	16-17	7-19-3	16.5	741	7.3	2.5	740	7.7	67	270	38	24	0	.25
2	201+00	P-9	13	7-19-4	18.2	0.005*	7.1	4.2	1370	7.4	116	425	139	32	0	.27
3	220+10	P-7	14	7-19-5	17.7	553	7.4	1.2	530	7.8	8.4	250	42	18	0	.03
3	228+50	P-6	12	7-19-6	17.6	914	7.1	1.9	900	7.4	87	285	67	27	0	.31
4	249+80	P-4	12	7-19-7	18.2	659	7.0	1.2	670	7.2	73	190	39	24	0	.38
5	272+00	P-2	14	7-19-8	17.6	1086	7.2	1.5	1060	7.6	141	270	70	41	0	.52
6	274+15	P-13	12	7-19-9	18.1	1449	7.0	2.1	1330	7.8	172	305	123	46	0	.56

*Erroneous reading due to instrument malfunction.

to sum of carbonates and bi-carbonates varied from 0.27 to 0.38 in Borings P-11, D-16, P-10, P-9, P-7, P-6, and P-4, which is indicative of fresh water. However, the ratio value of Boring P-2 was 0.52, and that of Boring P-13 was 0.56, which indicates slight contamination by sea water.

Analyses

Gradients and Directions of Ground Water Flow

The thickness of alluvium varies from about 50 feet in the east end to about 300 feet in the west end of this project. This alluvial bed contains a multiple aquifer system, consisting of thin to thick silt or sand and gravel lenses. Some of these lenses are not connected with one another while others are spatially inter-connected. The presence of perched water tables and mounds was confirmed during the field investigations. It is assumed that these water bearing layers are in hydraulic continuity with sea water.

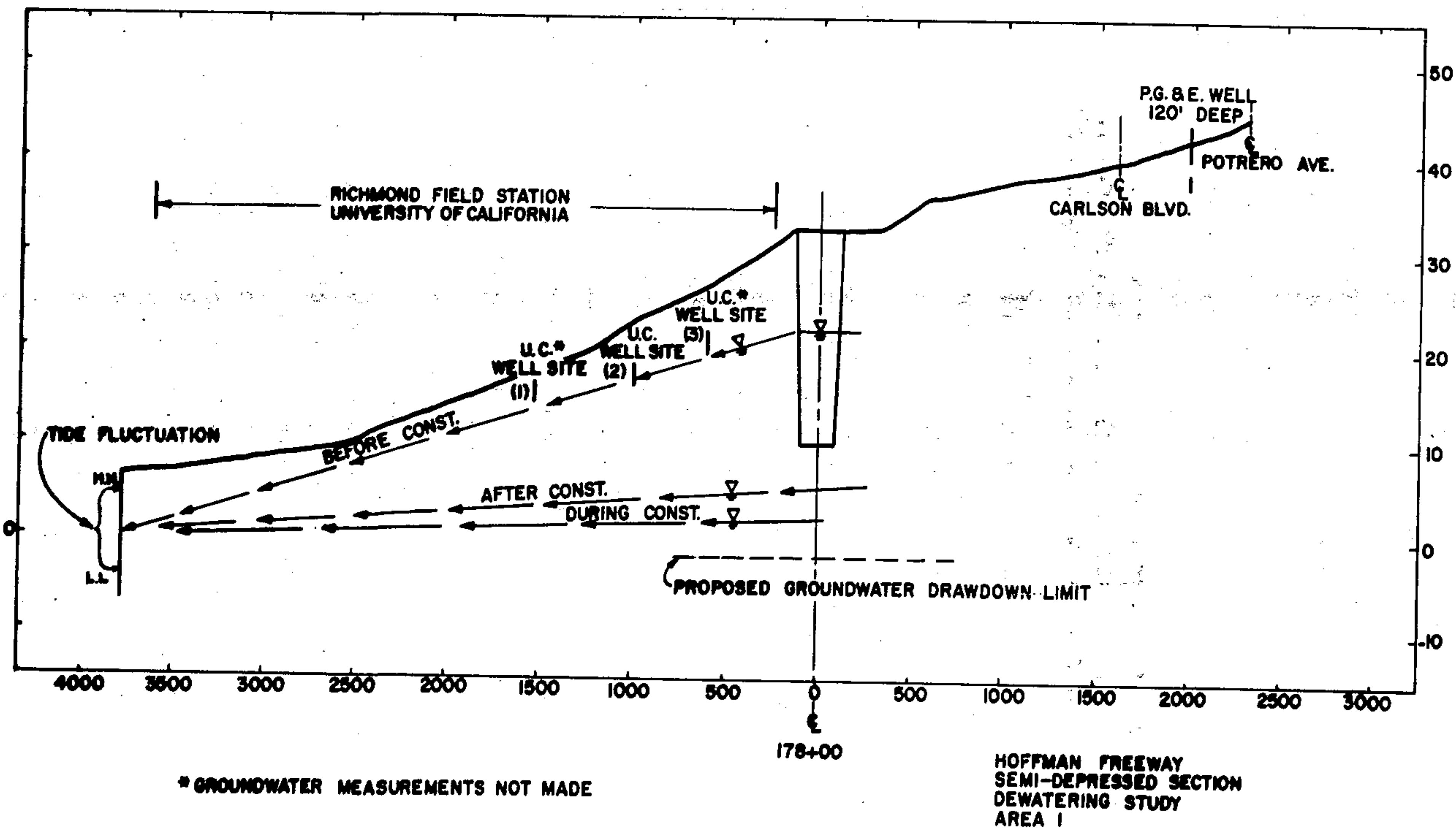
From a review of proposed profile grades it is evident that subexcavation levels will be below mean sea level between the approximate limits of Station 222 and Station 274. These limits are based on a maximum excavation depth of 5 feet below profile grade, which corresponds to the after construction condition as far as the depressed ground water level is concerned.

Six cross sections were drawn to scale at Stations 178+00, 189+00, 221+00, 238+50, 260+00 and 270+00 (as shown in Figures 34 through 39) to determine if there are any gradients that would cause landward flow of sea water. At these six locations the gradients before construction were calculated using the difference between existing ground water levels at the time of ground water investigation (1977) and mean sea level, taking into account the shortest distance from the centerline of the semi-depressed section to the bay. For the "during construction condition", the ground water level was assumed to be depressed eight feet below profile grade. For the "after construction condition" the depressed ground water level was assumed to be five feet below profile grade. Based on these assumptions, the gradients and directions of flow were determined and are tabulated in Table 27.

TABLE 27, ESTIMATED GRADIENT AND DIRECTION OF GROUND WATER FLOW

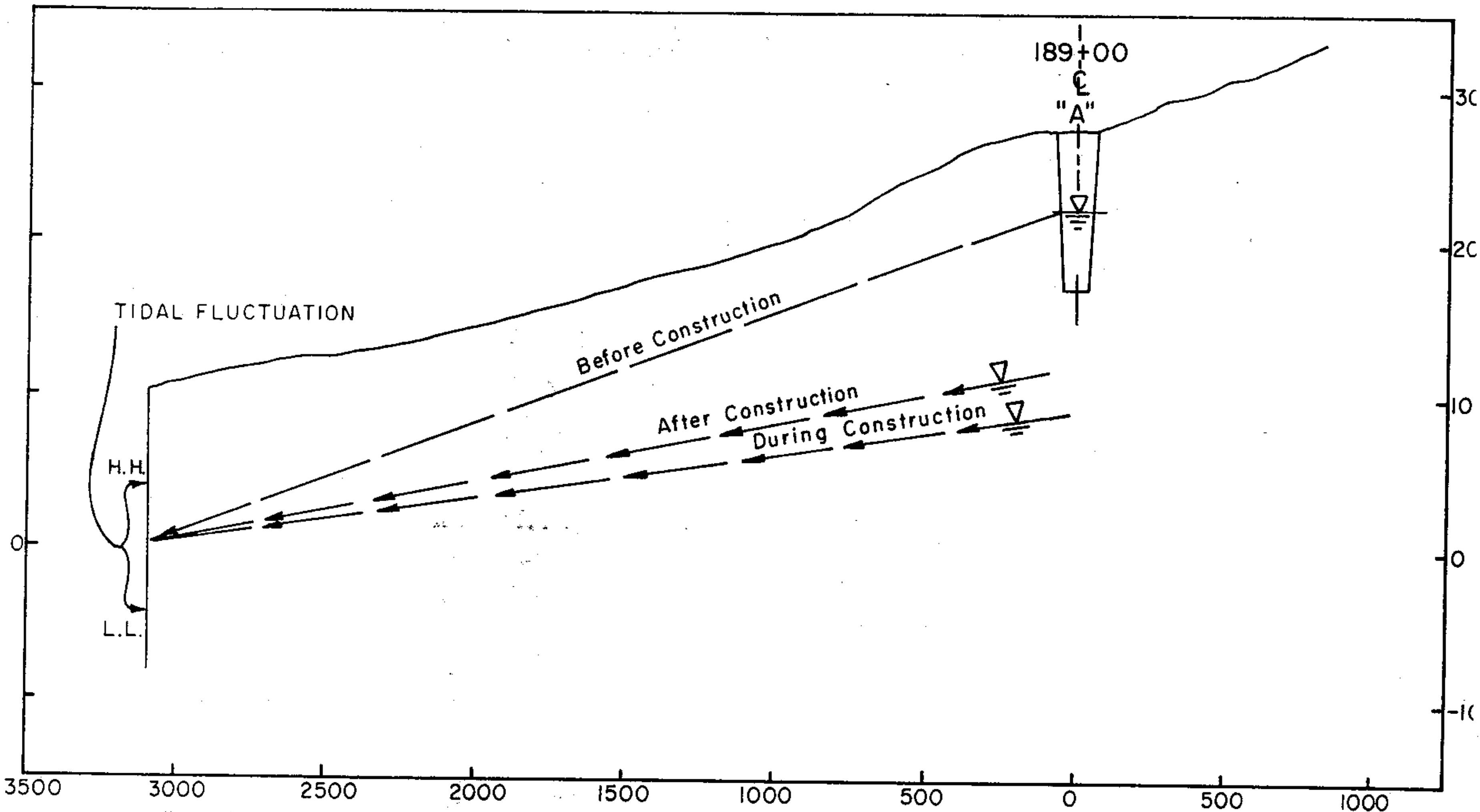
Cross Section	Gradient			Direction of Flow		
	Before Const.	During Const.	After Const.	Before Const.	During Const.	After Const.
178+00	.0060	.0005	.0014	Bayward	Bayward	Bayward
189+00	.0071	.0027	.0037	Bayward	Bayward	Bayward
221+00	.0073	.0006	.0003	Bayward	Landward	Bayward
238+50	.0024	.0039	.0027	Bayward	Landward	Landward
260+00	.0050	.0120	.0096	Bayward	Landward	Landward
270+00	.0020	.0082	.0058	Bayward	Landward	Landward

From Table 27, it is evident that from about Station 221+00 to Station 238+50 there will be temporary reversal to landward of the ground water flow during construction. After construction this trend will be reversed and bayward flow of ground water will resume. However, between Station 238+50 and 270+00 the landward flow that will be established during the construction period will continue without change. The



GRADIENTS AND DIRECTIONS OF GROUND WATER FLOW
AT STATION 178
NEAR UC RICHMOND FIELD STATION

FIGURE 34

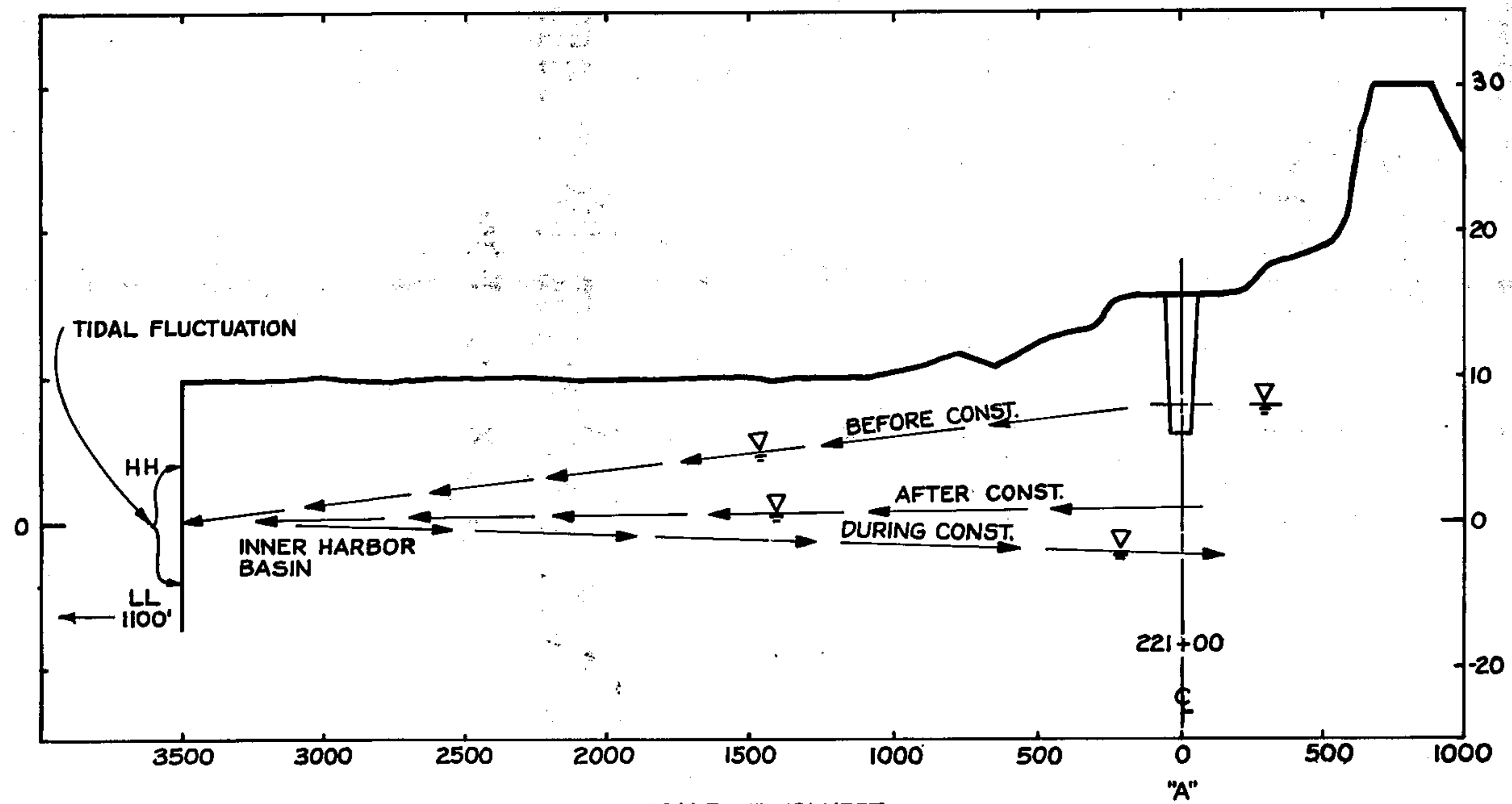


SCALE: 1" = 10' VERT.
1" = 500' HORIZ.

HOFFMAN FREEWAY
SEMI-DEPRESSED SECTION
DEWATERING STUDY
AREA 2

GRADIENTS AND DIRECTIONS OF GROUND WATER FLOW
AT STA. 189+00

FIGURE 35

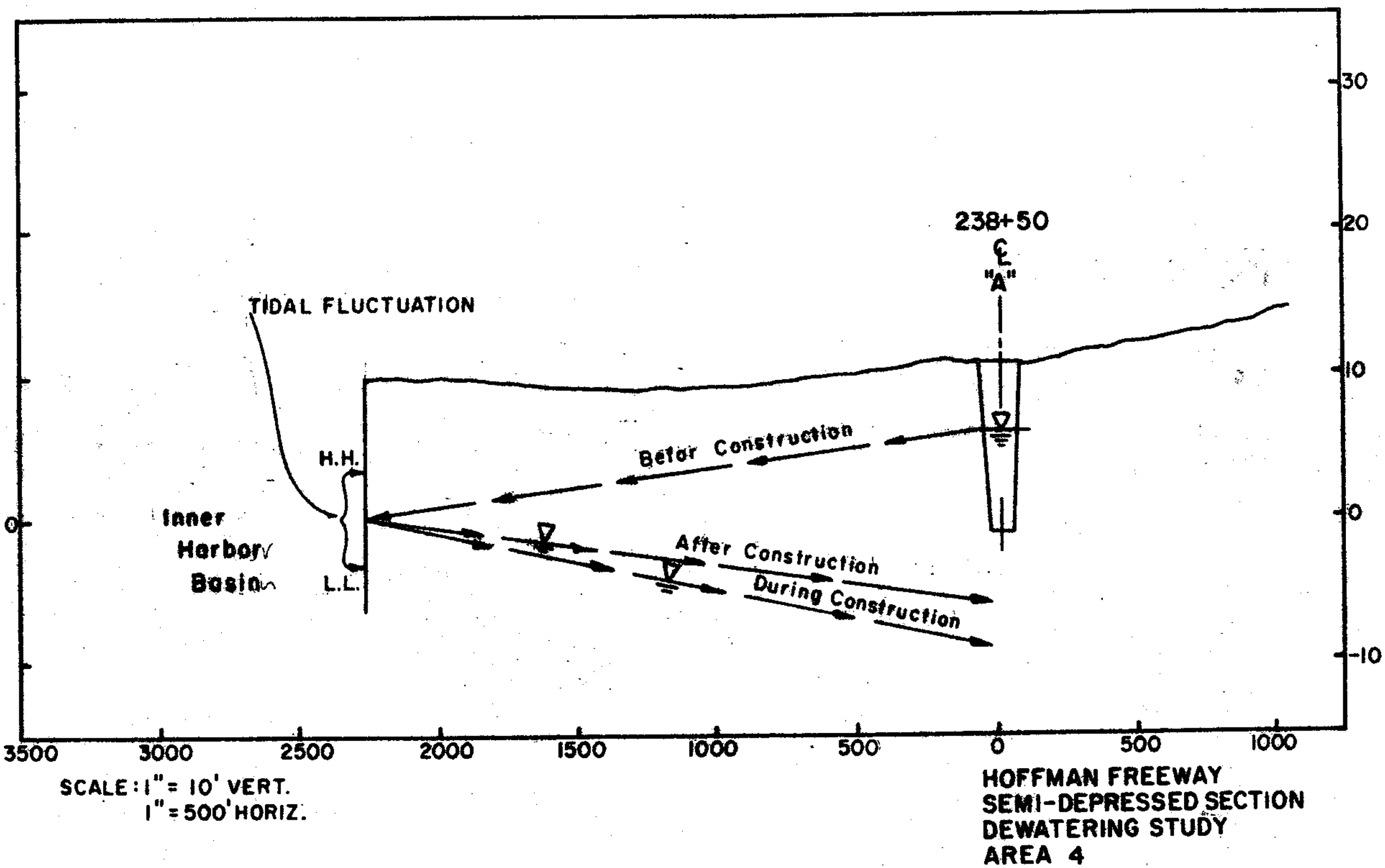


SCALE: 1" = 10' VERT.
1" = 500' HORIZ.

HOFFMAN FREEWAY
SEMI-DEPRESSED SECTION
DEWATERING STUDY
AREA 3

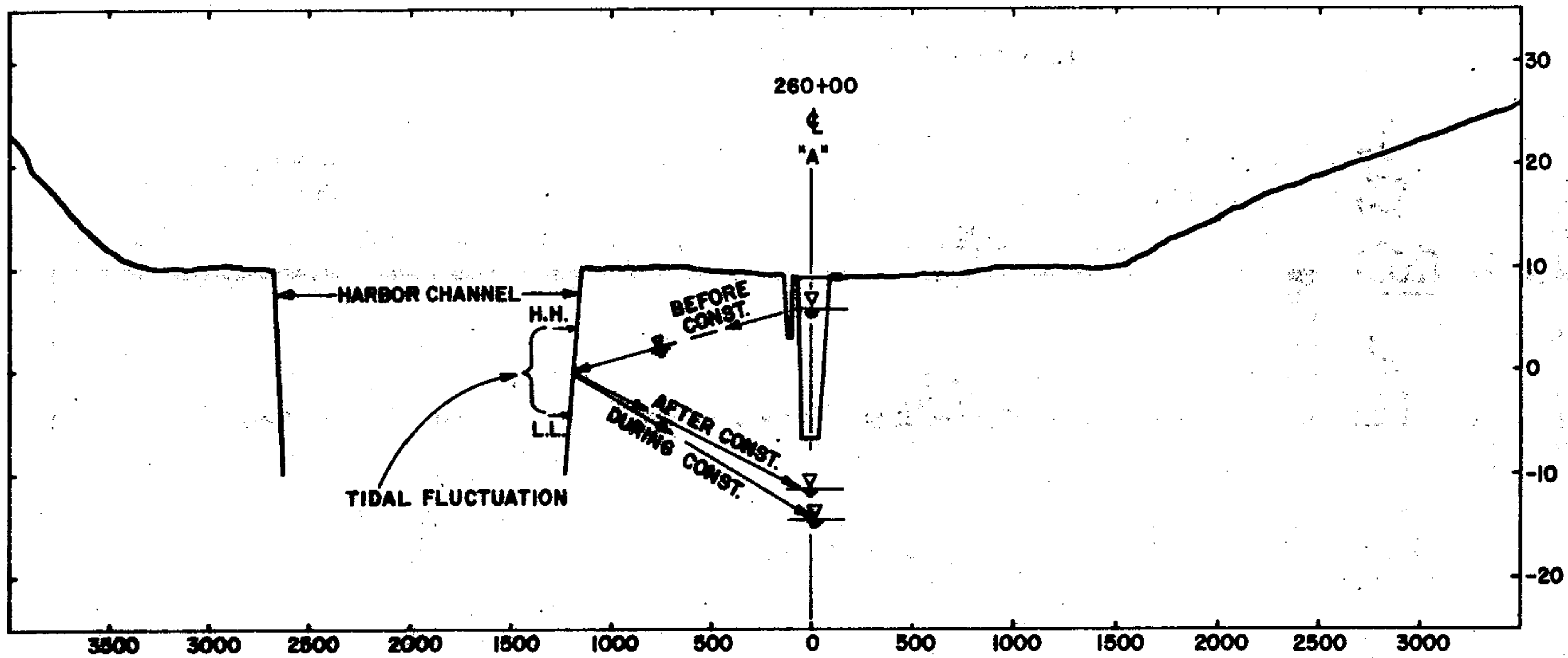
GRADIENTS AND DIRECTIONS OF GROUND WATER FLOW
AT STATION 221+00

FIGURE 36



GRADIENTS AND DIRECTIONS OF GROUND WATERFLOW
AT STA. 238+50

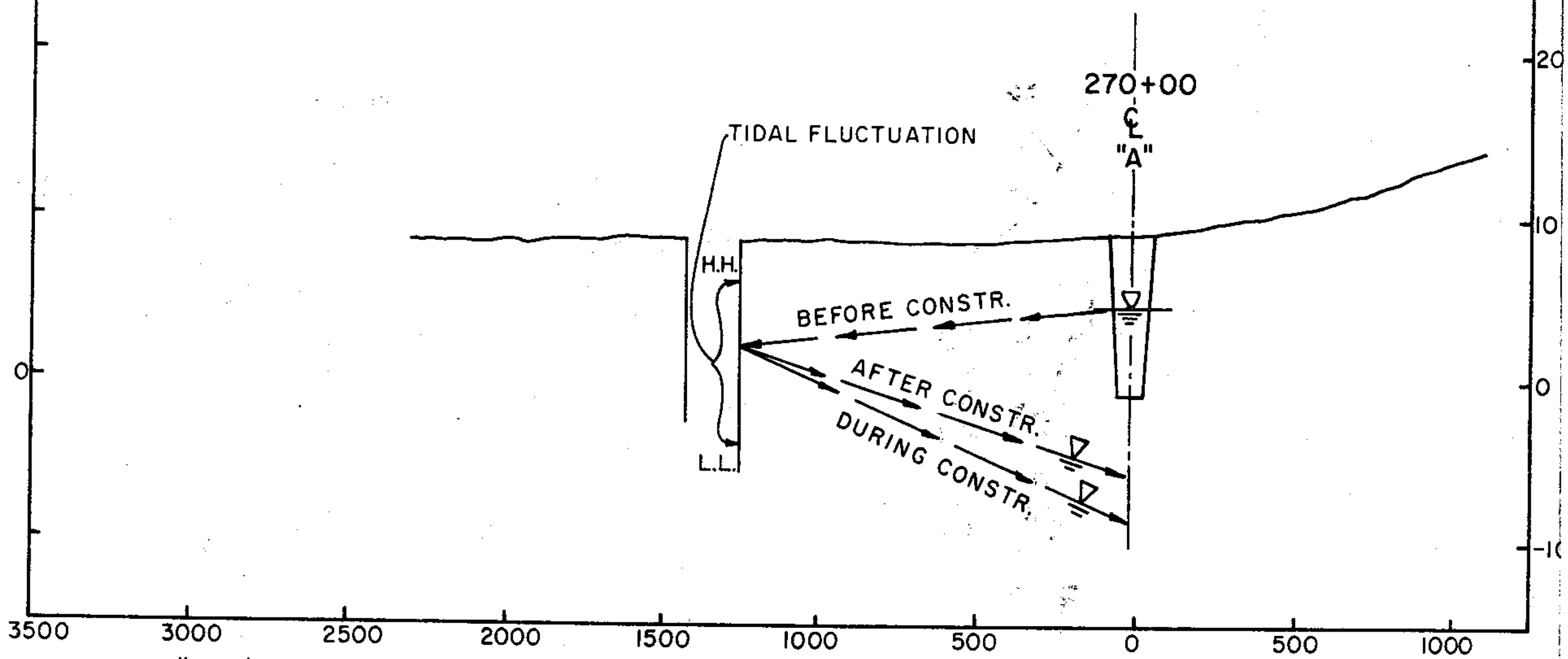
FIGURE 37



HOFFMAN FREEWAY
RICHMOND SEMI-DEPRESSED SECTION
DEWATERING STUDY
AREA 5

GRADIENTS AND DIRECTIONS OF GROUND WATER FLOW
AT STATION 260+00

FIGURE 38

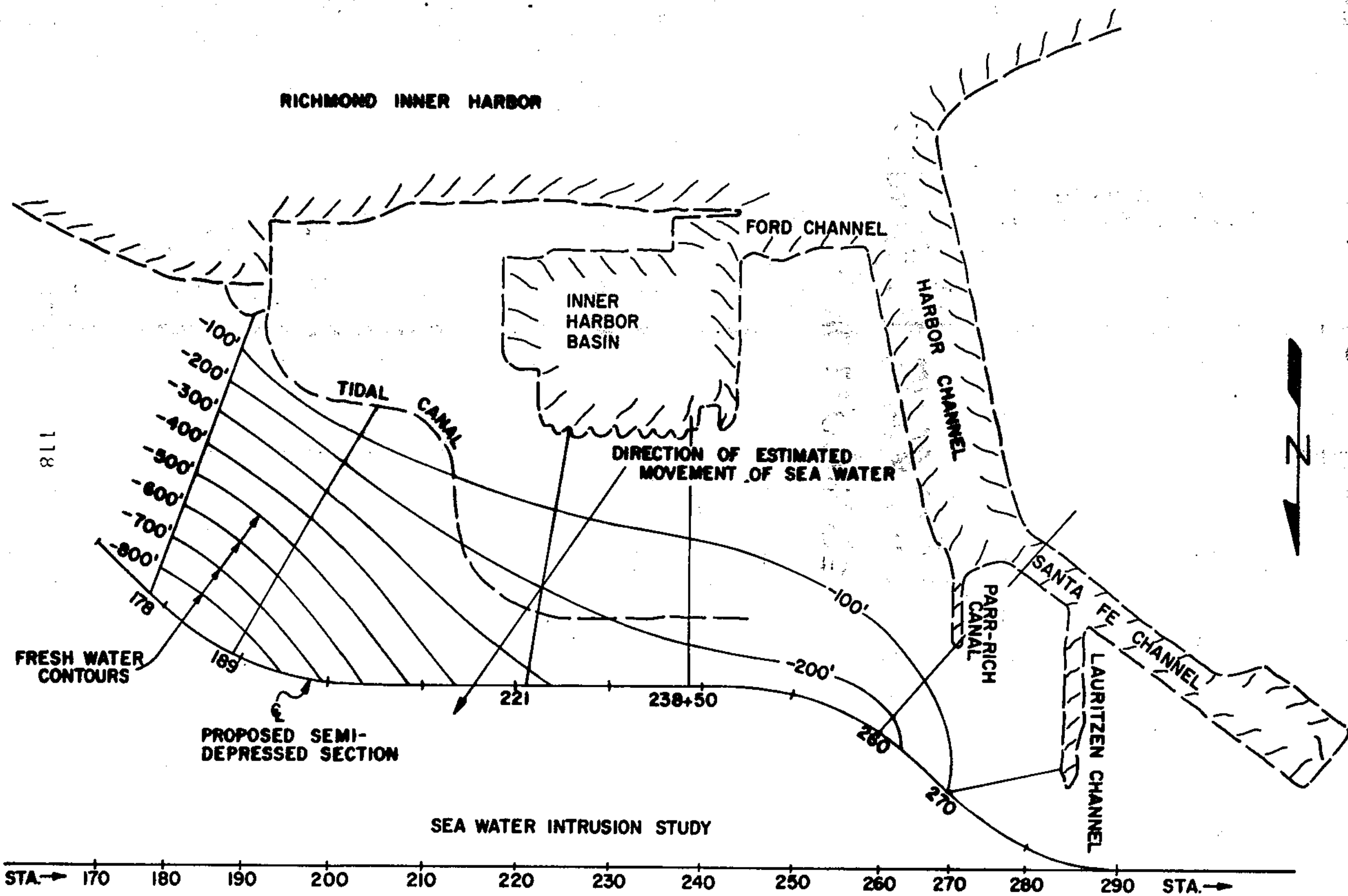


SCALE: 1" = 10' VERT.
1" = 500' HORIZ.

HOFFMAN FREEWAY
SEMI-DEPRESSED SECTION
DEWATERING STUDY
AREA 6

GRADIENTS AND DIRECTIONS OF GROUND WATERFLOW
AT STA. 270+00

FIGURE 39



DIRECTION OF SEA WATER ADVANCE

FIGURE 40

gradients and directions of flow described above will hold true only for the conditions assumed in this analysis. If these conditions are altered either during or after construction the gradients and directions of flow will not be the same. This is particularly true of the area between Stations 221 and 238+50; where, if more ground water is removed either because of facility-related requirements or due to any other cause, the temporary landward flow condition could become permanent.

The direction of sea water advance (encroachment or invasion) is shown in Figure 40. The contours are for fresh water depths below mean sea level.

Indications of Chemical Analysis

From chemical analyses it is known that slight contamination by sea water already exists around Station 270 presumably due to past construction activities which resulted in puncturing of the bay mud layers. Dredged ship channels are now located in this area. It is anticipated that the present sea water contamination will increase in this area due to the construction of the proposed semi-depressed freeway.

Disposition of Interface

The interface between sea water and ground water can be determined using the Ghyben-Herzberg principle (23). This principle considers the fact that saline water is about 1.025 times heavier than fresh water; and, when the two meet within a permeable formation, the lighter fresh water tends to float above the heavier or more dense saline water. Using this principle at six locations, the disposition of the interface has been evaluated and graphically delineated in Figures 41 through 46. The results are listed in Table 28.

TABLE 28 DISPOSITION OF INTERFACE

Station	Depth to Interface Below M.S.L.(Feet)
178+00	880
189+00	880
221+00	320
238+50	240
260+00	240
270+00	100

Note: Depths are below profile grade at centerline of freeway.

The depths given in Table 28 are based on the assumption that the soil is homogeneous to the depths considered in this report. In reality, it is well known that the subsoil conditions are quite heterogeneous.

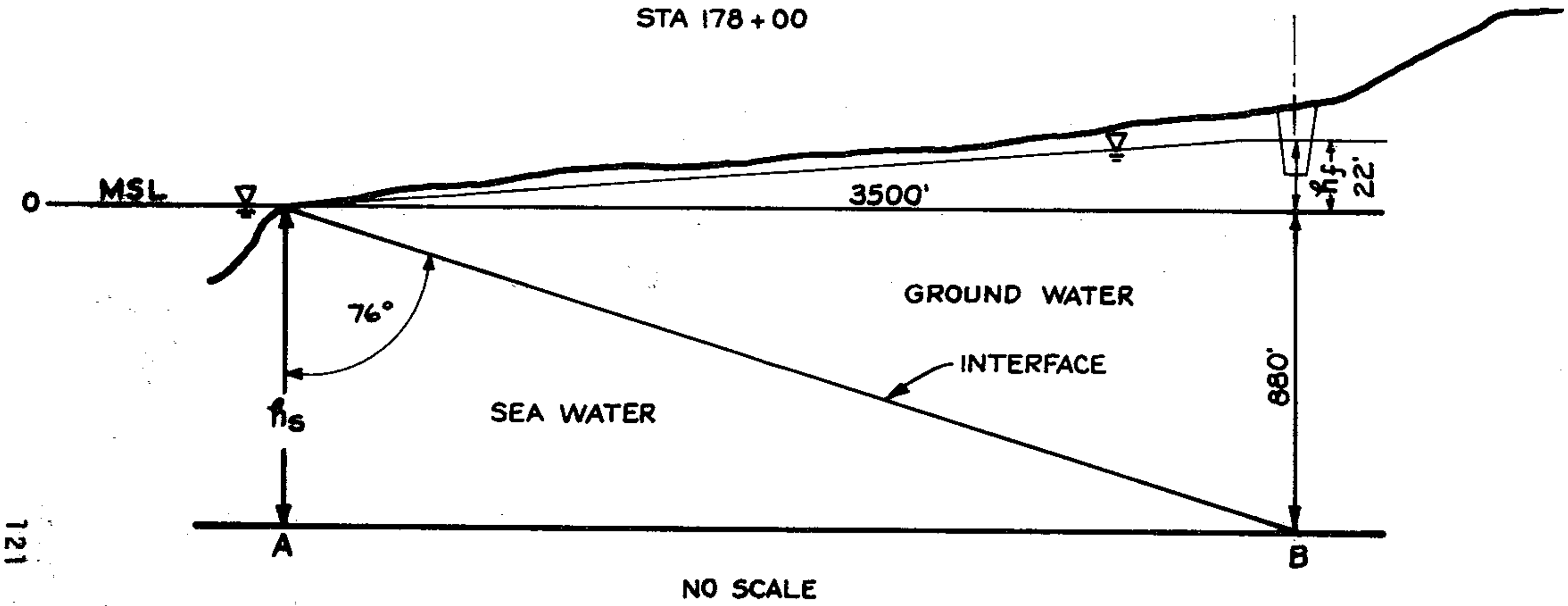
An observation can be made to the effect that the depth of fresh water is only about 100 feet in the vicinity of Station 270, or much less compared to about 800 feet around Station 178.

Another observation can now be stated, that at about Station 270 there will be a much lesser depth of fresh water due to the homogeneity of the soil conditions when compared to the depth of fresh water at Station 178.

The indications of chemical analyses are that there is slight contamination by sea water in the area of Station 270. The findings by Ghyben-Herzberg approximation also confirm this.

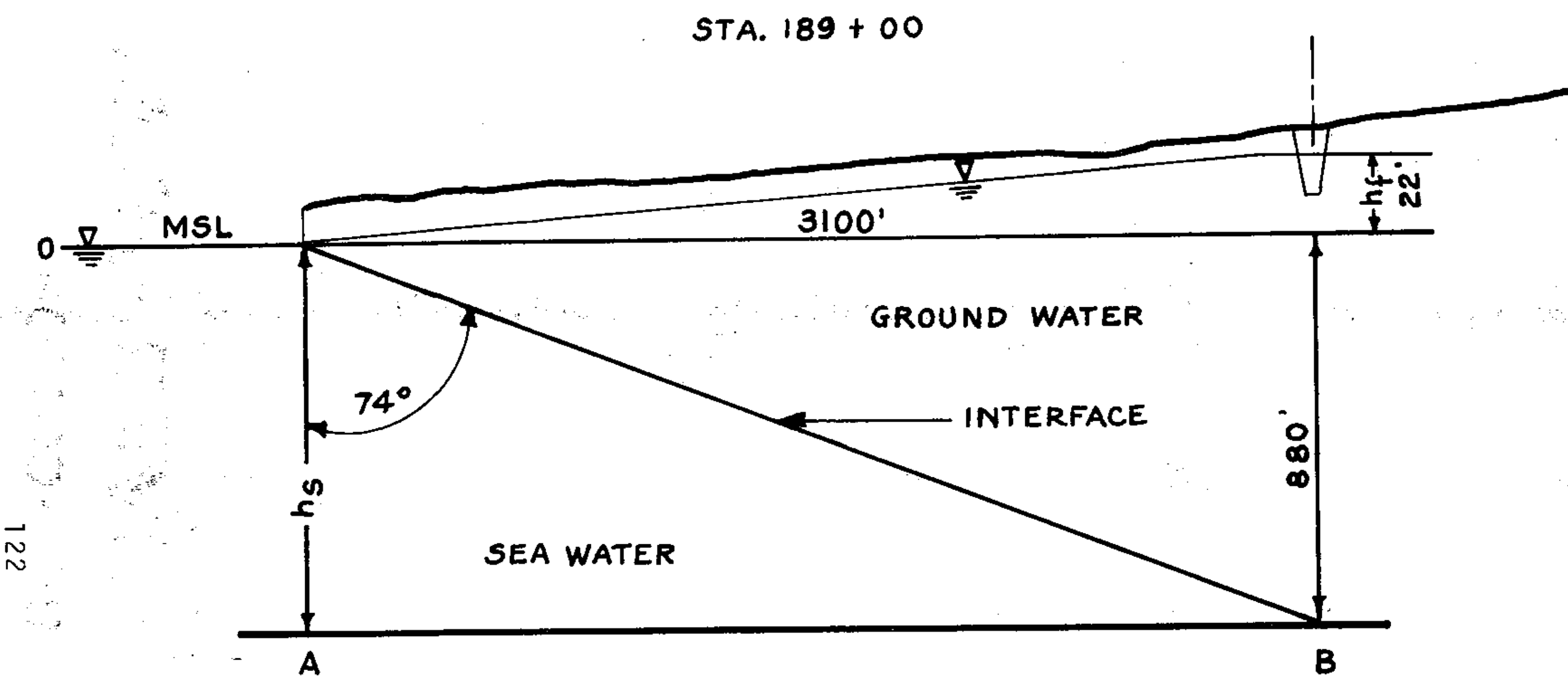
Proposed Models

Based on the subsurface information collected, two models (Models 3 and 4) of how the sea water intrusion occurs and



SKETCH OF SEA WATER INTRUSION STUDY
USING THE GHYBEN-HERZBERG RELATION

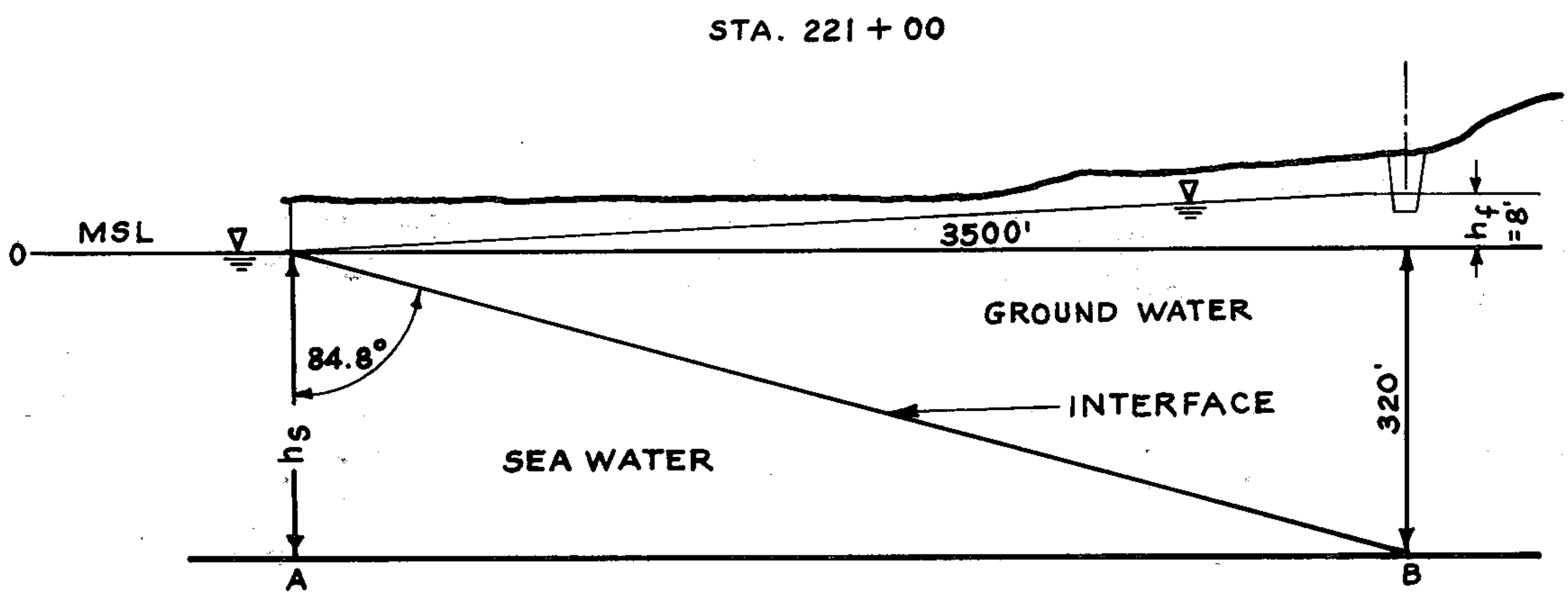
FIGURE 41



SKETCH OF SEA WATER INTRUSION STUDY
USING THE GHYBEN-HERZBERG RELATION

NO SCALE

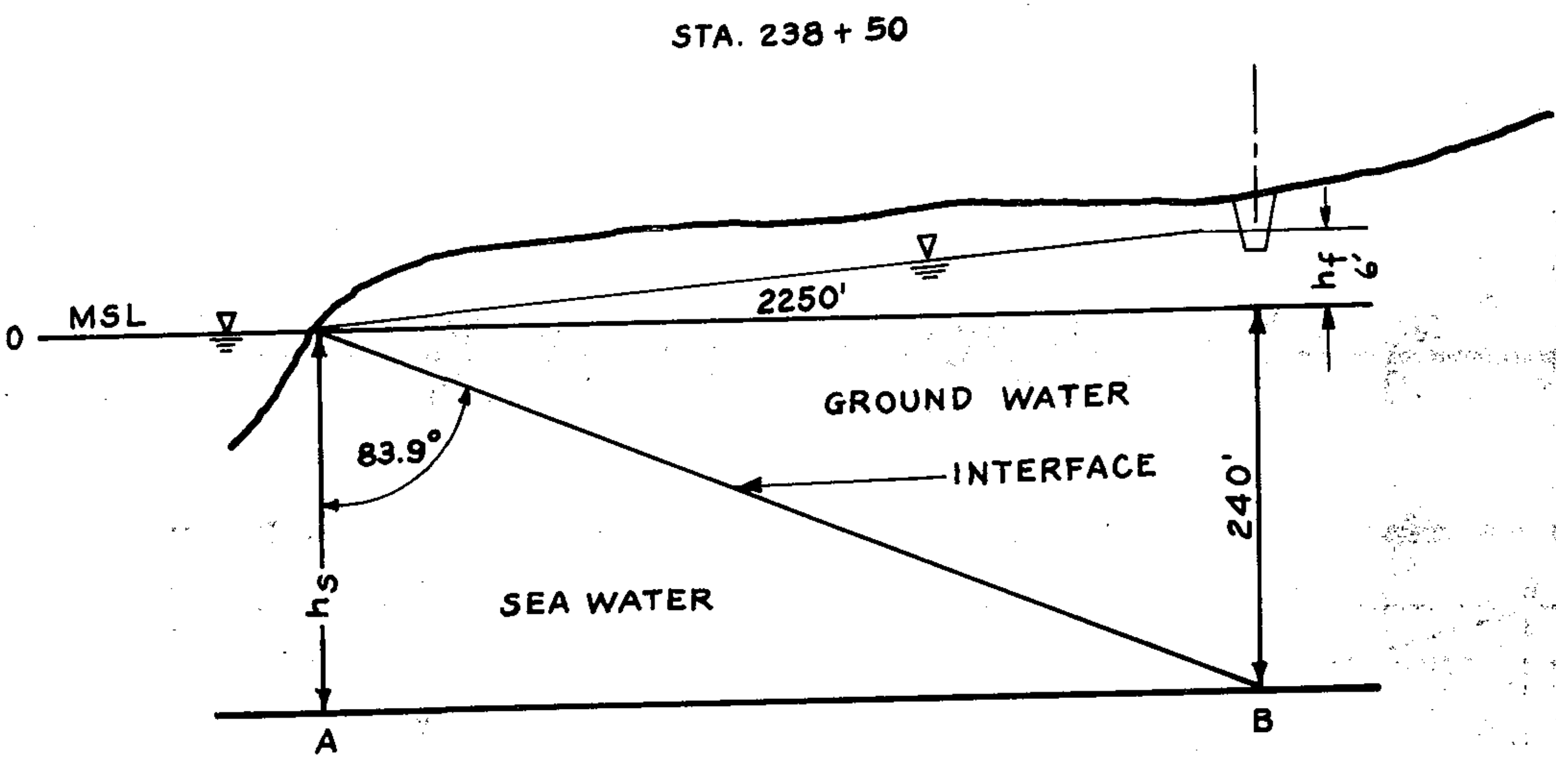
FIGURE 42



SKETCH OF SEA WATER INTRUSION STUDY
USING THE GHYBEN-HERZBERG RELATION

NO SCALE

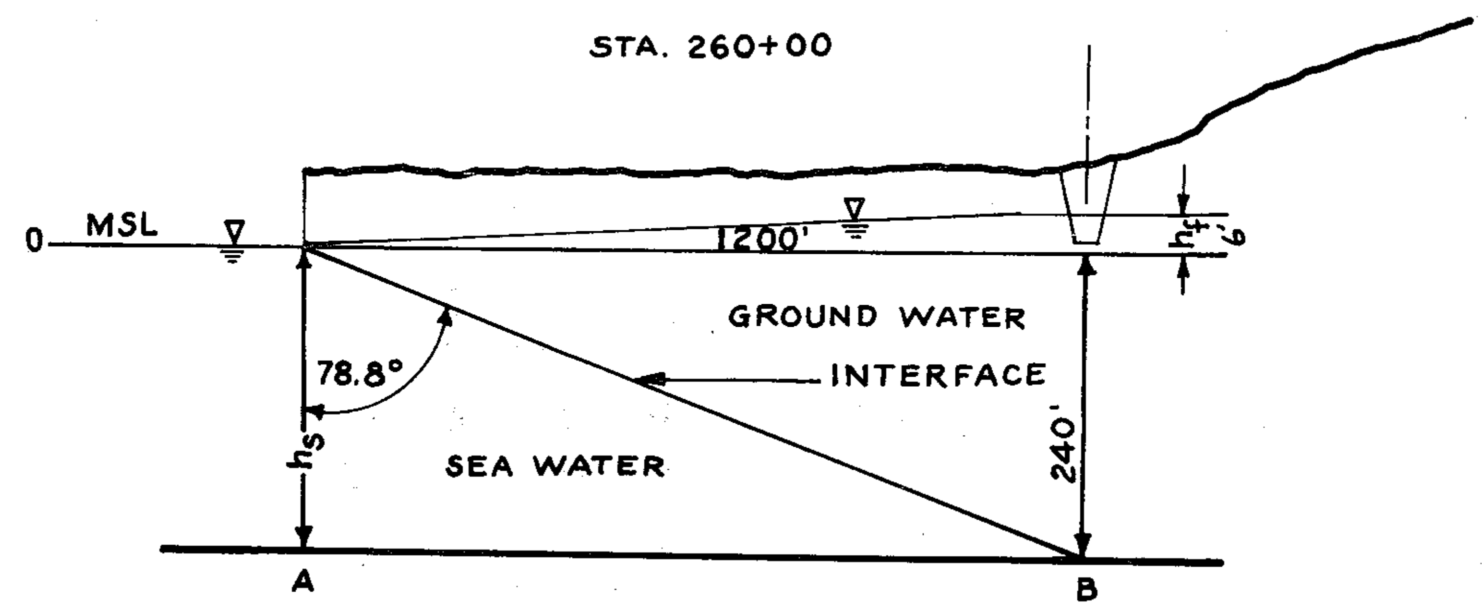
FIGURE 43



SKETCH OF SEA WATER INTRUSION STUDY
USING THE GHYBEN-HERZBERG RELATION

NO SCALE

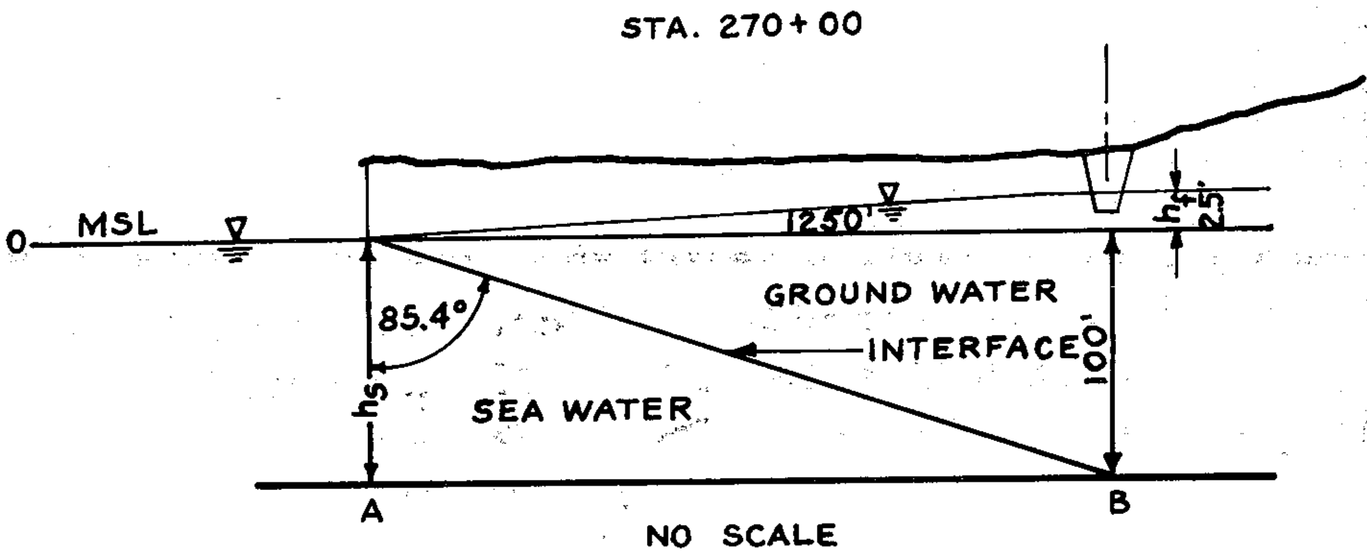
FIGURE 44



SKETCH OF SEA WATER INTRUSION STUDY
USING THE GHYBEN-HERZBERG RELATION

NO SCALE

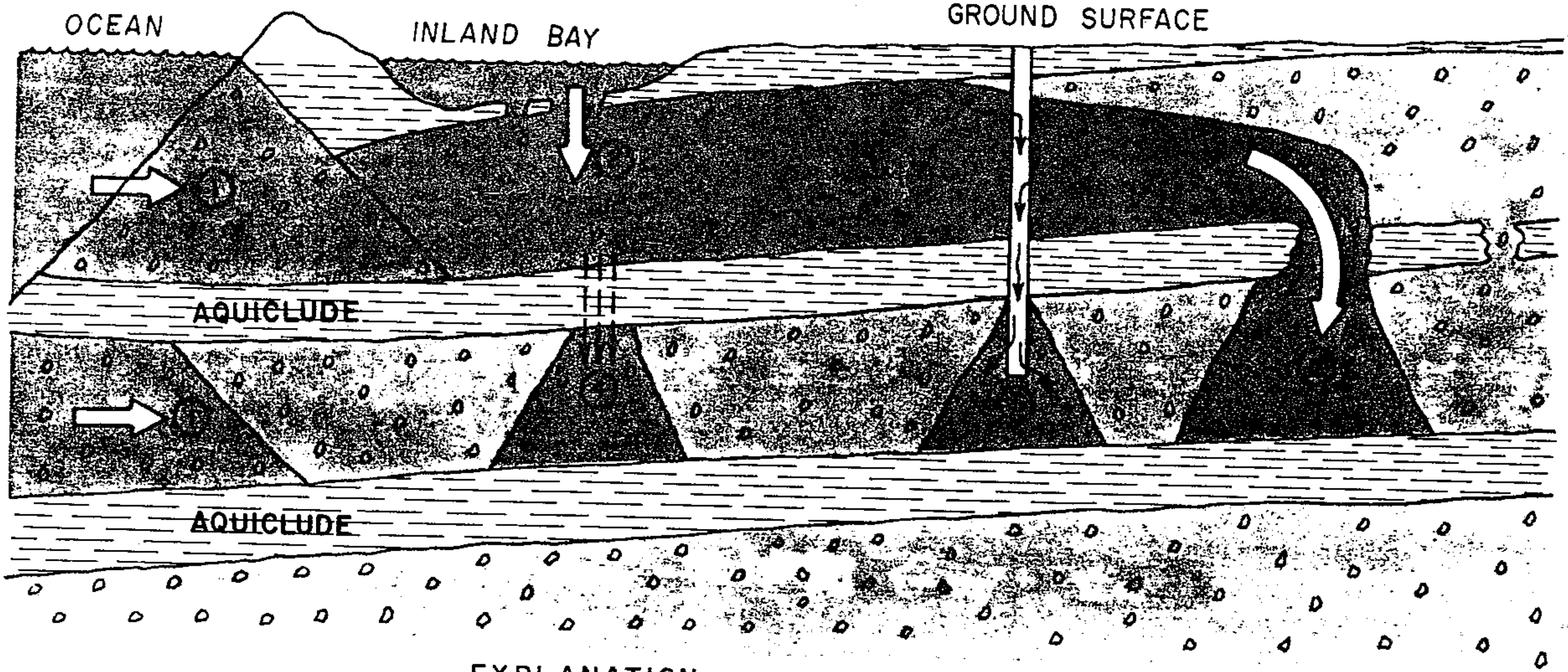
FIGURE 45



SKETCH OF SEA WATER INTRUSION STUDY
USING THE GHYBEN-HERZBERG RELATION

FIGURE 46

SEA WATER INTRUSION STUDY FOR RICHMOND SEMI-DEPRESSED SECTION

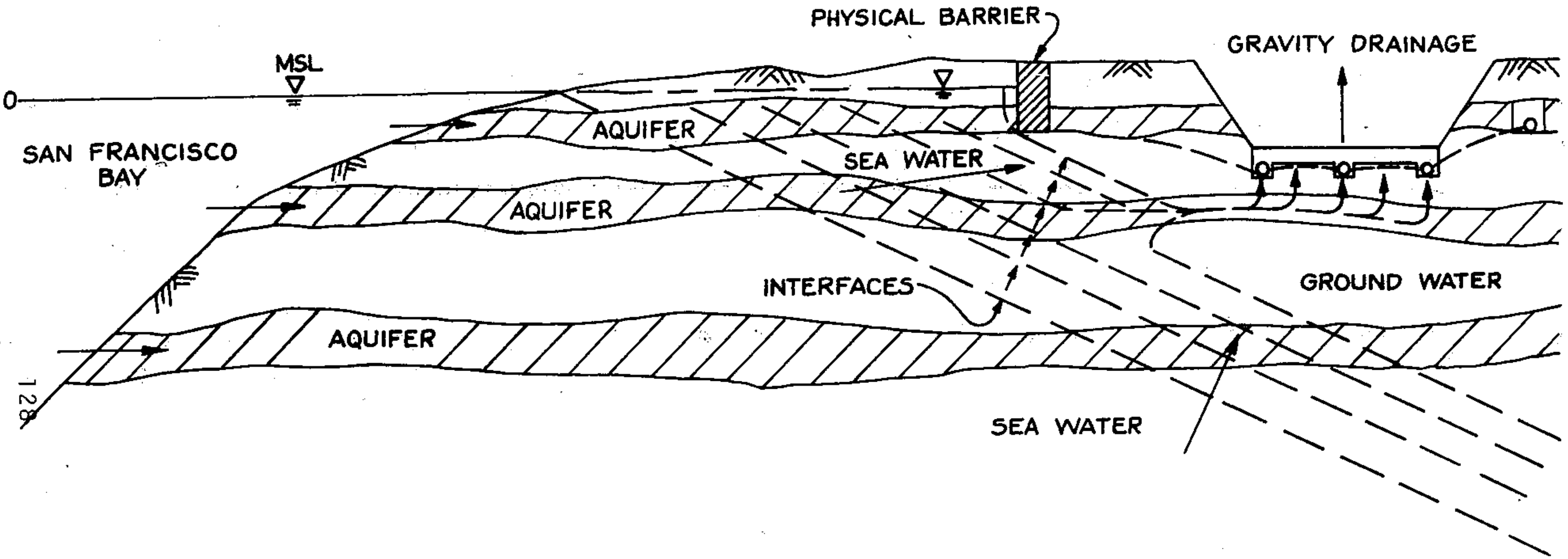


EXPLANATION

- CLAY BEDS
- AQUIFERS
- FRESH WATER
- SALINE WATER

1. DIRECT INTRUSION FROM OCEAN INTO COASTAL AQUIFERS AS A SEA-WATER WEDGE.
2. DIRECT MOVEMENT OF BAY WATER THROUGH NATURAL OR MAN-MADE BREAKS IN CLAY BEDS.
3. SPILLING OF DEGRADED SHALLOW AQUIFER WATERS INTO DEEPER AQUIFERS.
4. SLOW DOWNWARD MOVEMENT OF SALINE WATER THROUGH CLAY LAYERS INTO UNDERLYING AQUIFERS.
5. SPILLING OR CASCADING OF SALINE WATER INTO UNDERLYING AQUIFERS THROUGH IMPROPERLY CONSTRUCTED OR ABANDONED WELLS.

FIGURE 47, MODEL 3 REFERENCE 17, PAGE 392



SEA WATER INTRUSION STUDY, RICHMOND SEMI-DEPRESSED SECTION
MODEL 4, LONG TERM EFFECTS OF A PHYSICAL BARRIER

FIGURE 48

how a physical barrier would affect it in the long term, are here proposed. Models 1 and 2 were proposed earlier in this report.

Model 3, Idealized Model of Existing Conditions

Model 3, shown in Figure 47, depicts the presence of alternating layers of aquifers and aquicludes. It summarizes the five usual sources of sea water intrusion, namely: direct intrusion, leakage of sea water through man-made breaks in upper clay beds, interconnections between upper and lower aquifers, movement through aquicludes, and leakage through improperly constructed and/or sealed wells.

Model 4, Idealized Model of Long Term Effects of a Physical Barrier

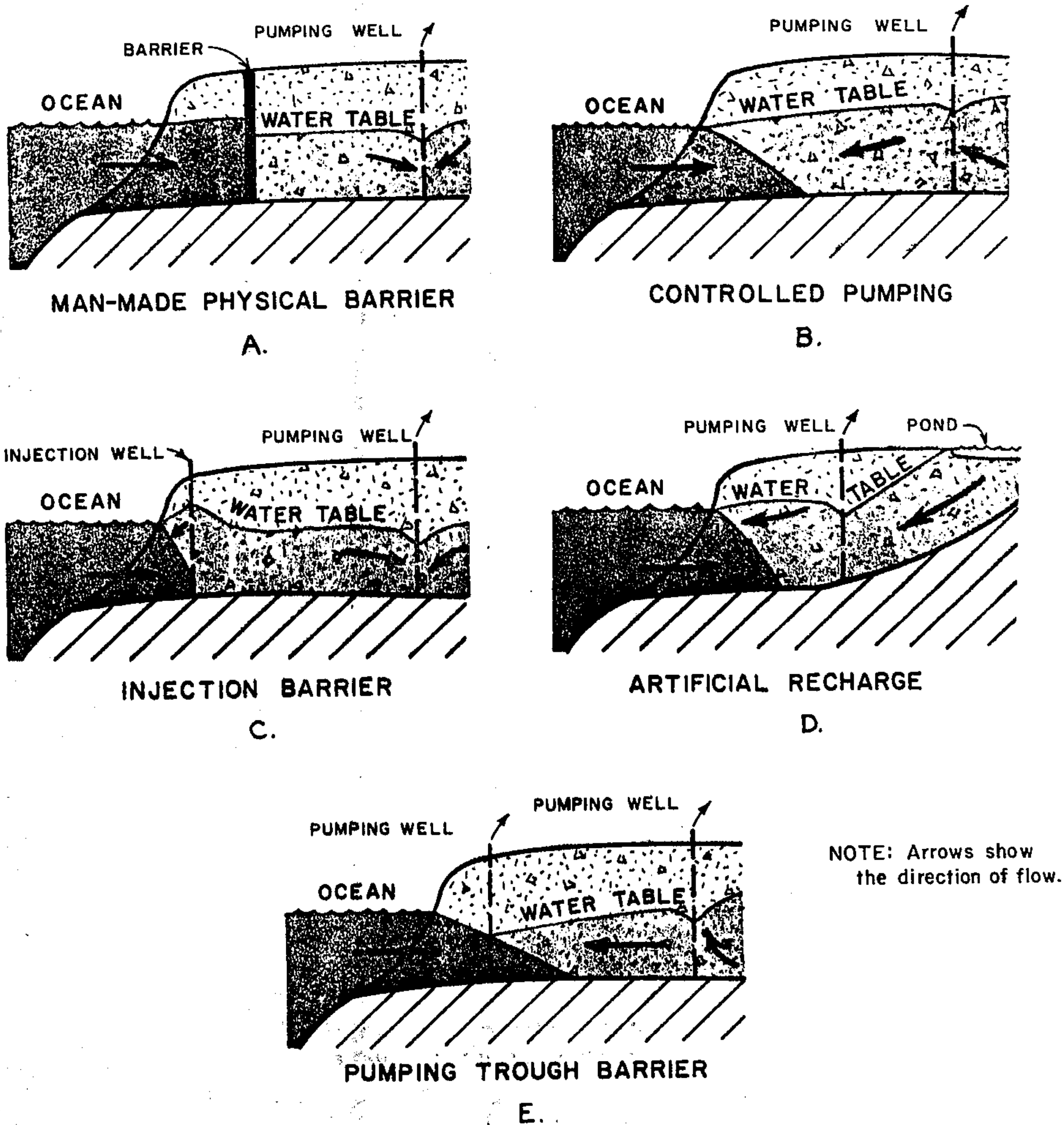
Model 4, shown in Figure 48, illustrates the long term effects of a physical barrier. If there is continued drainage around the constructed area of the semi-depressed freeway, the interface could continue to move until gravity drainage is established between the freeway subsurface drainage system and the closest aquifer. The physical barrier can stop gravity seepage due to the advance of the interface but cannot prevent gravity drainage. (Figure 30 graphically defines gravity seepage and gravity drainage.)

Potential Protective Measures

The three conditions that signal the beginning of an intrusion problem are: ground water levels falling below sea level, landward hydraulic gradients being established, and increase in mineral constituents of ground water. When such conditions become evident corrective steps must begin unless degradation of ground water quality is not considered important.

Several methods of repelling saline invasion shown in Figure 49. They are briefly described below:

POTENTIAL PROTECTIVE MEASURES



SEA WATER INTRUSION STUDY, RICHMOND SEMI-DEPRESSED SECTION

FIGURE 49 REFERENCE 17, PAGE 394

(a) Water Conservation (Controlled Pumping). This method involves convincing all users of ground water in the project area of the urgent need to reduce usage. If water use is curtailed the likelihood of sea water encroachment is lessened, or prevented for a long time.

(b) Limitation of Ground Water Extraction. If alternate sources of water are developed this would reduce dependency on ground water supplies. Alternative sources might consist of imported water, or reclaimed municipal waste water.

(c) Recharging Operations Using Reclaimed Waste Water. If this alternative is considered, adequate provisions should be made to ensure approximate water quality for the intended use(18).

(d) Physical Barrier. Construction of an impermeable membrane between the bay and the semi-depressed freeway is feasible for this project because of the thin upper aquifers. This procedure has been recommended for this project in the Section: "Methods of Dewatering". In this case the physical barrier could consist of a slurry wall extending to the bottom of the first aquifer. As shown in Model 4 (Figure 48), this would permit considerable latitude in allowing sea water encroachment. The sea water can be permitted to meet the physical barrier and ground water pumped to allow that condition to develop. On a long term basis, even this method may not be able to prevent ground water - sea water gravity drainage.

(e) Artificial Recharge by Surface Spreading. The presence of extensive permeable forebays suitable for recharge and well defined permeable aquifers capable of transmitting and storing large quantities of ground water is a prime requisite. As this study did not cover areas beyond the vicinity of the project area, the presence of extensive permeable forebays was not established. During the subsurface investigation no well defined aquifers were located. Hence, this method could not be recommended for this project.

(f) Injection Barrier (Hydraulic Barrier). This method involves the use of a line of injection wells separating the bay from the semi-depressed freeway. This requires an alternate source of water for operating the injection wells. The injected water would create a hydraulic pressure barrier that would restrain a landward advance of sea water. This method allows great flexibility of ground water level fluctuation below sea level in the inland area.

(g) Pumping Trough. This method involves continuous maintenance of a line of control pumping wells between the bay and the water supply wells in the area of the project. This method is very expensive, and much usable water would be wasted. The success of this method depends on two factors, namely: monitoring ground water levels at the trough; and determining the amount of water to be pumped at the control wells. This method is not recommended for this project.

(h) Combination Barrier. This method combines the injection barrier and the pumping trough. The pumping trough would be developed on the bay side, with the injection barrier installed on the landward side. Although this method would offer greater flexibility in managing ground water levels, it is a very expensive alternative.

Effect of Freeway Construction

Sea water encroachment already exists in the westerly end of this project where ship channels are situated. The depth of fresh water is estimated at about 100 feet. Even this fresh water was found to be slightly contaminated. During construction, between Stations 221+00 and end of project, the normally bayward direction of flow of ground water will be reversed to landward flow. After construction this trend would be reversed and original conditions restored between Stations 221+00 and 238+50; whereas between Station 238+50 and end of the project, the direction of flow will be permanently landward. The import of this is that one of the most important causes of sea water encroachment will be established. This, added to any increase in demand on local aquifer-supplied water or drought-related lowering of ground water levels due to slow recharge, would add to these problems. The use of a physical barrier on the bayward side as recommended in this section would mitigate most of this encroachment and control it within the confines of the semi-depressed freeway section.

GROUND WATER QUALITY

A three-phase program to determine the present ground water quality was started during July, 1978. The objective was to determine the potability of ground water that would be removed both during and after construction, as well as to define potential uses of the ground water in its present condition. In the following paragraphs a partial summary of the results of tests conducted under Phase I is reported. Sampling for Phase II was completed during August, 1978, and the laboratory tests are in progress. Phase III sampling will be done in September, 1978, and a supplemental report will be prepared summarizing test data from all three phases.

Some of the tests are being conducted at the State Department of Public Health Laboratory, in Berkeley. The remainder are being performed at the Transportation Laboratory in Sacramento.

Standards Used

The California Domestic Water Quality and Monitoring Regulations(19) published by the State Department of Health were used to assess present water quality. As a supplemental source, proposed interim standards for safe drinking water as required by the Safe Drinking Water Act (PL 93-523), published by the Federal Environmental Protection Agency, were used. The California Department of Health water quality standards are presented in Tables 29 through 32. The Interim Primary Standards for safe drinking water according to the Safe Drinking Water Act PL 93-523 are summarized in Table 33.

CALIFORNIA DOMESTIC WATER QUALITY STANDARDS (1977)

TABLE 29 MAXIMUM CONTAMINANT LEVELS INORGANIC CHEMICALS

Constituent	Maximum Contaminant Level, mg/l
Arsenic	0.05
Barium	1.
Cadmium	0.010
Chromium	0.05
Lead	0.05
Mercury	0.002
Nitrate (as NO_3)	45.
Selenium	0.01
Silver	0.05

TABLE 30 MAXIMUM CONTAMINANT LEVELS ORGANIC CHEMICALS

Constituent	Maximum Contaminant Level, mg/l
(a) Chlorinated Hydrocarbons	
Endrin	0.002
Lindane	0.004
Methoxychlor	0.1
Toxaphene	0.005
(b) Chlorophenoxys	
2, 4-D	0.1
2, 3, 4 - TP Silvex	0.01

- pp1708 and 1717 of Ref (19)

CALIFORNIA DOMESTIC WATER QUALITY STANDARDS (1977)

TABLE 31 CONSUMER ACCEPTANCE LIMITS-SECONDARY DRINKING WATER STANDARDS

Constituents	Maximum Contaminant Levels
Color	15 Units
Copper	1.0 mg/l
Corrosivity	Relatively low
Iron	0.3 mg/l
Manganese	0.05 mg/l
Odor - Threshold	3 Units
Foaming Agents (MBAS)	0.5 mg/l
Turbidity	5 Units
Zinc	5.0 mg/l

TABLE 32 MINERALIZATION-SECONDARY DRINKING WATER-STANDARDS

	Maximum Contaminant Levels		
Constituent, Units	Recommended	Upper	Short Term
Total Dissolved Solids, mg/l	500	1,000	1,500
or			
Specific Conductance, micromhos	900	1,600	2,200
Chloride, mg/l	250	500	600
Sulfate, mg/l	250	500	600

- pp 1708 and 1717 of Ref (19)

TABLE 33 PROPOSED INTERIM PRIMARY STANDARD FOR SAFE DRINKING WATER (As Required by Safe Drinking Water Act PL 93-523)

Organic Chemicals (mg/l)

Chlorinated Hydrocarbons

Chlordane	0.003
Endrin	0.0002
Heptachlor	0.0001
Heptachlor Epoxide	0.0001
Lindane	0.004
Methoxychlor	0.1
Toxaphene	0.005

Chlorophenoxys

2, 4-D	0.1
2, 4, 5-TP Silvex	0.01

Microbes

When the membrane filter technique is used, coliform densities may not exceed one per 100 milliliters as arithmetic mean of all samples examined per month.

For the fermentation tube method, coliforms shall not be present in more than 10% of the portions in any month.

No greater than 500 organisms per one milliliter (as determined by the standard bacterial plate count) will be allowed.

Inorganic Chemicals (mg/l)

Arsenic	0.05
Barium	1.00
Cadmium	0.010
Chromium	0.05
Cyanide	0.20
Lead	0.05
Mercury	0.002
Nitrate	10.00
Selenium	0.01
Silver	0.05

Fluorides (mg/l)

50.0 - 53.7°F	2.4
53.8 - 58.3°F	2.2
58.4 - 63.8°F	2.0
63.9 - 70.6°F	1.8
70.7 - 79.2°F	1.6
79.3 - 90.5°F	1.4

Turbidity

The level at representative entry points to the distribution system is one turbidity unit except that five or fewer turbidity units may be allowed if the supplier can demonstrate the higher turbidity does not:

Interfere with disinfection
Prevent maintenance of an effective disinfectant agent through the distribution system
Interfere with microbiological determinations

Ref: Page 10, American City - May 1975

Test Conducted Under Phase I

The State Department of Public Health Laboratory in Berkeley conducted tests for hydrocarbons and bateriological content. The Transportation Laboratory in Sacramento conducted tests for heavy metals, minerals, pH, and electrical conductivity.

A Mar-Tek Mark V water quality analyzer was used in the field to determine pH, dissolved oxygen, electrical conductivity, and temperature.

Tabulation of Results

Four ground water samples were taken during the 1977 pump tests. Test results from those samples are presented in Table 34. Laboratory test results from the sea water intrusion study been presented earlier in Table 25 and 26.

The various hydrocarbons are presented in Table 35. The inorganic chemicals are summarized in Tables 36 and 37. Table 38 presents the total dissolved solids in the nine samples tested. Table 39 contains statements on the pesticides and herbicides analyses. The bacteriological quality tests are posted in Table 40.

TABLE 34 CONDITION OF WATER DURING 1977 PUMP TESTS

Pump Test Number and Location	Date of Sampling	Sample No.	Conductivity μ mhos/cm	pH	Cl_2 mg/l	NO_3-N mg/l	CO_3 mg/l	HCO_3 mg/l	SO_4 mg/l	Cl_2/ CO_3+HCO_3
1 Hoffman Blvd. and 11th St.	1-11-77	11-1	880	7.7	145	3.7	0	345	120	.42
		11-2	760	7.7	180	4.1	0	370	140	.49
		11-3	740	7.7	195	4.2	0	365	150	.53
2 Hoffman Blvd. and 30th St.	1-30-77	30.1	3200	7.9	14	0.6	0	150	30	.09

TABLE 35

Ground Water Quality Studies, Phase I

Organic Chemicals

Area	Station	Boring	Chlorinated Hydrocarbons: Endrin	Lindane	Methoxychlor	Toxaphene	Chlorophenoxys: 2, 4 - D	2, 4, 5 - TP Silvex
			Calif. Standard - .002 mg/l	Calif. Standard - .004 mg/l	Calif. Standard 0.1 mg/l	Calif. Standards - .005 mg/l	Calif. Standards - 0.1 mg/l	Calif. Standard - 0.01 mg/l
			PL-93-523 - .0002 mg/l	PL-93-523 .004 mg/l	PL-93-523 0.1 mg/l	PL-93-523 .005 mg/l	PL-93-523 0.1 mg/l	PL-93-523 0.01 mg/l
			Actual	Actual	Actual	Actual	Actual	Actual
1	177+00	P11	.00001	.00001	.0001	.0003	.00002	.00001
1	181+20	D16	.00001	.00001	.0005	.0006	.00002	.00001
2	187+80	P10	.00008	.00001	.0002	.0006	.0002	.00001
2	201+00	P 9	.00001	.00001	.0003	.0001	.0002	.00001
3	220+10	P 7	.00001	.00001	.0005	.0006	.00002	.00001
3	228+50	P 6	.00001	.00008	.0003	.0001	.00002	.00001
4	249+80	P 4	.00001	.00001	.0003	.0001	.00002	.00001
6	272+00	P 2	.00001	.00004	.0003	.0001	.00002	.00001
6	274+15	P13	.00001	.00001	.0005	.0006	.00002	.00001
Comments			Meets Standards	Meets Standards	Meets Standards	Meets Standards	Meets Standards	Meets Standards

TABLE 36 INORGANIC CHEMICALS IN GROUNDWATER

Area	Station	Boring	(1) Nitrate mg/l		(2) Copper mg/l		(3) Manganese mg/l		(4) Specific Conductance μmho/cm				(5) Chlorides, mg/l				(6) Sulfates, mg/l			
			Max. Level	Actual	Max. Level	Actual	Max. Level	Actual	Recommended	Upper	Short-term	Actual	Recommended	Upper	Short-term	Actual	Recommended	Upper	Short-term	Actual
1	177+00	P11	45	19	1.0	.04	.05	1.9	900	1600	2200	680	250	500	600	60	250	500	600	65
1	181+20	P16		28		.00		.73				740				65				93
2	187+80	P10		24		.02		.14				740				67				38
2	201+00	P9		32		.02		.11				1370*				116				139
3	220+10	P7		18		.00		.20				530				84				42
3	228+50	P6		27		.02		.20				900				87				67
4	249+80	P4		24		.01		.42				670				73				39
5	272+00	P2		41		.02		.40				1060*				141				70
6	274+15	P13		46		.00		1.3				1330*				172				123
Comments			Meets Standards		Meets Standards		Does Not Meet Standards		*Does Not Meet Standards	Meets Standards			Meets Standards				Meets Standards			
Standard			Inorganic Chemical		Secondary Drinking		Secondary Drinking		Secondary Drinking Water Standards				Secondary Drinking				Secondary Drinking			

TABLE 37, MINERAL ANALYSIS

Boring No.	Calcium mg/l	Magnesium mg/l	Bicarbonate mg/l
P11	50	26	185
P16	55	25	180
P10	60	29	270
P9	127	50	425
P7	47	17	250
P6	73	29	285
P4	55	19	190
P2	84	27	270
P13	98	33	305

Note:

Calcium Ref: Table 17, Vol. V, Ref. 20

1. Calcium is beneficial to humans.
2. No standards in USPHS 1962 drinking water standards.
3. Calcium desirable in irrigation water.
4. It should be <52 mg/l for fish and aquatic life.

Magnesium Ref: Table 27, Vol. V, Ref. 20

5. No limit in 1962 USPHS Drinking Water Standards (500 mg/l, taste threshold for humans).
6. Industrial 5 mg/l to 30 mg/l.
7. Fish and aquatic life 14 mg/l.

TABLE 37 (Continued)

<u>Bicarbonate</u> Ref: Table 12, Vol. V Ref. 20

8. For municipal drinking use, it should be <150 mg/l and for washing <60 mg/l.

9. For industrial use, <100 mg/l.

10. For irrigation, not harmful, but can affect Na ratio (formation of $CaCO_3$).

11. For fish and wild life it should be <180 mg/l.

<u>Carbonate</u> (CO_3)

12. No carbonate was present in any of the samples.

13. No standards exist for drinking, or fish and wildlife.

TABLE 38 GROUND WATER QUALITY ANALYSIS - PHASE I

Total Dissolved Solids

Area	Station	Boring No.	Total Dissolved Solids mg/l
1	177+00	P11	456
1	181+20	D16	504
2	187+80	P10	543
2	201+00	P9	998
3	220+10	P7	430
3	228+50	P6	635
4	249+80	P4	451
6	272+00	P2	724
6	274+15	P13	917

NOTE: Calif. standards for allowable total dissolved solids (mg/l) in secondary drinking water are:

500 (Recommended), 1000 (Upper), 1500 (Short Term)

Potential Uses in Present Condition

The test results presented in Table 34 indicate that the ground water at the site of Pump Test 1 has a tendency to be brackish in taste whereas at the site of Pump Test 2 the water was found to be fresh. The various organic contaminants, both chlorinated hydrocarbons and chlorophenoxys, are presented in Table 35. Current standards for drinking water are not exceeded.

Referring to Table 36, the nitrates, clorides, sulfates, and copper concentrations are within allowable limits for drinking water. The manganese content is fairly high ranging from 0.11 mg/l to 1.9 mg/l. The allowable limit is 0.05 mg/l. For irrigation purposes and to encourage plant growth the permissible level of manganese is 0.5 mg/l. It will be harmful to plant life if greater than 0.5 mg/l. Trout can survive up to 7 days where the manganese content is 15 mg/l. River crayfish can tolerate only up to 1 mg/l. Thus, the Manganese content does not meet the standards for secondary drinking purposes. Three of the nine samples have specific conductance values greater than the recommended 900 micromhos. But these values are lower than 1600 micromhos which is the upper limit for purposes of secondary drinking.

Table 37 presents contents of calcium, magnesium and bicarbonates. At present there are no standards for calcium for drinking purposes. In fact, it is beneficial to humans. This is desirable in irrigation waters. To support fish and aquatic life 95% of good fish water should contain less than 52 mg/l. In this case it varies from 50 mg/l to 127 mg/l. Hence it is doubtful whether it can support fish and aquatic life. There are no limits for magnesium for drinking purposes. It varies from 17 mg/l to 50 mg/l. At a level of 500 mg/l of magnesium humans begin to detect its taste. For industrial purposes the suggested range is 5 mg/l to 30 mg/l. To support fish and aquatic life it should be less than 14 mg/l. The bicarbonate values range from 180 mg/l to 425 mg/l. For washing purposes it should be less than 60 mg/l, for drinking purposes it should be less than 150 mg/l and for industrial use it should be less than 100 mg/l. To support fish and aquatic life it should be less than 180 mg/l.

From Table 38 it can be seen that the total dissolved solids range from 451 mg/l to 998 mg/l. For drinking water the recommended value is 500 mg/l and only 3 samples had values less than 500 mg/l. The upper allowable limit is 1,000 mg/l and 1,500 mg/l for short term. Ninety-five percent of water supporting fish fauna contains less than 400 mg/l, although most aquatic forms will not be affected up to a limit of 2,000 mg/l.

There were no pesticides or herbicides found in the ground water sample as presented in Table 39.

The coliform density varied from 70 MPN/100 ml to greater than 24,000 MPN/100 ml, as shown in Table 40. According to California standards it should not exceed 1 MPN/100 ml. For municipal water supply the desirable criterion is less than 100 MPN/100 ml with a permissible criterion of 10,000 MPN/100 ml. Also it should be noted the coliform density can depend upon the variation of temperature.

Based on the California Department of Health drinking water standards and keeping in mind the objectives for San Francisco Bay tidal waters required by the San Francisco Bay Basin Water Quality Control Plan, Gary Winters of the Water Quality Unit at the Transportation Laboratory reviewed the test data for Phase I and summarizes his conclusions as follows(21):

"Review of the analyses indicates a couple of areas of concern regarding the water quality of samples analyzed:

1. Dissolved oxygen levels ranged from 1.2-4.2 mg/l, averaging 2.1 mg/l. The Water Quality Control Plan objective is to maintain a minimum of 5.0 mg/l in the Bay below the Carquinez Bridge and in all aquatic life habitats of the San Francisco Bay. The input from the dewatering of this project would not meet this standard.

2. The total coliform analysis on the water samples derived by the Multiple Tube Technique indicates the samples were moderate to severely contaminated. The MPN (Most Probable Number) for the samples

TABLE 39, PESTICIDES AND HERBICIDES ANALYSES

LABORATORY REPORT

State of California - Department of Public Health

Sanitation and Radiation Laboratory

2151 Berkeley Way, Room 235, Berkeley 94704

Lab No. 20-28

Description of sample Well   Place collected Richmond Depressed Section

Collector Caltrans   Report to Caltrans

Date collected 7-18-78 Date received 7-18-78 Date reported 7-27-78

Pesticides Analysis:

Extract 1/2 gallon with 20 ml. Benzene, inject into Glc using (1.5% OV 17 + 1.95% OV 210 and 5% OV 210) as column.

Herbicides

Same as standard methods for the examination of water and waste water using the same above columns for the analysis by Glc.

None of either the pesticides or the herbicides was found in these samples #20-28.

Zene Jasaitis
Saufi Khalifa

Form LAB-1418 LM 8/25/70

TABLE 40, BACTERIOLOGICAL QUALITY

Area	Station	Boring	Coliform Density		
			Calif Std.	PL -93-523	Actual
1	177+00	P11	1.0 MPN/100 ml	1.0 MPN/100 ml	9200 MPN/100 ml
1	181+20	D16			790
2	187+80	P10			1100
2	201+00	P 9			≥ 24000
3	220+10	P 7			130
3	228+50	P 6			330
4	249+80	P 4			230
6	272+00	P 2			70
6	274+15	P13			790
Comments					Does not meet standards

Note: MPN = Most Probable Number

ranged from 70 to 24,000 (Ave. 4,071) bacteria per 100 ml sample. The California Department of Health standards indicate 2.2-9.2 bacteria per 100 ml sample as the maximum for potable water. Additionally, levels of bacteria in some of the samples exceed the Water Quality Objectives for Coliform Bacteria for contact recreation uses in the San Francisco Bay Region. In the Bay tidal waters, total coliform counts should not exceed a median of 240/100 ml with no sample exceeding 10,000/100 ml.

The serious bacterial contamination will have to be addressed prior to export of the effluent anticipated from the project. Whether the high counts are seasonal, from existing fill material, or some type of external sources, e.g. animal droppings, etc. into the wells, would require further investigation.

Heavy metals reported so far (Cu, Fe, Hg, and Mn) do not present a water quality problem at the levels detected. The additional heavy metals parameters currently being analyzed should also be reviewed. Major ions such as Ca, Cl, $H1O_3$, Mg, Na, and SO_4 show levels that are not excessive. The Department of Health tested for hydrocarbons (pesticides and herbicides) and no contamination was found.

It should be noted nitrate concentrations in the P-13 boring were slightly above the maximum 45 mg/l standard for drinking water. More importantly, the nitrates levels are relatively high in numerous wells and should be addressed when disposal of the dewatering effluent is considered. The input of these nitrate levels may be deleterious to Bay algal populations."

CONCLUSIONS AND RECOMMENDATIONS

Results of Investigations

Based upon extensive study of boring profiles, visual identification of soil samples, and laboratory tests, the existance of a multiple aquifer system with perched water conditions was confirmed. This aquifer system had both artesian and water table conditions. Two models of existing subsurface conditions are proposed.

A ground water reconnaissance survey disclosed the fact that many private wells, in addition to cathodic protection wells, exist in the project vicinity. Only one well is currently used for drinking purposes. Most of the wells are inoperational. In areas near the U.C. Richmond Field Station private companies use ground water from shallow aquifers (within 25 feet) as well as from deeper aquifers (90 to 125 feet). Ground water over the entire project area was found to be fresh with a slight contamination by sea water near the westerly end.

A literature survey to clarify the term "Richmond Aquifer" revealed that confusion presently exists concerning its identity and location. To avoid future confusion, names for the ground water aquifers encountered and the basin containing them are proposed.

Conclusions From Drawdown Analysis

The aquifers had a system transmissibility factor ranging from 194 gpd/ft to 5247 gpd/ft. The system storage coefficient values ranged from 0.000116 to 0.00795. These values indicate the yield of these aquifers may not be adequate for industrial, municipal, or irrigation purposes and confirmed the existance of artesian conditions. Slow recharge occurs in the project vicinity.

The total pumpage would vary depending upon the combinations of dewatering components adopted. Assuming two rows of deep wells, one on either side of the semi-depressed section, the total pumpage during construction (short term) would be between 2.5×10^6 and 3.5×10^6 gpd. Assuming a good pavement subsurface drainage system with intercepting trench on the landward side and a physical barrier on the bayward side the quantity to be removed after construction (long term) would be a maximum of 1.4×10^6 gpd. The estimated quantities for both short term and long term conditions represent upper limits. Actual quantities may be substantially lower, depending upon the soil conditions encountered and the dewatering components adopted.

Suggested Dewatering Components

Three dewatering components: namely, deep wells, interceptor trenches, and physical barriers, are suggested for use in combination during construction. Of the four alternates previously discussed the most useful combination appears to be a longitudinal interceptor trench on the landward side with a physical barrier on the bayward side as construction proceeds. Any seepage from the bottom of excavation could be removed by sumps and pumps. If, in certain locations, the capacity of the above recommended dewatering systems is exceeded, supplemental pump wells could be introduced for the particular area.

Five dewatering components: namely, longitudinal trenches physical barriers, pavement drainage system with longitudinal drains and lateral drains, observation wells, and collection stations are suggested for use after construction. Most of the seepage from the bay would be blocked by the physical barrier and the interceptor trench would collect the seepage from the landward side. The gravity drainage from the bottom of the semi-depressed section would be intercepted by a system of blanket drains, and longitudinal and lateral drains.

Effects of Dewatering

The maximum area affected by any settlement would comprise a maximum distance of 300 ft outside the right of way on either side. Maximum settlement would be in the order of 2.2 inches nearest the excavation. Differential settlements will be minimal. A condition survey of sensitive structures in the project vicinity before, during, and after construction is suggested.

Sea water intrusion already exists at the westerly end of the project due to the presence of ship channels and man-made fill over bay muds. Construction of the proposed semi-depressed section would induce sea water encroachment over a certain length of the project both during and after construction. This sea water intrusion would be stabilized at the semi-depressed section after a period of time subsequent to construction. The saline waters intercepted by the drainage system probably would be discharged into the bay.

Ground Water: Quality and Potential Uses

Work on Phase I sample is still in progress. Even though there are no hydrocarbons, pesticides, or herbicides in the groundwater it has high manganese content and high coliform count. In its present condition the ground water is not potable according to California Department of Health Standards. The apparent bacterial contamination of this water would preclude its use for support of fish and aquatic life. Heavy metals analyzed to date indicate no ground water quality problem. The high nitrate content in some samples may be injurious to algae.

These conclusions on ground water quality should be considered preliminary as the date of this report. A supplemental report will be prepared after Phases II and III sampling and testing are completed.

Note: In addition to the foregoing conclusions and recommendations reference is made to Appendix D for a discussion concerning cathodic protection systems located in the project area.

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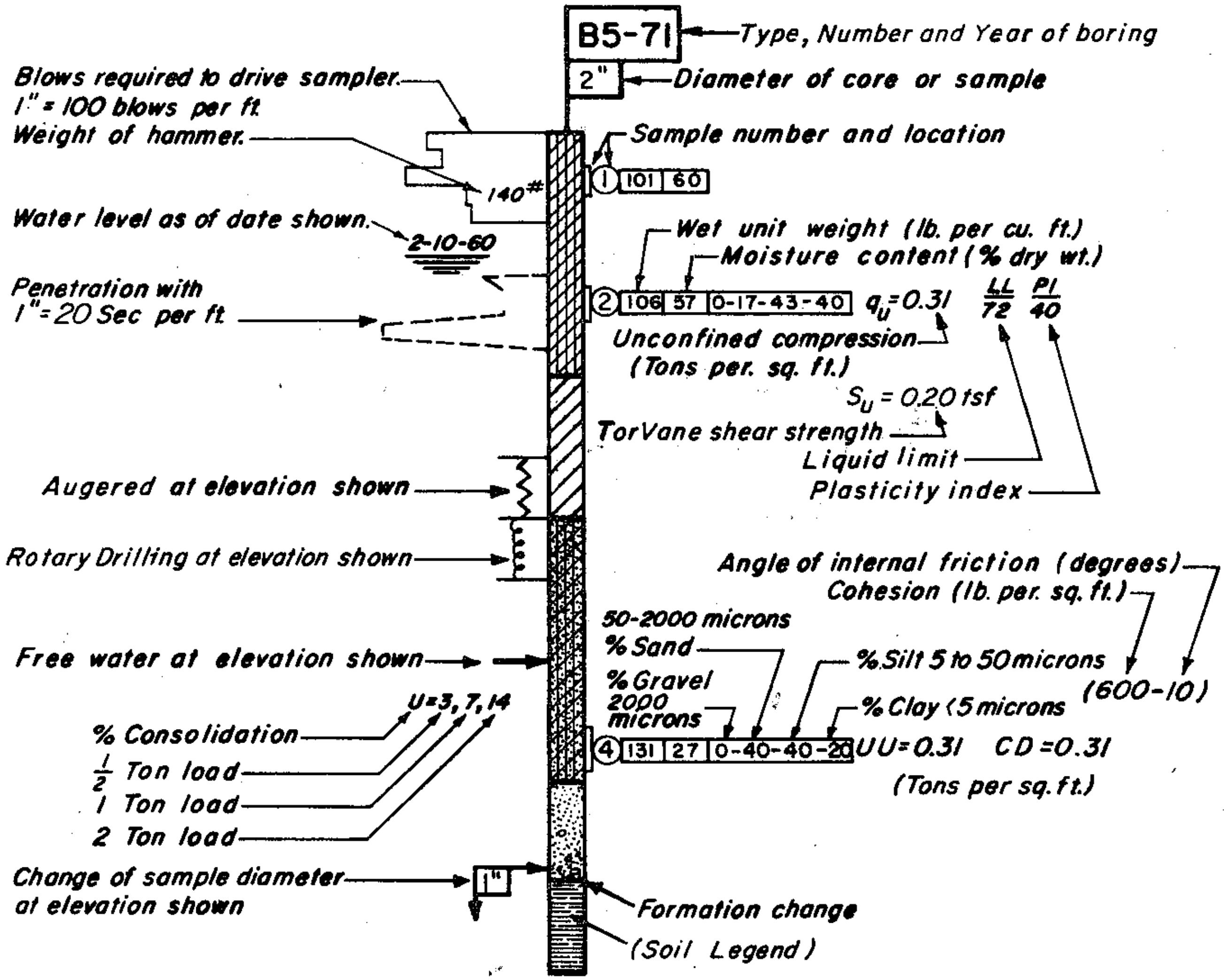
APPENDIX A

BORING PROFILES

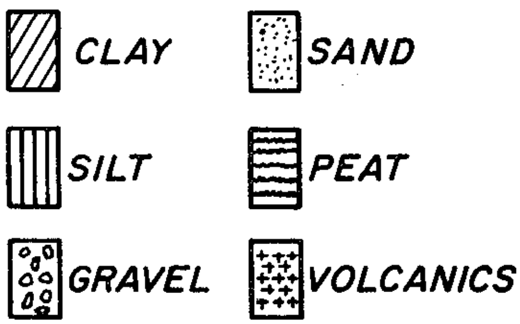
BORING LEGEND
CROSS-SECTION & PROFILE SHEETS



STRENGTH TESTS

S_u - Torvane Shear Strength
q_u - Unconfined Compression
UU - Unconsolidated Undrained
CU - Consolidated Undrained
CD - Consolidated Drained

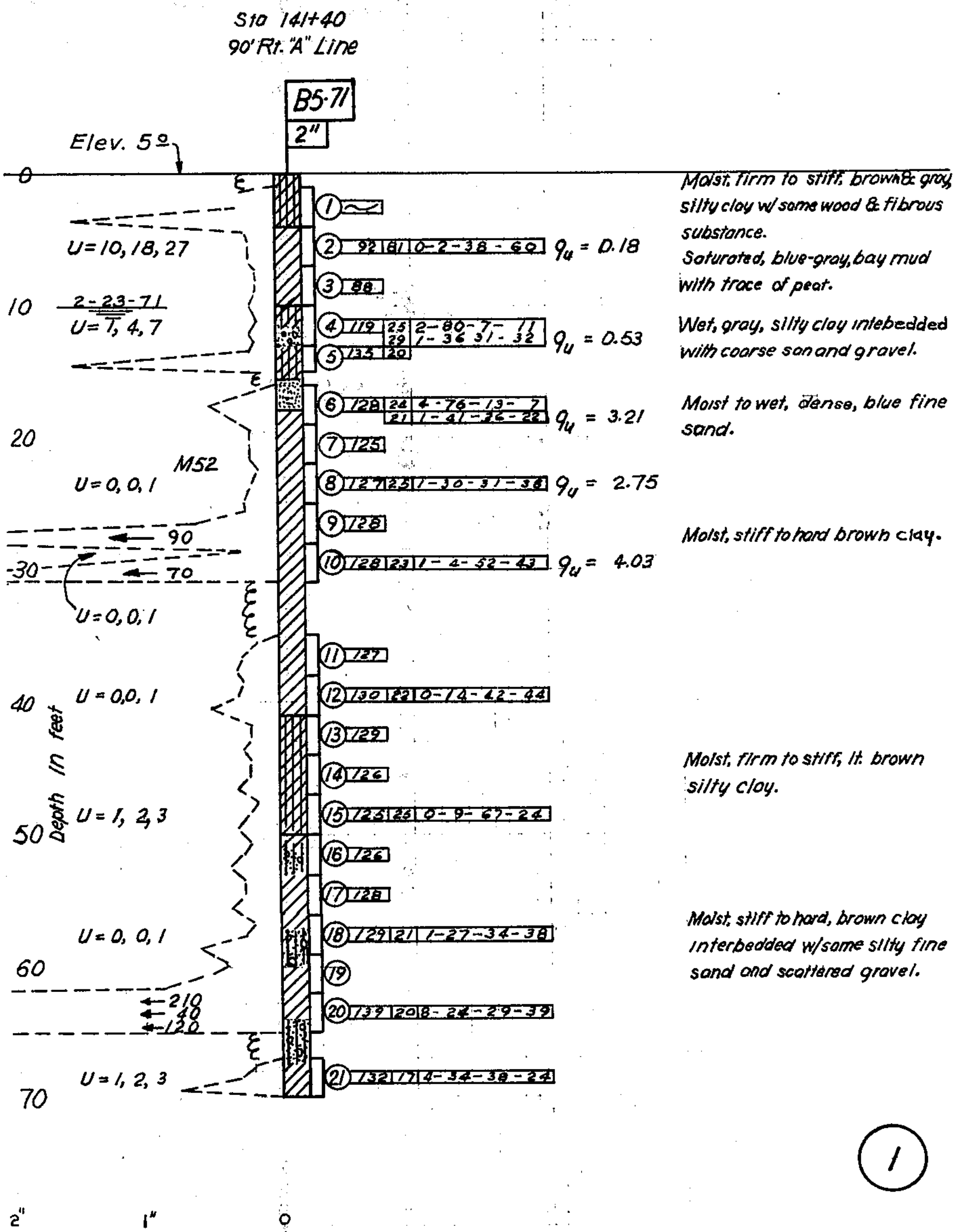
SOIL LEGEND



Note:

B = Translab Boring
P&D = District 04 Boring
Br = Bridge Boring

STATE OF CALIFORNIA — HIGHWAY TRANSPORTATION AGENCY DEPARTMENT OF TRANSPORTATION DIVISION OF CONSTRUCTION TRANSPORTATION LABORATORY		
BORING PROFILES		
DATE	SUBMITTED BY: R. H. Prysock SR. MAT'LS. & RES. ENG'R.	DWG. NO.
DR BY / CK. BY	APPROVED BY: Raymond A. Forsyth SUP'R. MAT'LS. & RES. ENG'R.—GEOT.	SHEET NO. OF SHTS.


Sta 141+40
90' Rt. "A" Line
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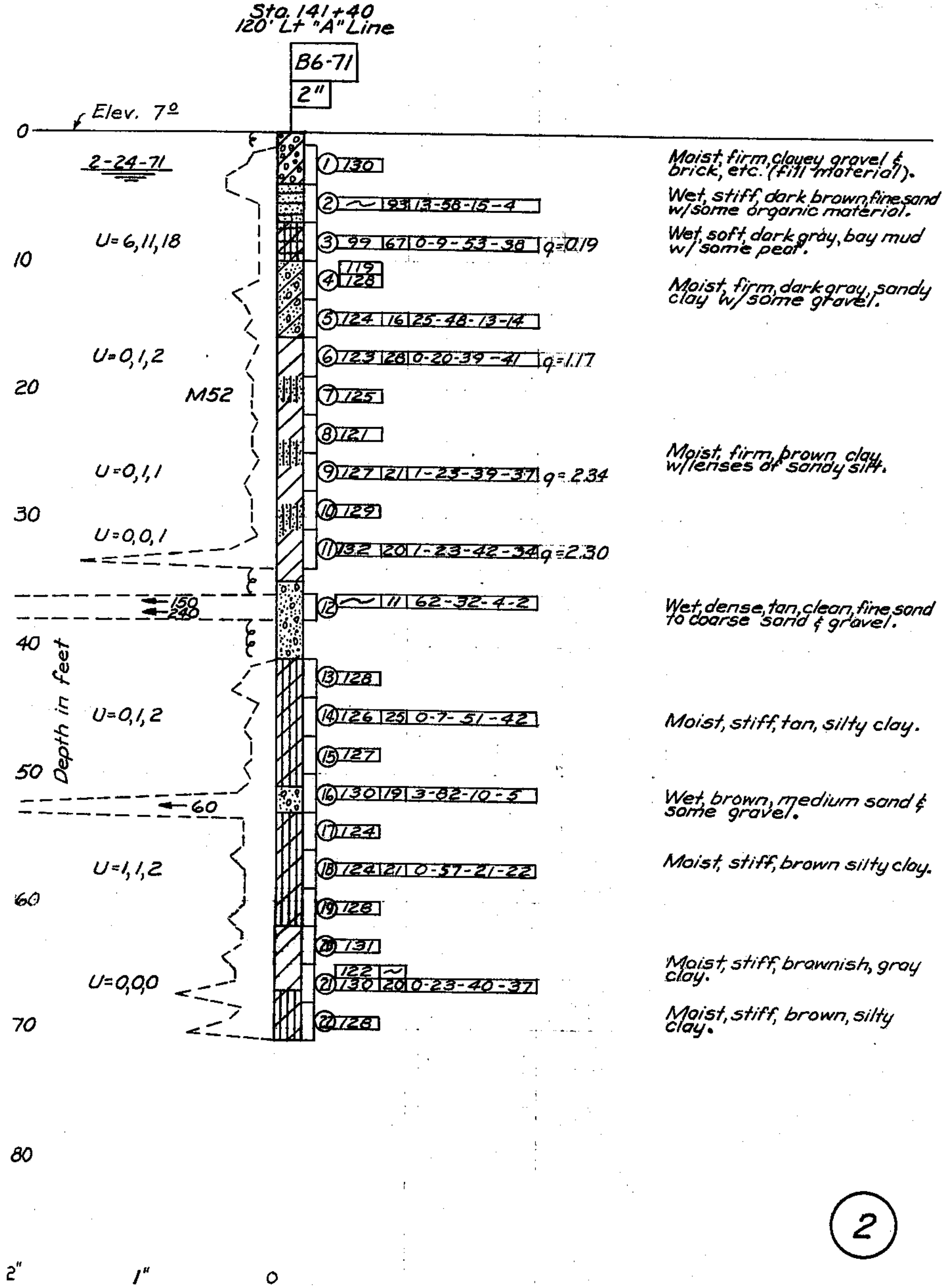
2"
Elev. 5º
0
10
20
-30
40
50
60
70
Depth in feet
U=10, 18, 27
2-23-71
U=T, 4, 7
M52
U=0, 0, 1
90
70
U=0, 0, 1
U=0, 0, 1
U=1, 2, 3
U=0, 0, 1
210
40
120
U=1, 2, 3
1
2 92 61 0-2-38-60 q_u = 0.18
3 88
4 119 25 2-80-7-11
29 1-36 31-32 q_u = 0.53
5 135 20
6 128 24 4-76-13-7
21 1-41-36-22 q_u = 3.21
7 125
8 127 25 1-30-31-38 q_u = 2.75
9 128
10 128 23 1-4-52-43 q_u = 4.03
11 127
12 130 22 0-14-42-44
13 129
14 126
15 125 25 0-9-67-24
16 126
17 128
18 129 21 1-27-34-38
19
20 139 20 8-24-29-39
21 132 17 4-34-38-24
Moist, firm to stiff, brown & gray silty clay w/ some wood & fibrous substance.
Saturated, blue-gray, bay mud with trace of peat.
Wet, gray, silty clay intebedded with coarse san and gravel.
Moist to wet, dense, blue fine sand.
Moist, stiff to hard brown clay.
Moist, firm to stiff, lt. brown silty clay.
Moist, stiff to hard, brown clay interbedded w/ some silty fine sand and scattered gravel.
2"
1"
0

1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M52
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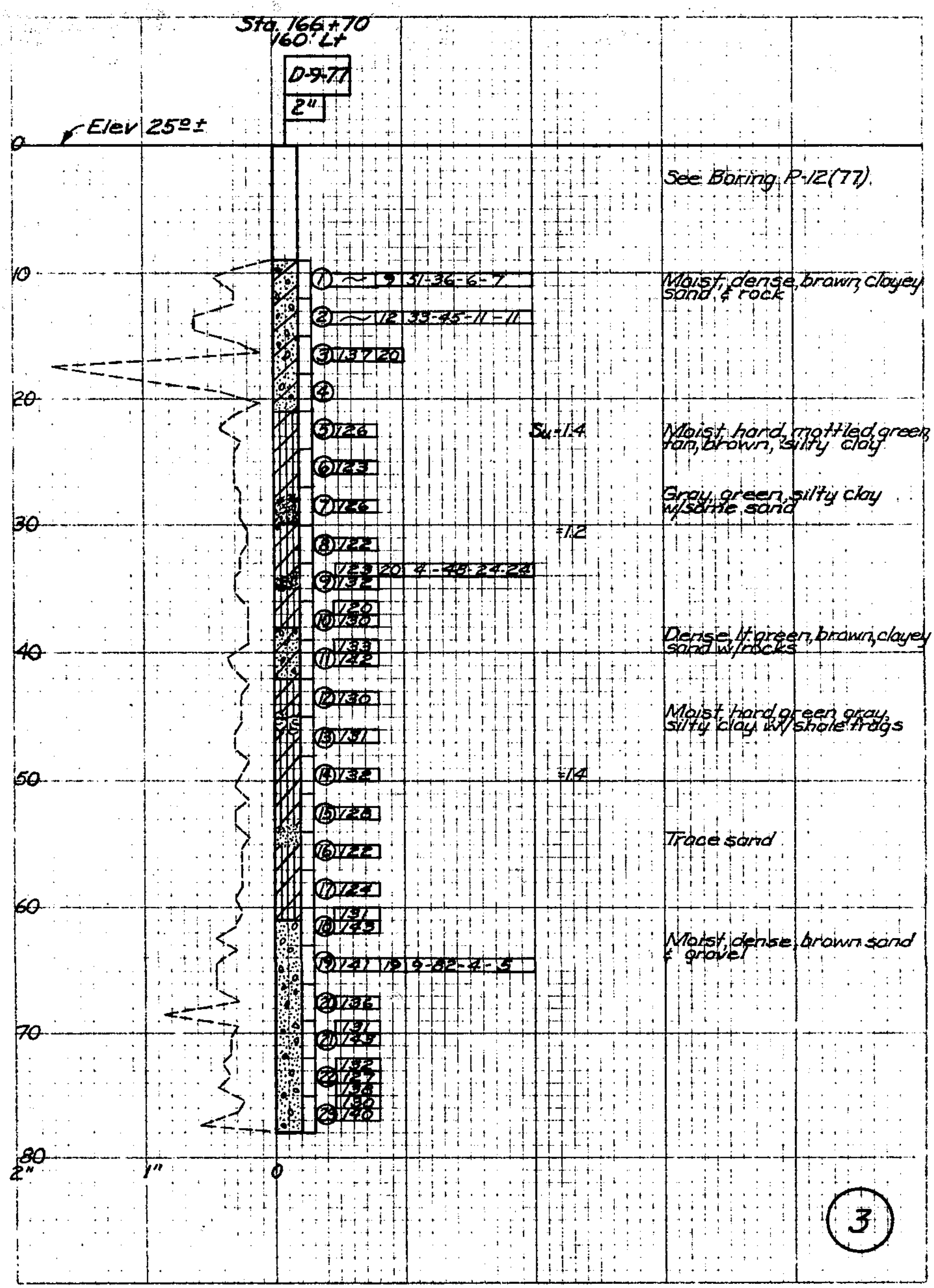
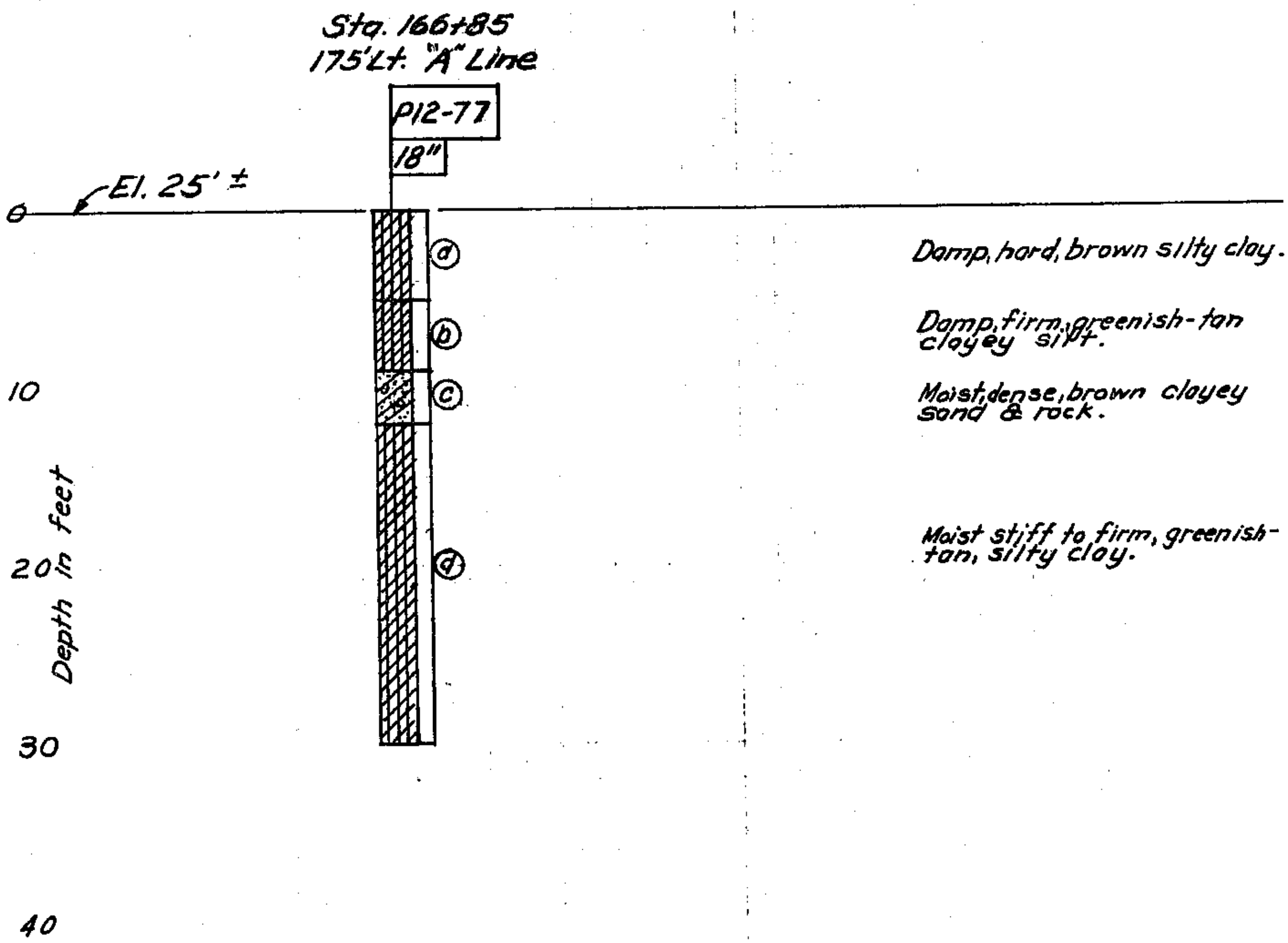
U=0,0,1
150
240
U=0,1,2
60
U=1,1,2
U=0,0,0
Depth in feet
0
10
20
30
40
50
60
70
80
2"
1"
0
① 130
② ~ 93 13-58-15-4
③ 99 67 0-9-53-38 q=0.19
④ 119 128
⑤ 124 16 25-48-13-14
⑥ 123 28 0-20-39-41 q=1.17
⑦ 125
⑧ 121
⑨ 127 21 7-23-39-37 q=2.34
⑩ 129
⑪ 132 20 1-23-42-34 q=2.30
⑫ ~ 11 62-32-4-2
⑬ 128
⑭ 126 25 0-7-51-42
⑮ 127
⑯ 130 19 3-82-10-5
⑰ 124
⑱ 124 21 0-57-21-22
⑲ 128
⑳ 131
㉑ 122 ~ 130 20 0-23-40-37
㉒ 128
Moist, firm, clayey gravel & brick, etc. (fill material).
Wet, stiff, dark brown, fine sand w/some organic material.
Wet, soft, dark gray, bay mud w/some peat.
Moist, firm, dark gray, sandy clay w/some gravel.
Moist, firm, brown clay w/lenses of sandy silt.
Wet, dense, tan, clean, fine sand to coarse sand & gravel.
Moist, stiff, tan, silty clay.
Wet, brown, medium sand & some gravel.
Moist, stiff, brown silty clay.
Moist, stiff, brownish, gray clay.
Moist, stiff, brown, silty clay.
2


Sta. 166+70
160' Lt
D-9-77
2"
Elev 25º±
See Boring P-12(77)
Moist, dense, brown, clayey sand & rock
Moist, hard, mottled green tan, brown, silty clay
Gray, green, silty clay w/some sand
Dense, lt green, brown, clayey sand w/rocks
Moist, hard green gray, silty clay w/shale frags
Trace sand
Moist, dense, brown sand & gravel
Su=1.4
=1.2
=1.4
3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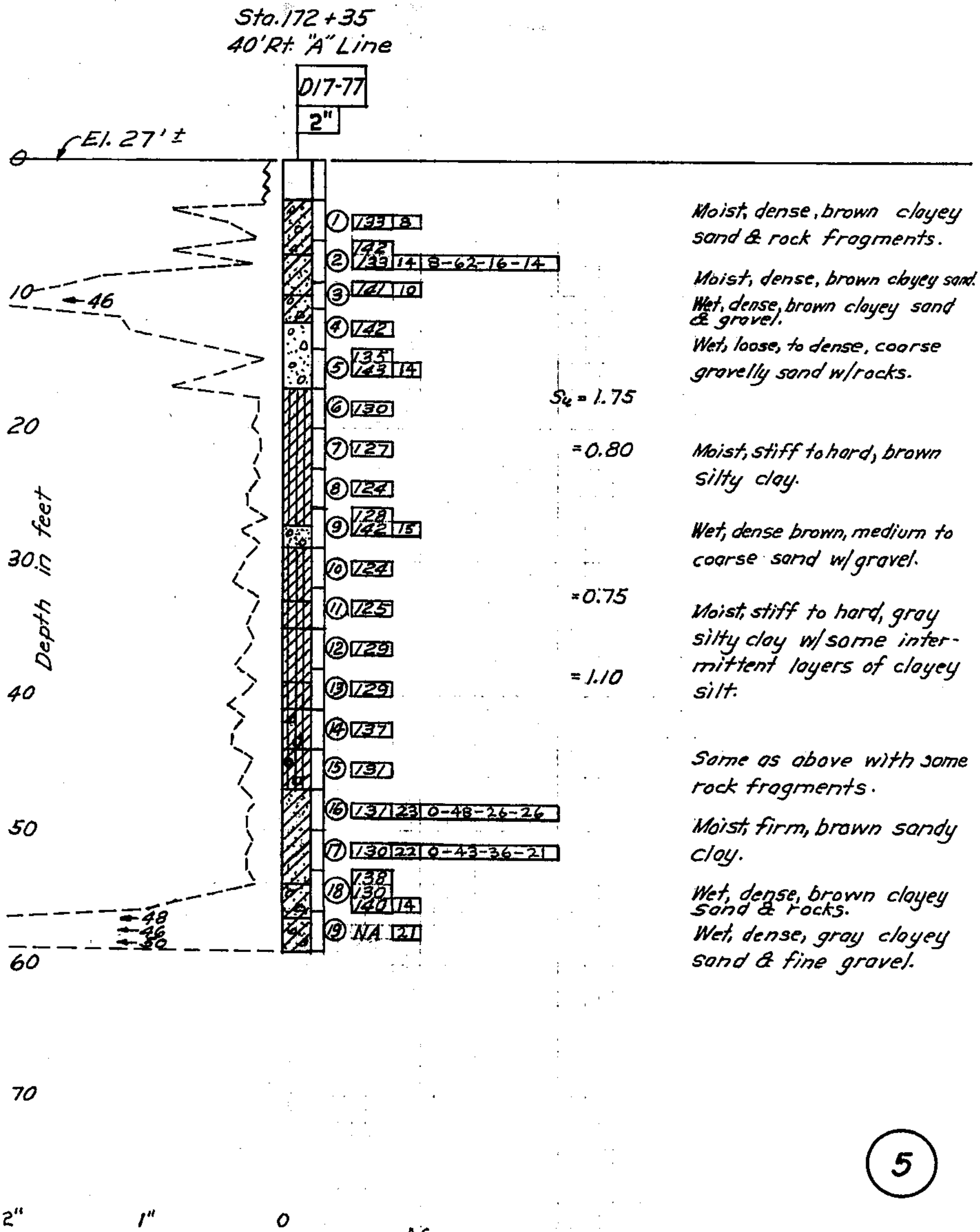
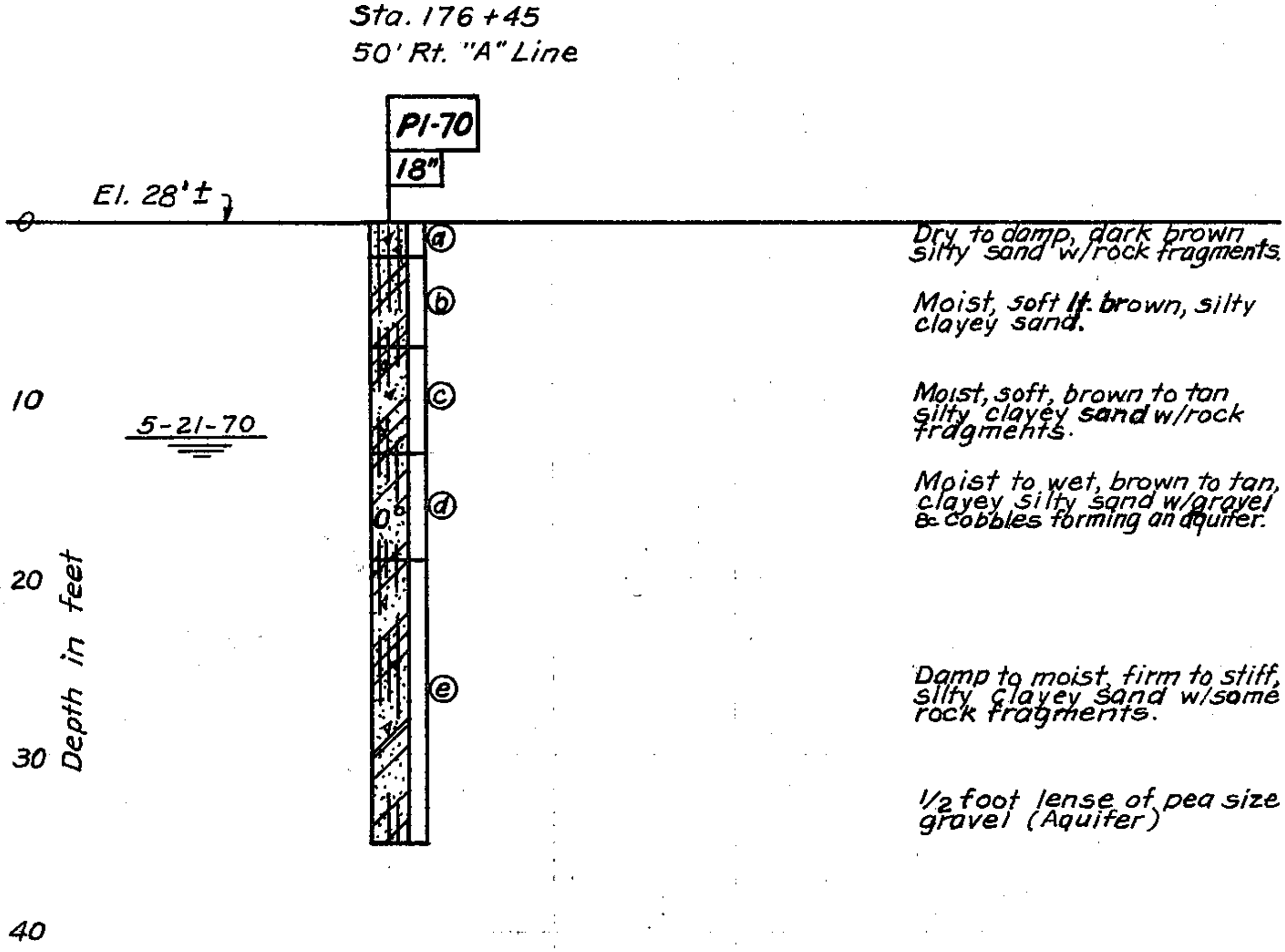
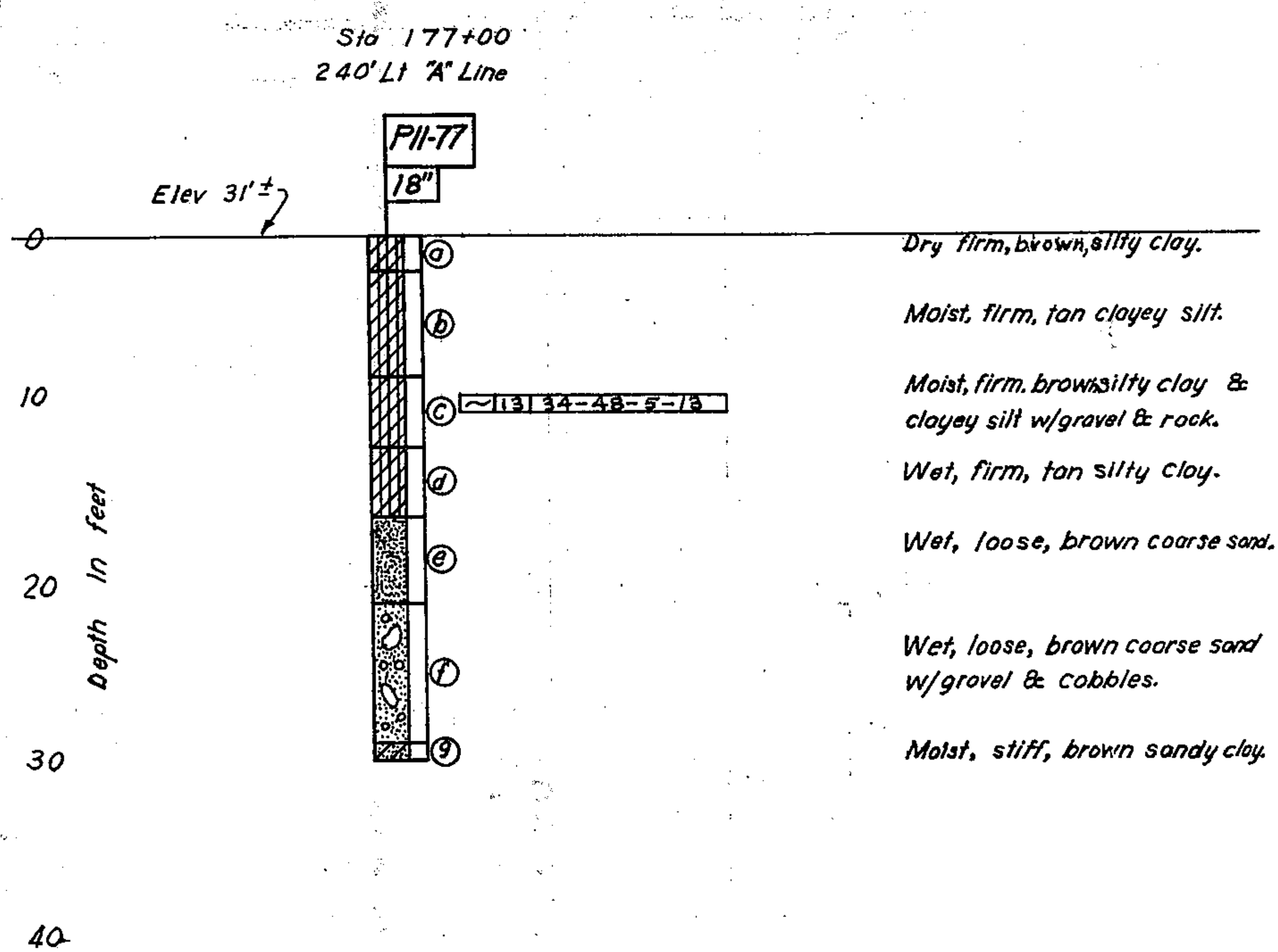
18"
El. 25' ±
0
10
20
30
40
Depth in feet
a
b
c
d
Damp, hard, brown silty clay.
Damp, firm, greenish-tan clayey silt.
Moist, dense, brown clayey sand & rock.
Moist stiff to firm, greenish-tan, silty clay.

4

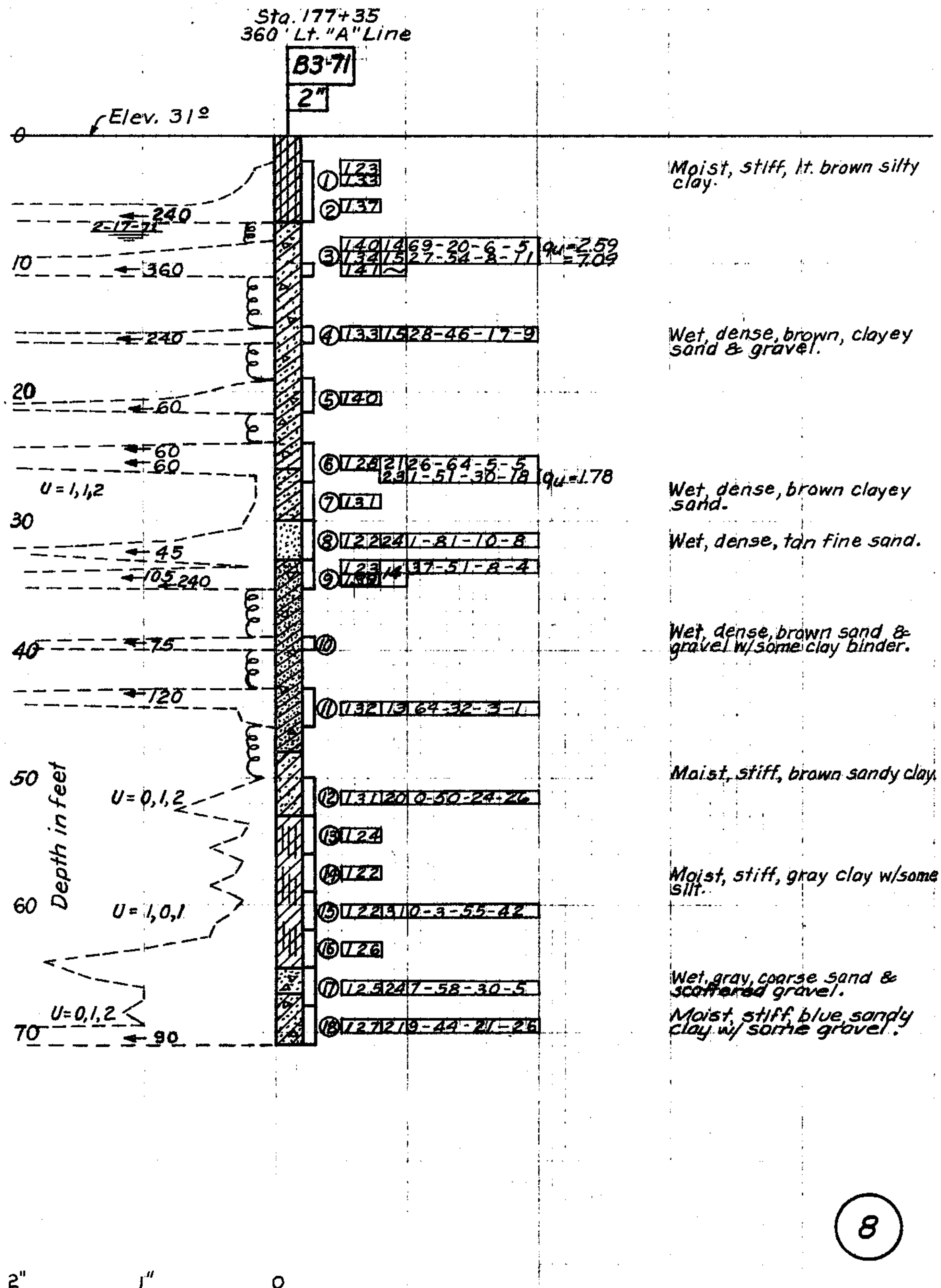

Sta. 172+35
40' Rt. "A" Line
D17-77
2"
El. 27' ±
0
10
20
30
40
50
60
70
Depth in feet
46
48
46
50
2" 1" 0
① 133 8
② 142 133 14 8-62-16-14
③ 121 10
④ 142
⑤ 135 143 14
⑥ 130
⑦ 127
⑧ 124
⑨ 128 142 15
⑩ 124
⑪ 125
⑫ 129
⑬ 129
⑭ 137
⑮ 131
⑯ 131 23 0-48-26-26
⑰ 130 22 0-43-36-21
⑱ 138 130 140 14
⑲ NA 21
S_u = 1.75
= 0.80
= 0.75
= 1.10
Moist, dense, brown clayey sand & rock fragments.
Moist, dense, brown clayey sand.
Wet, dense, brown clayey sand & gravel.
Wet, loose, to dense, coarse gravelly sand w/rocks.
Moist, stiff to hard, brown silty clay.
Wet, dense brown, medium to coarse sand w/gravel.
Moist, stiff to hard, gray silty clay w/some intermittent layers of clayey silt.
Same as above with some rock fragments.
Moist, firm, brown sandy clay.
Wet, dense, brown clayey sand & rocks.
Wet, dense, gray clayey sand & fine gravel.


Sta. 176 +45
50' Rt. "A" Line
PI-70
18"
El. 28' ±
0
10
20
30
40
Depth in feet
5-21-70
a
b
c
d
e
Dry to damp, dark brown silty sand w/rock fragments.
Moist, soft lt. brown, silty clayey sand.
Moist, soft, brown to tan silty clayey sand w/rock fragments.
Moist to wet, brown to tan, clayey silty sand w/gravel & cobbles forming an aquifer.
Damp to moist, firm to stiff, silty clayey sand w/some rock fragments.
1/2 foot lense of pea size gravel (Aquifer)

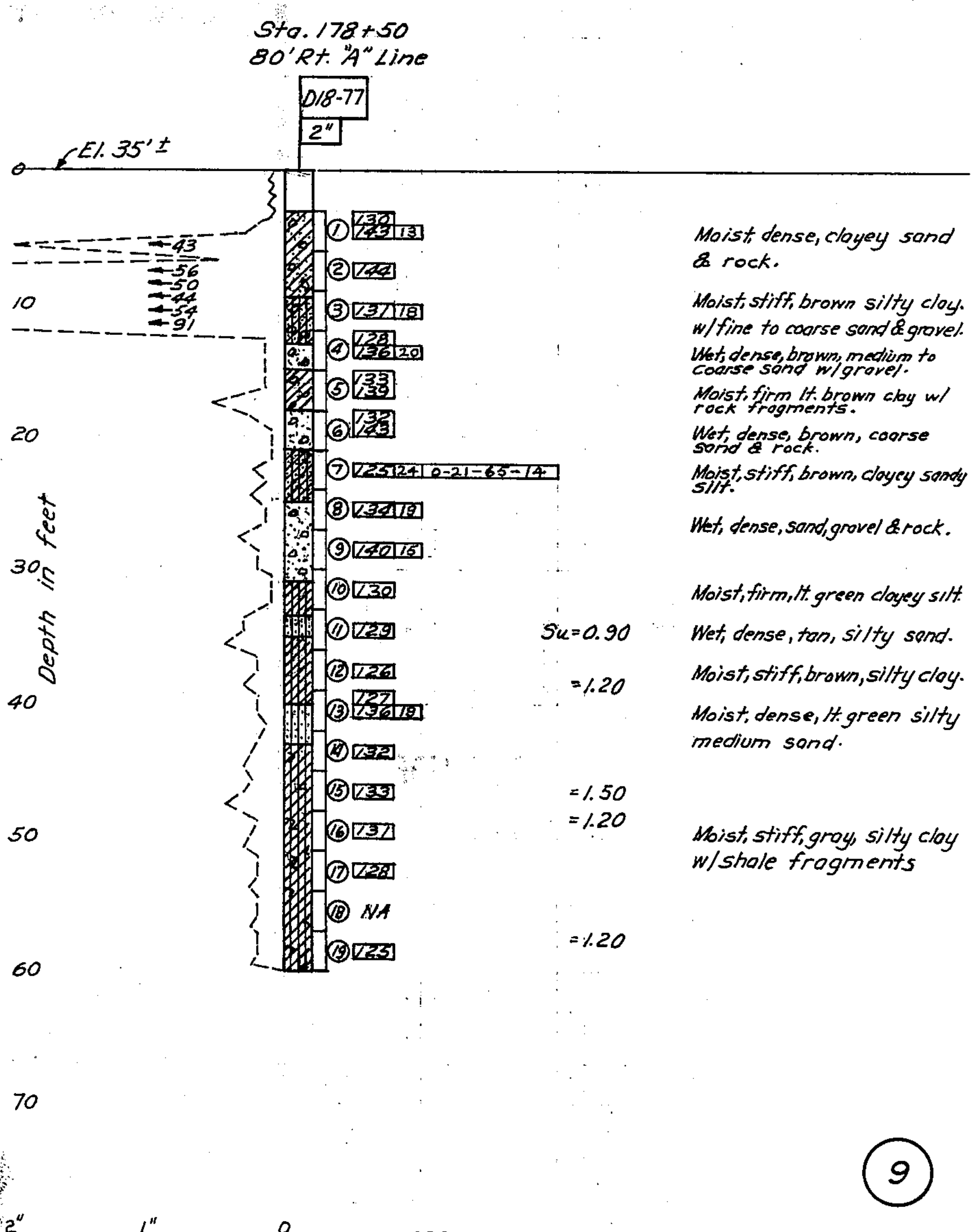
6

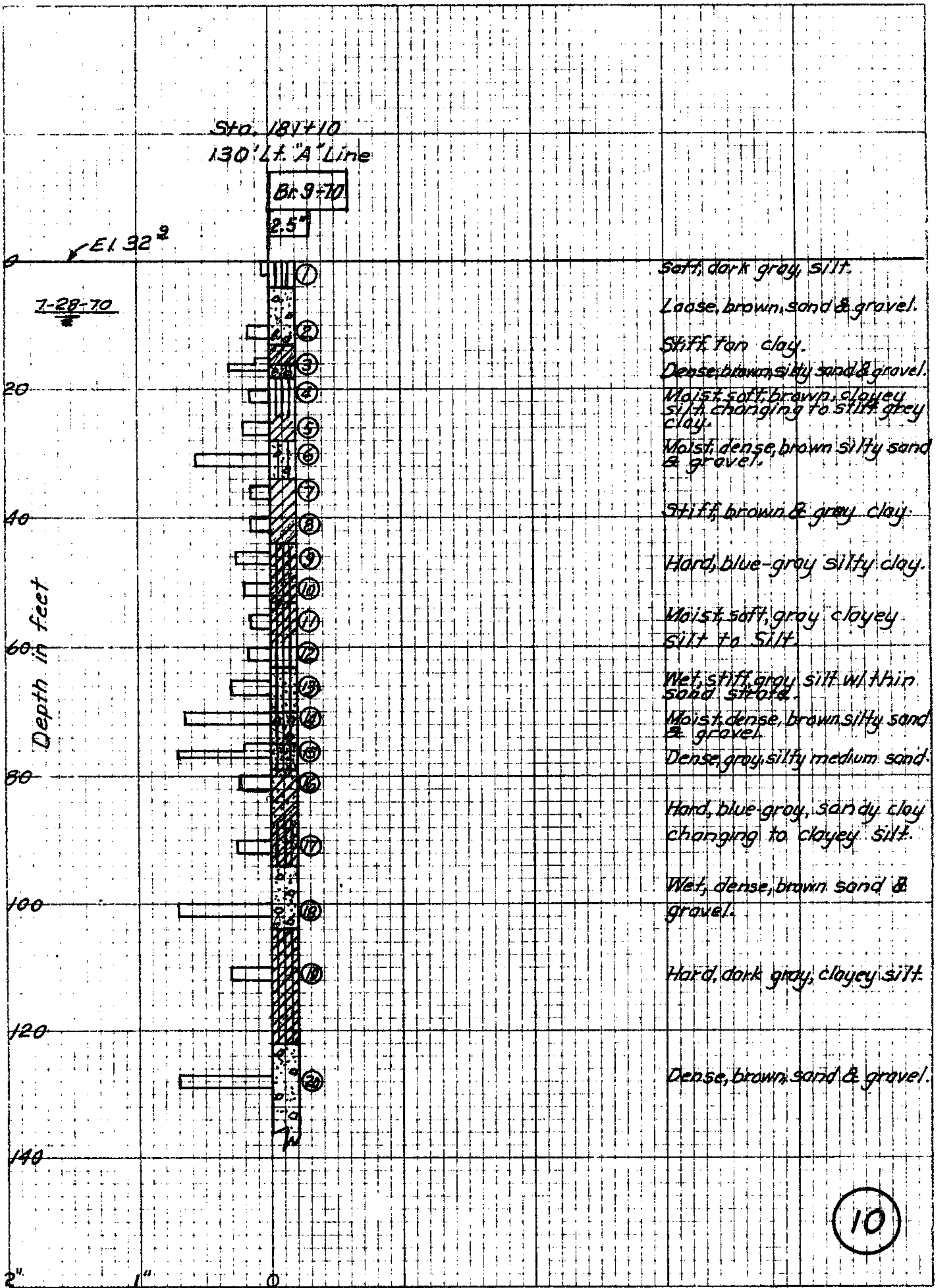

Sta 177+00
240' Lt "A" Line
P11-77
18"
Elev 31' ±
0
10
20
30
40
Depth in feet
a
b
c
d
e
f
g
~ (13) 34-48-5-13
Dry firm, brown, silty clay.
Moist, firm, tan clayey silt.
Moist, firm. brown silty clay &
clayey silt w/gravel & rock.
Wet, firm, tan silty clay.
Wet, loose, brown coarse sand.
Wet, loose, brown coarse sand
w/gravel & cobbles.
Moist, stiff, brown sandy clay.

7

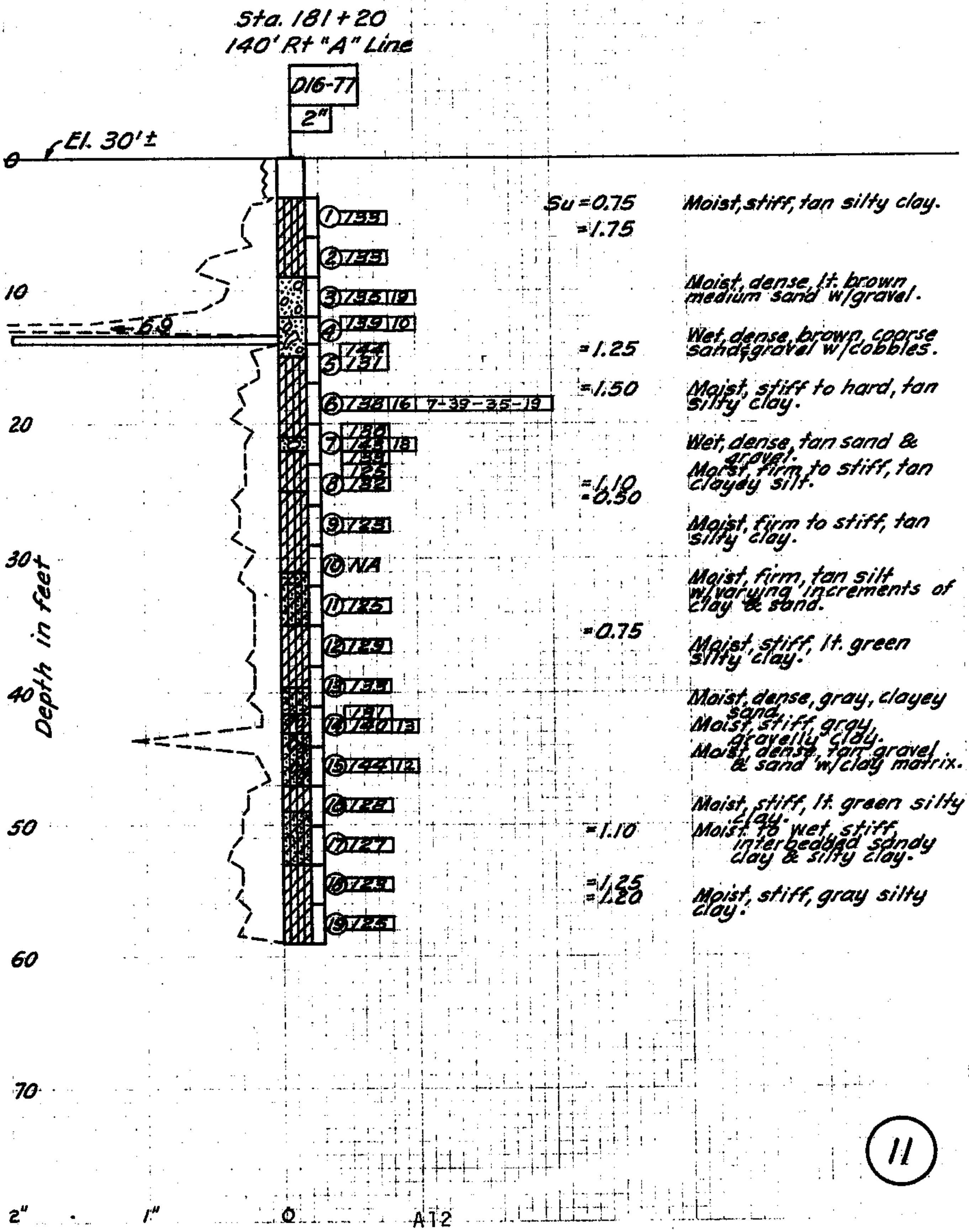


Sta. 177+35
360' Lt. "A" Line
B3-71
2"
Elev. 31º
Depth in feet
0
10
20
30
40
50
60
70
2-17-71
240
360
240
60
60
60
45
105
240
75
120
90
U = 1,1,2
U = 0,1,2
U = 1,0,1
U = 0,1,2
qu = 2.59
= 7.09
qu = 1.78
Moist, stiff, lt. brown silty clay.
Wet, dense, brown, clayey sand & gravel.
Wet, dense, brown clayey sand.
Wet, dense, tan fine sand.
Wet, dense, brown sand & gravel w/some clay binder.
Moist, stiff, brown sandy clay.
Moist, stiff, gray clay w/some silt.
Wet, gray, coarse sand & scattered gravel.
Moist, stiff, blue sandy clay w/ some gravel.
2"
1"
0

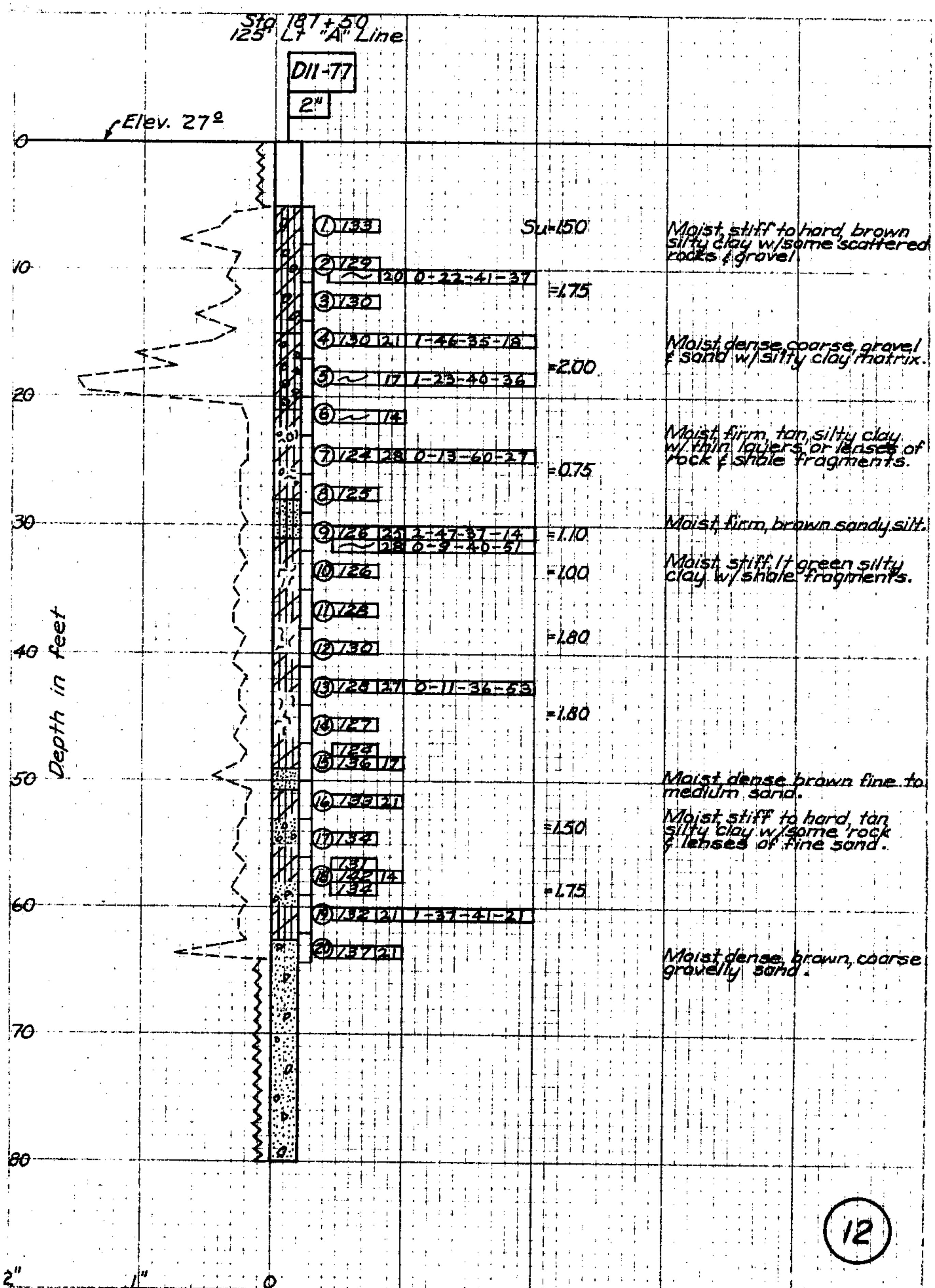

Sta. 178+50
80' Rt. "A" Line
D18-77
2"
El. 35' ±
0
10
20
30
40
50
60
70
Depth in feet
43
56
50
44
54
91
1 130 143 13
2 144
3 137 18
4 128 136 20
5 133 139
6 132 143
7 125 24 0-21-65-14
8 134 19
9 140 16
10 130
11 129
12 126
13 127 136 19
14 132
15 133
16 137
17 128
18 NA
19 125
Su=0.90
=1.20
=1.50
=1.20
=1.20
Moist, dense, clayey sand & rock.
Moist, stiff, brown silty clay. w/fine to coarse sand & gravel.
Wet, dense, brown, medium to coarse sand w/gravel.
Moist, firm lt. brown clay w/ rock fragments.
Wet, dense, brown, coarse sand & rock.
Moist, stiff, brown, clayey sandy silt.
Wet, dense, sand, gravel & rock.
Moist, firm, lt. green clayey silt.
Wet, dense, tan, silty sand.
Moist, stiff, brown, silty clay.
Moist, dense, lt. green silty medium sand.
Moist, stiff, gray, silty clay w/shale fragments
2"
1"
0

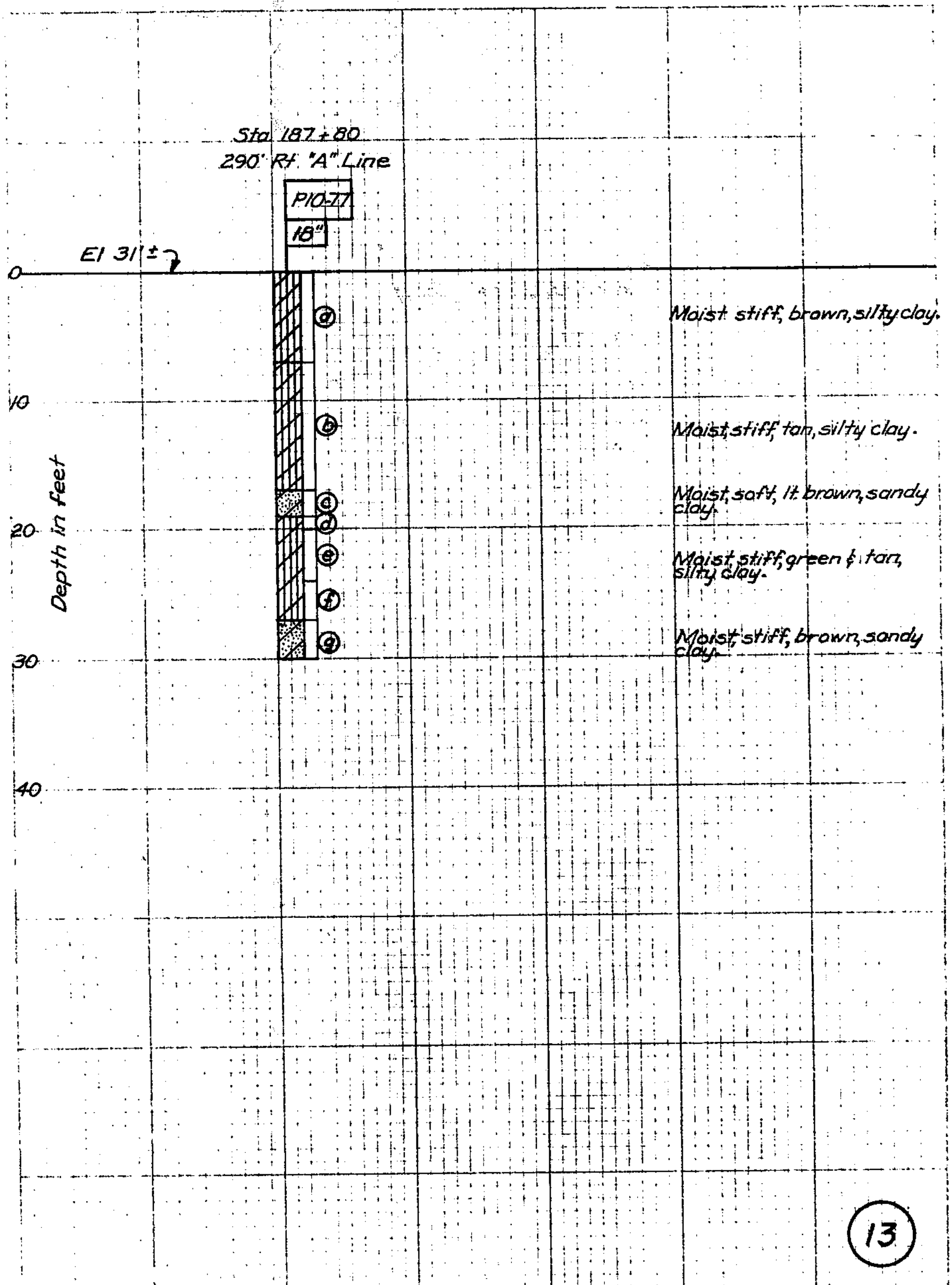

Sta. 181+10
130' Lt. "A" Line
Br. 9-70
2.5"
El. 32.9
7-28-70
Depth in feet
0
20
40
60
80
100
120
140
2"
1"
0
Soft, dark gray silt.
Loose, brown, sand & gravel.
Stiff tan clay.
Dense, brown, silty sand & gravel.
Moist, soft, brown, clayey silt changing to stiff grey clay.
Moist, dense, brown silty sand & gravel.
Stiff, brown & gray clay.
Hard, blue-gray silty clay.
Moist, soft, gray clayey silt to Silt.
Wet, stiff, gray silt w/ thin sand strata.
Moist, dense, brown silty sand & gravel.
Dense, gray, silty medium sand.
Hard, blue-gray, sandy clay changing to clayey silt.
Wet, dense, brown sand & gravel.
Hard, dark gray, clayey silt.
Dense, brown, sand & gravel.
10

[illegible]

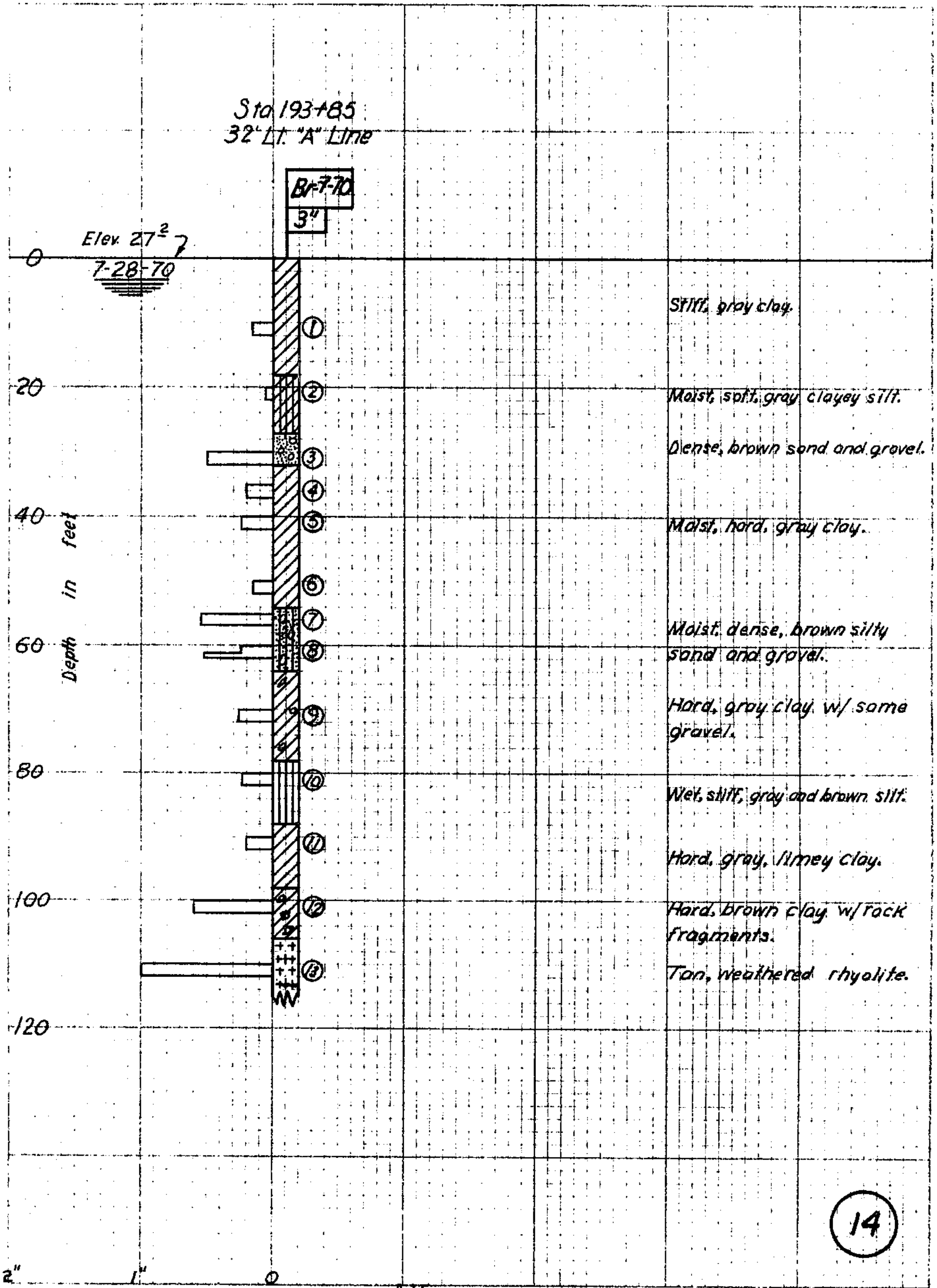

Sta. 181+20
140' Rt "A" Line
D16-77
2"
El. 30'±
0
10
20
30
40
50
60
70
Depth in feet
6.9
Su=0.75
=1.75
=1.25
=1.50
=1.10
=0.50
=0.75
=1.10
=1.25
=1.20
7-39-35-19
NA
Moist, stiff, tan silty clay.
Moist, dense, lt. brown medium sand w/gravel.
Wet, dense, brown, coarse sand&gravel w/cobbles.
Moist, stiff to hard, tan silty clay.
Wet, dense, tan sand & gravel.
Moist, firm to stiff, tan clayey silt.
Moist, firm to stiff, tan silty clay.
Moist, firm, tan silt w/varying increments of clay & sand.
Moist, stiff, lt. green silty clay.
Moist, dense, gray, clayey sand.
Moist, stiff, gray, gravelly clay.
Moist, dense, tan gravel & sand w/clay matrix.
Moist, stiff, lt. green silty clay.
Moist to wet, stiff, interbedded sandy clay & silty clay.
Moist, stiff, gray silty clay.
2"
1"
0
11

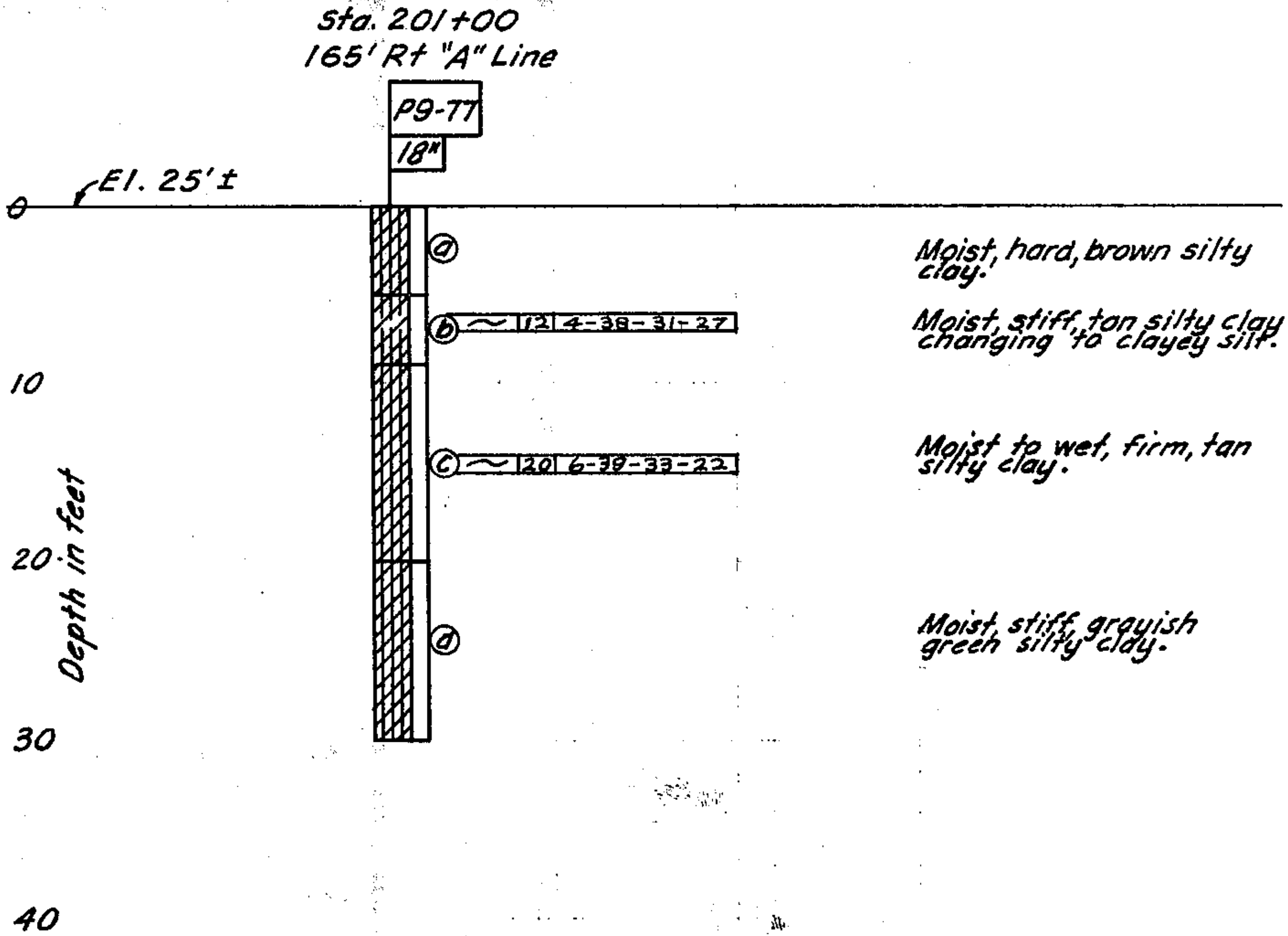
Permanent
PLOVER BOND
25% COTTON FIBER
U.S.A.

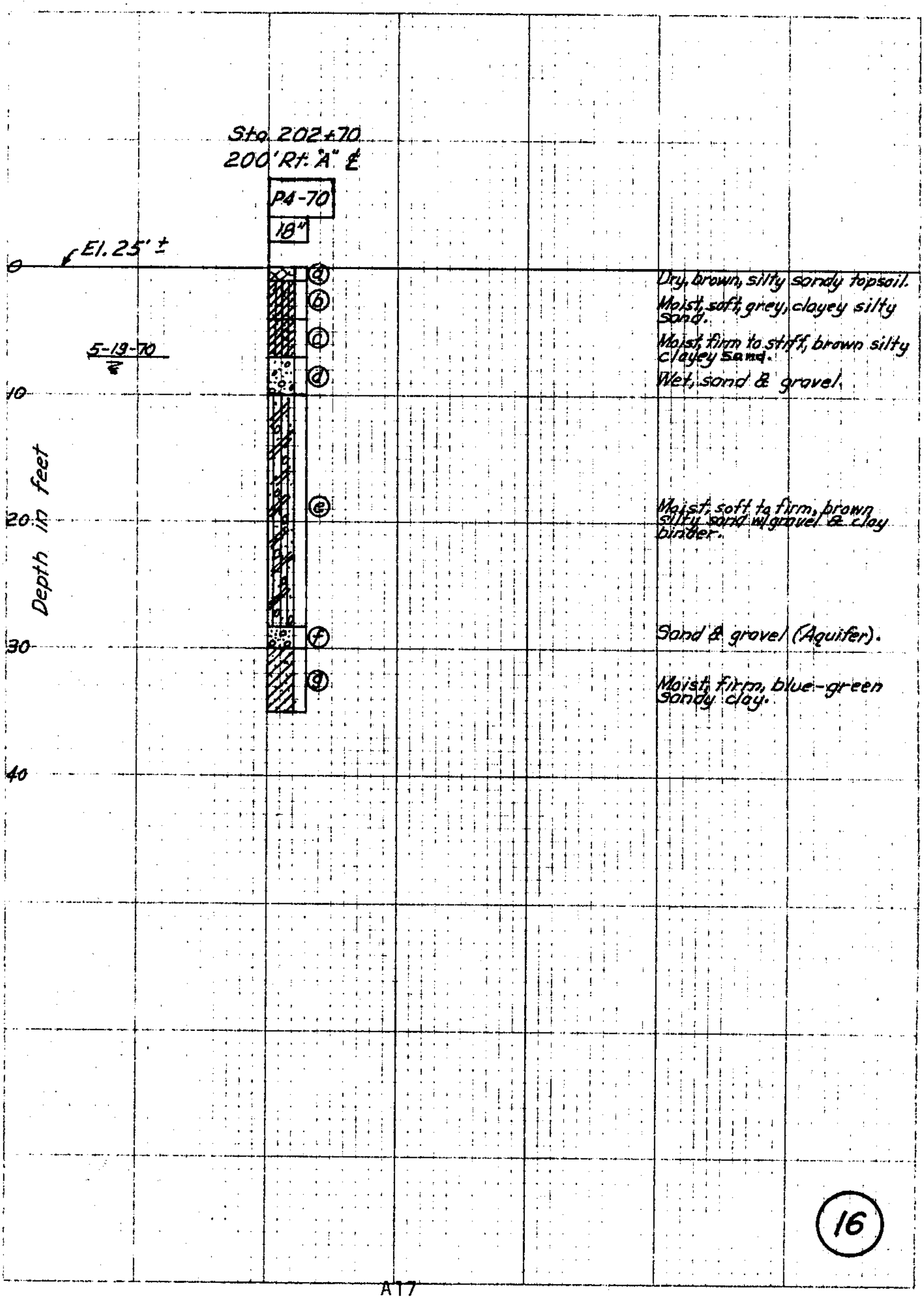

Sta 187+50
125' Lt "A" Line
D11-77
2"
Elev. 27°
Depth in feet
0
10
20
30
40
50
60
70
80
2"
1"
0
1 133
Su=1.50
2 129
20 0-22-41-37
=1.75
3 130
4 130 21 1-46-35-18
=2.00
5 17 1-23-40-36
6 14
7 124 28 0-13-60-27
=0.75
8 125
9 126 25 2-47-37-14
28 0-9-40-51
=1.10
10 126
=1.00
11 128
=1.80
12 130
13 128 27 0-11-36-53
=1.80
14 127
128
15 136 17
16 139 21
=1.50
17 134
131
18 122 14
134
=1.75
19 132 21 1-37-41-21
20 137 21
Moist, stiff to hard, brown silty clay w/some scattered rocks & gravel.
Moist, dense, coarse gravel & sand w/silty clay matrix.
Moist, firm, tan, silty clay w/thin layers or lenses of rock & shale fragments.
Moist, firm, brown sandy silt.
Moist, stiff, lt green silty clay w/shale fragments.
Moist, dense, brown fine to medium sand.
Moist, stiff to hard, tan silty clay w/some rock & lenses of fine sand.
Moist, dense, brown, coarse gravelly sand.

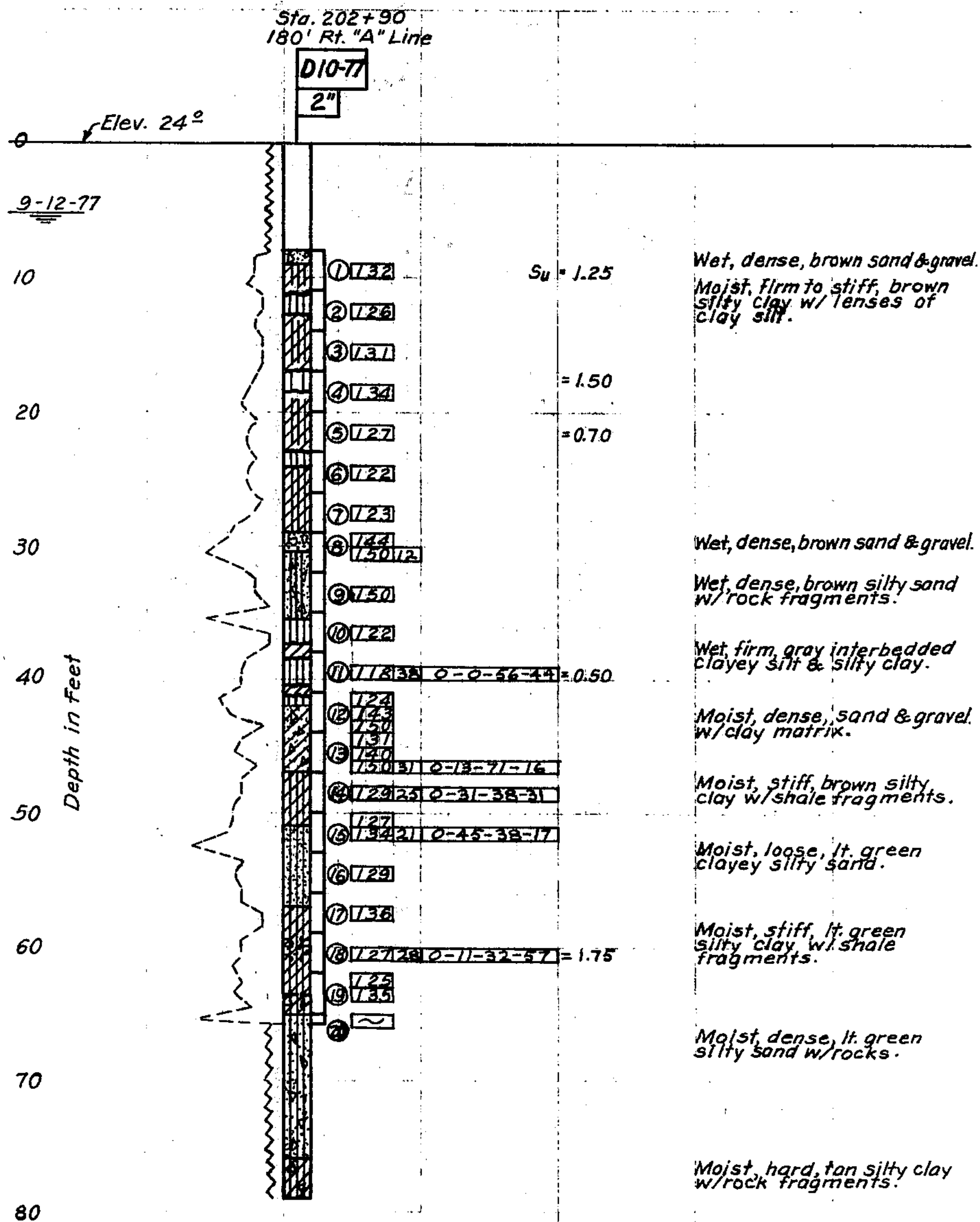


Sta 187+80
290' Rt. "A" Line
P10-77
18"
El 31 ±
0
10
20
30
40
Depth in feet
a
b
c
d
e
f
g
Moist stiff, brown, silty clay.
Moist, stiff, tan, silty clay.
Moist, soft, lt. brown, sandy clay.
Moist, stiff, green & tan, silty clay.
Moist, stiff, brown, sandy clay.
13


Sta 193+85
32' Lt. "A" Line
Br-7-70
3"
Elev. 27²
7-28-70
0
20
40
60
80
100
120
Depth in feet
Stiff, gray clay.
Moist, soft, gray clayey silt.
Dense, brown sand and gravel.
Moist, hard, gray clay.
Moist, dense, brown silty sand and gravel.
Hard, gray clay w/ some gravel.
Wet, stiff, gray and brown silt.
Hard, gray, limey clay.
Hard, brown clay w/ rock fragments.
Tan, weathered rhyolite.
2"
1"
0
14

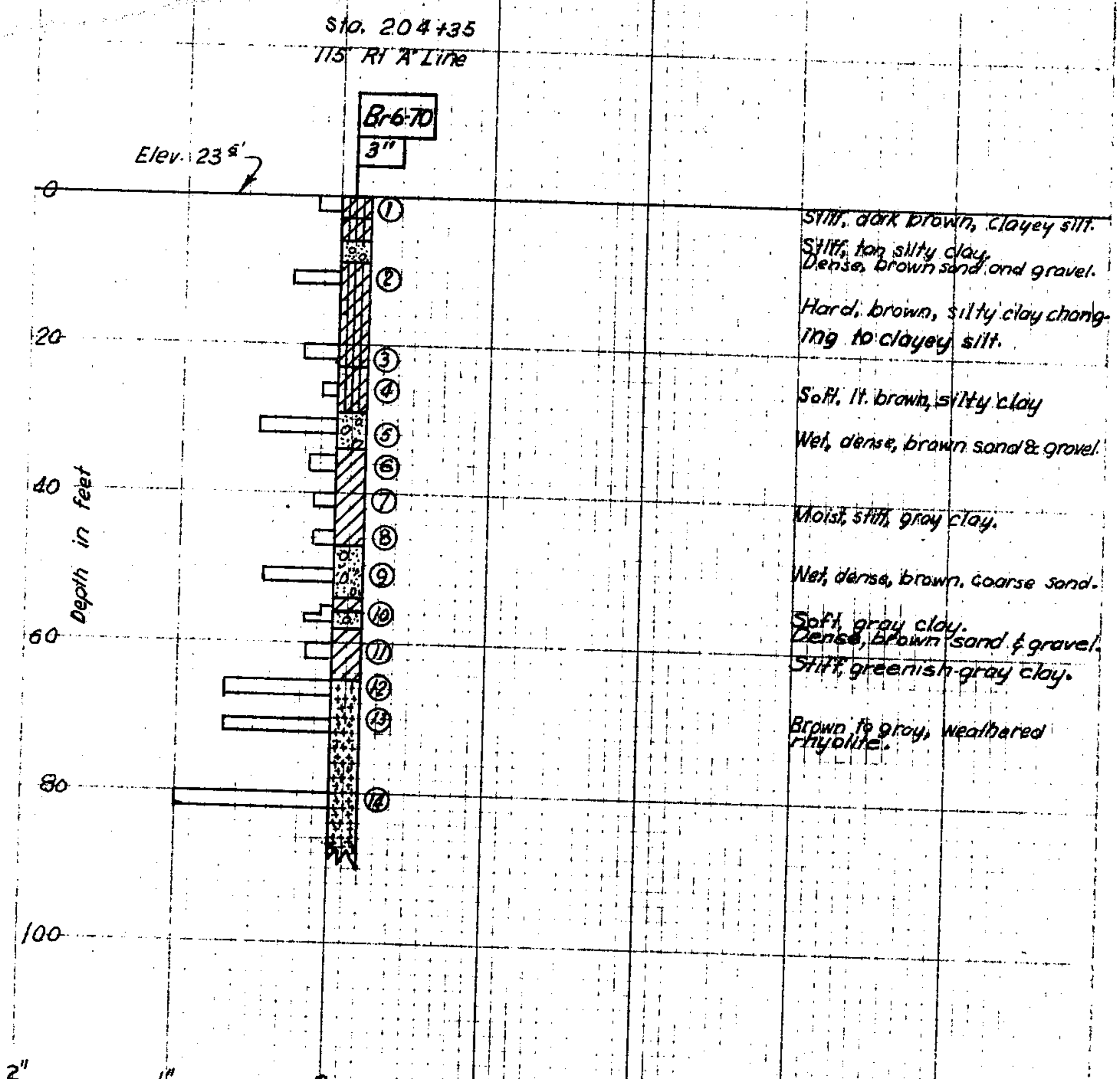

Sta. 201+00
165' Rt "A" Line
P9-77
18"
El. 25'±
0
10
20
30
40
Depth in feet
a
b
12 4-38-31-27
c
20 6-39-33-22
d
Moist, hard, brown silty clay.
Moist, stiff, tan silty clay changing to clayey silt.
Moist to wet, firm, tan silty clay.
Moist, stiff, grayish green silty clay.


Sta. 202+70
200' Rt. "A" ℄
P4-70
18"
El. 25' ±
5-19-70
0
10
20
30
40
Depth in feet
ⓐ
ⓑ
ⓒ
ⓓ
ⓔ
ⓕ
ⓖ
Dry, brown, silty sandy topsoil.
Moist, soft, grey, clayey silty sand.
Moist, firm to stiff, brown silty clayey sand.
Wet, sand & gravel.
Moist, soft to firm, brown silty sand w/gravel & clay binder.
Sand & gravel (Aquifer).
Moist, firm, blue-green sandy clay.
16



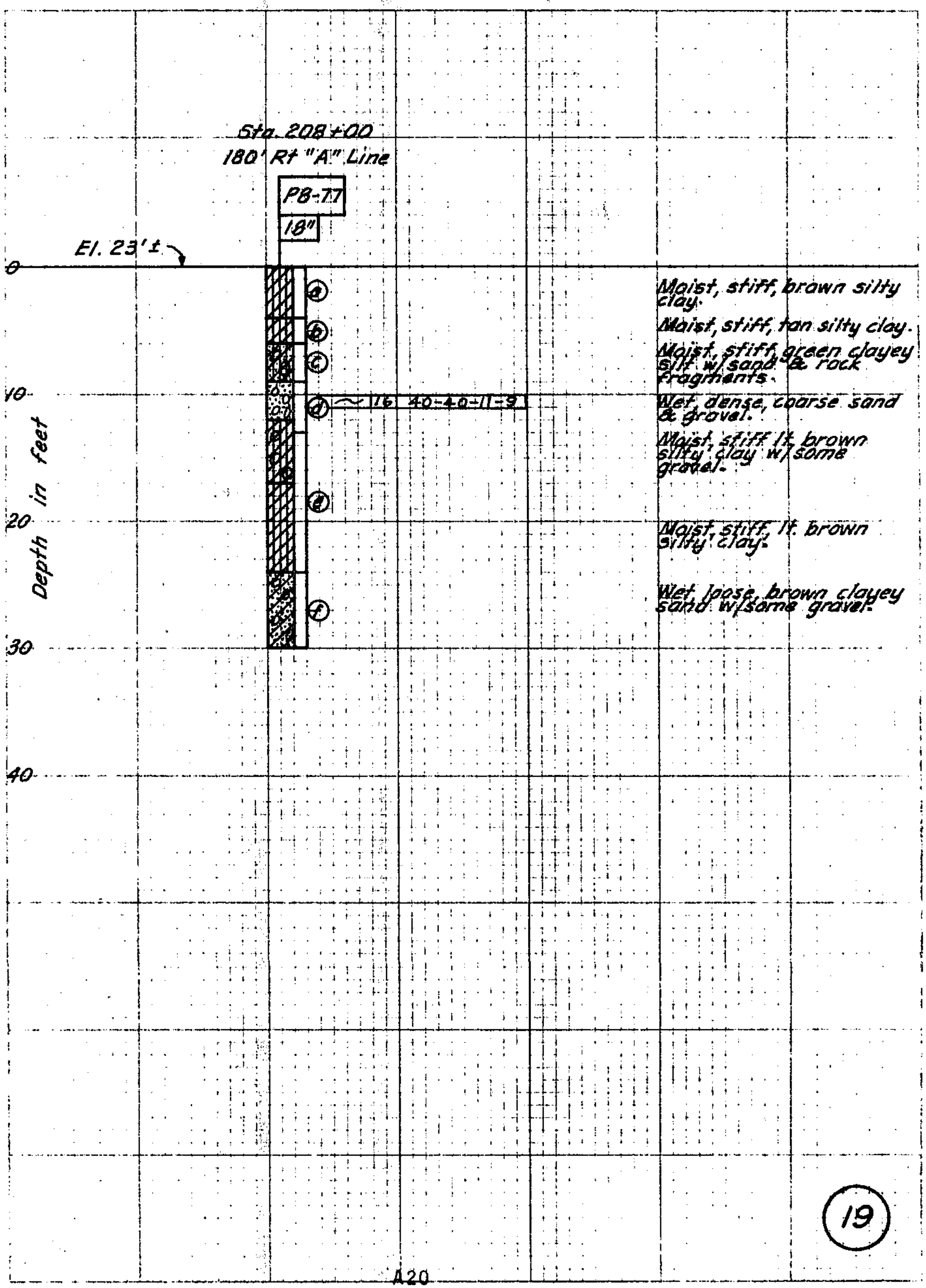
Sta. 202+90
180' Rt. "A" Line
D10-77
2"
Elev. 24º
0
9-12-77
10
20
30
40
50
60
70
80
Depth in Feet
Su = 1.25
= 1.50
= 0.70
= 0.50
= 1.75
1 132
2 126
3 131
4 134
5 127
6 122
7 123
8 144 150 12
9 150
10 122
11 118 38 0-0-56-44
12 124 143 150
13 137 140 150 31 0-13-71-16
14 129 25 0-31-38-31
15 127 134 21 0-45-38-17
16 129
17 136
18 127 28 0-11-32-57
19 125 135
20 ~
Wet, dense, brown sand & gravel.
Moist, firm to stiff, brown silty clay w/ lenses of clay silt.
Wet, dense, brown sand & gravel.
Wet, dense, brown silty sand w/rock fragments.
Wet, firm, gray interbedded clayey silt & silty clay.
Moist, dense, sand & gravel w/clay matrix.
Moist, stiff, brown silty clay w/shale fragments.
Moist, loose, lt. green clayey silty sand.
Moist, stiff, lt. green silty clay w/ shale fragments.
Moist, dense, lt. green silty sand w/rocks.
Moist, hard, tan silty clay w/rock fragments.
2"
1"
0

17

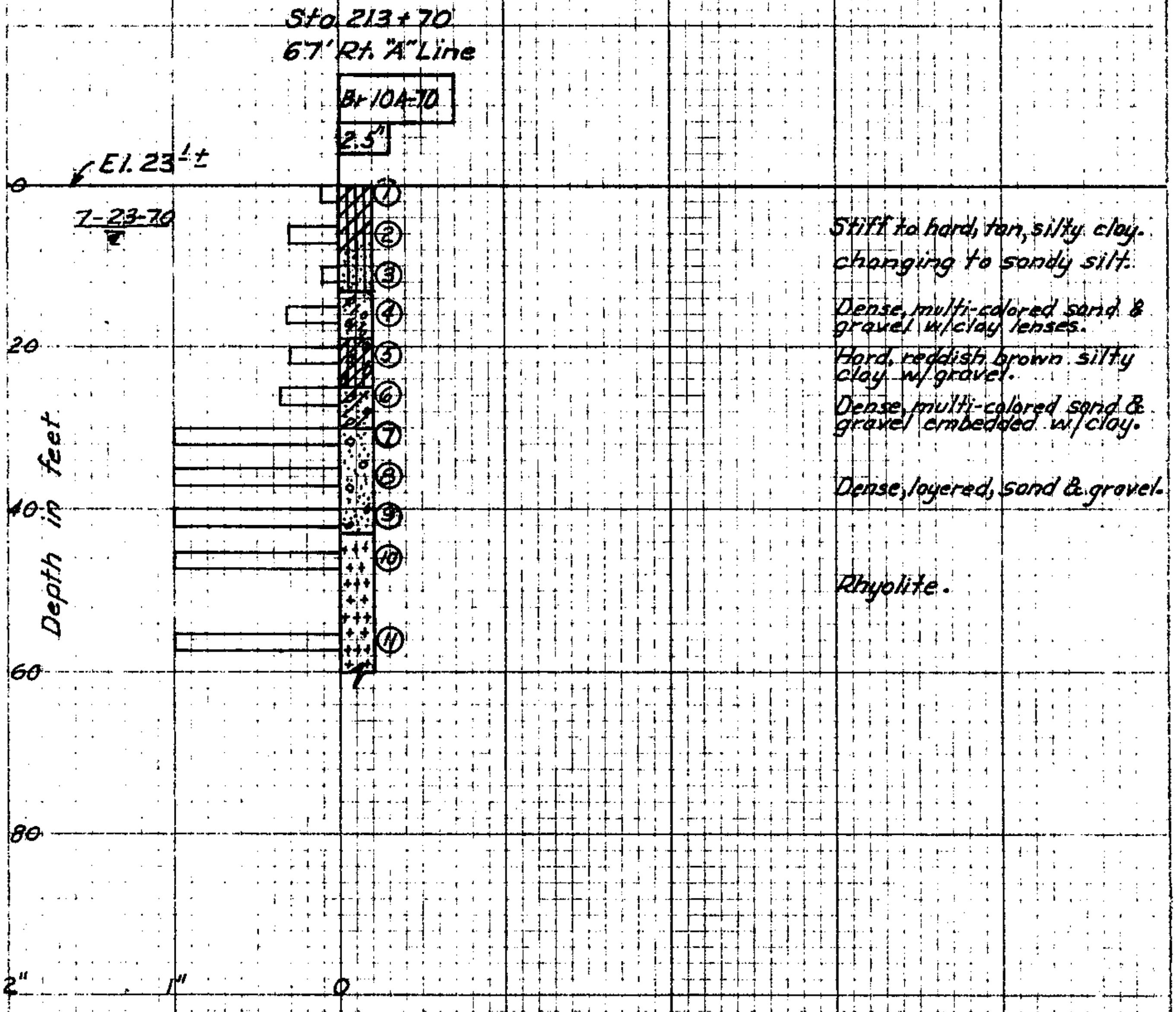


Sta. 204+35
115' Rt "A" Line
Br-6-70
3"
Elev. 23.5'
0
20
40
60
80
100
Depth in feet
2"
1"
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
Stiff, dark brown, clayey silt.
Stiff, tan silty clay.
Dense, brown sand and gravel.
Hard, brown, silty clay changing to clayey silt.
Soft, lt. brown, silty clay
Wet, dense, brown sand & gravel
Moist, stiff, gray clay.
Wet, dense, brown, coarse sand.
Soft, gray clay.
Dense, brown sand & gravel.
Stiff, greenish-gray clay.
Brown to gray, weathered rhyolite.

18

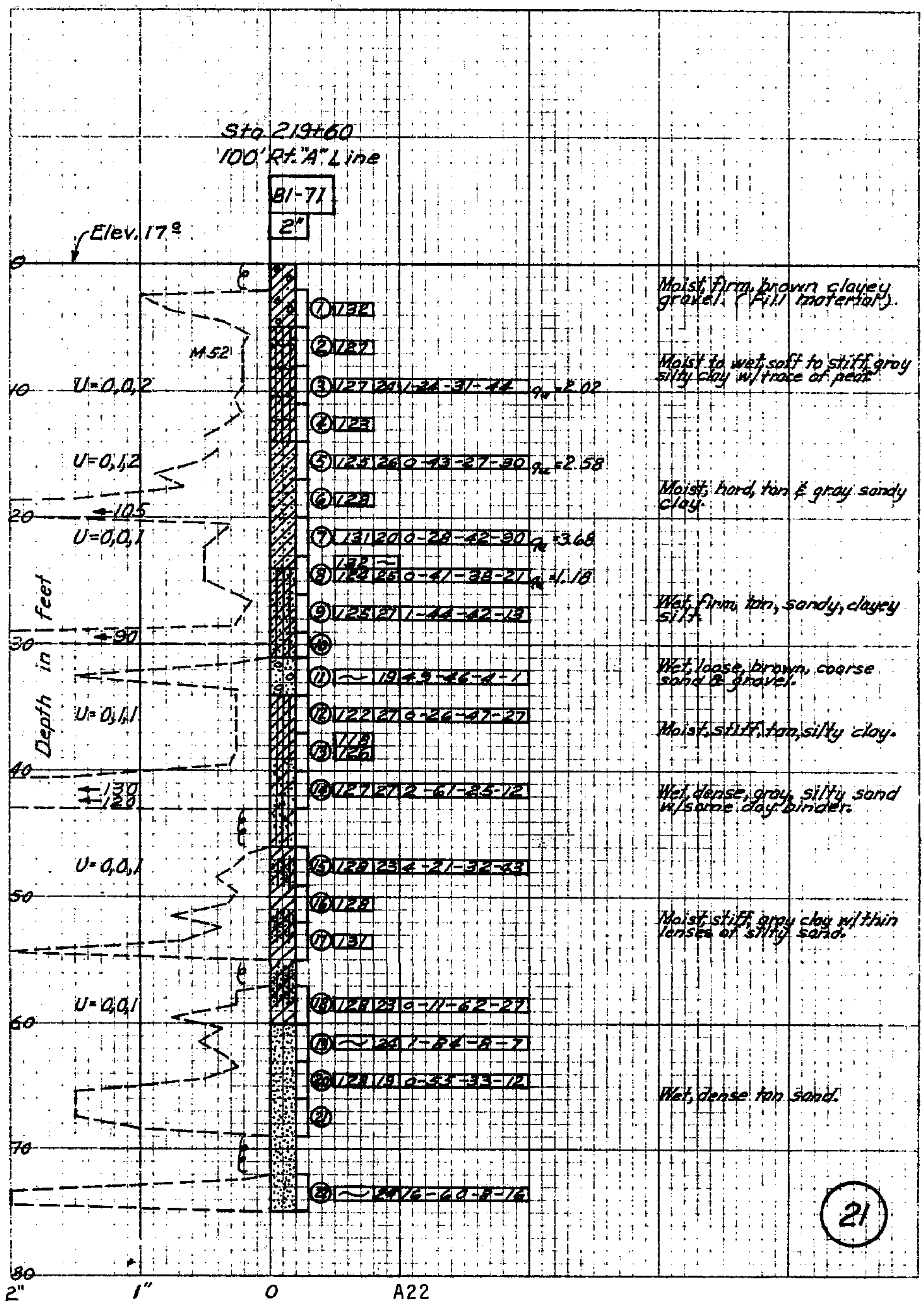


Sta. 208+00
180' Rt "A" Line
PB-77
18"
El. 23'±
0
10
20
30
40
Depth in feet
a
b
c
d
e
f
16 40-40-11-9
Moist, stiff, brown silty clay.
Moist, stiff, tan silty clay.
Moist, stiff, green clayey silt w/sand & rock fragments.
Wet, dense, coarse sand & gravel.
Moist, stiff lt. brown silty clay w/some gravel.
Moist, stiff, lt. brown silty clay.
Wet, loose, brown clayey sand w/some gravel.
19



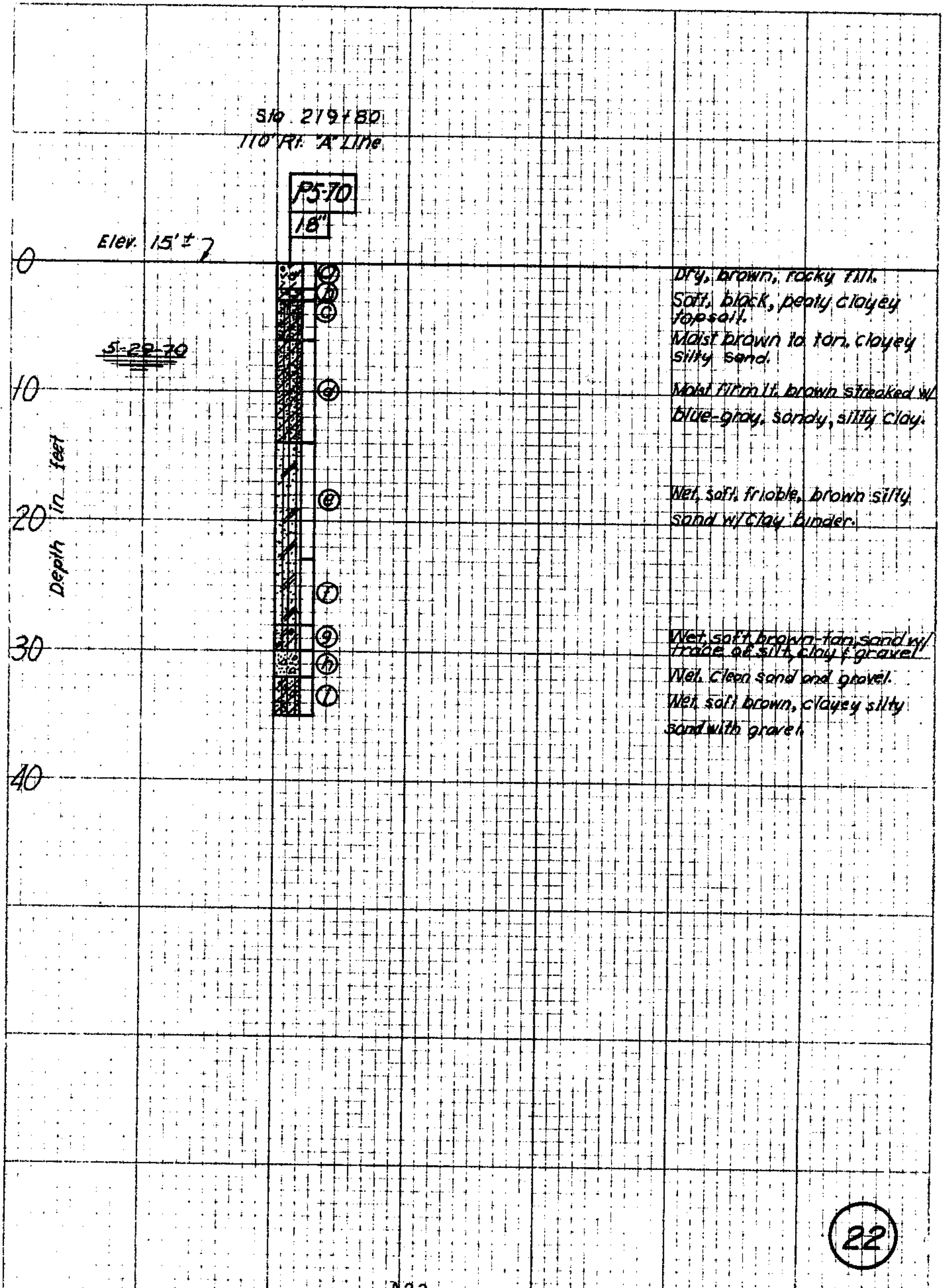
Sta. 213+70
67' Rt. "A" Line
Br 10A-70
2.5"
El. 23±
7-23-70
0
20
40
60
80
Depth in feet
1
2
3
4
5
6
7
8
9
10
11
Stiff to hard, tan, silty clay.
changing to sandy silt.
Dense, multi-colored sand & gravel w/clay lenses.
Hard, reddish brown silty clay w/gravel.
Dense, multi-colored sand & gravel embedded w/clay.
Dense, layered, sand & gravel.
Rhyolite.
2"
1"
0

20

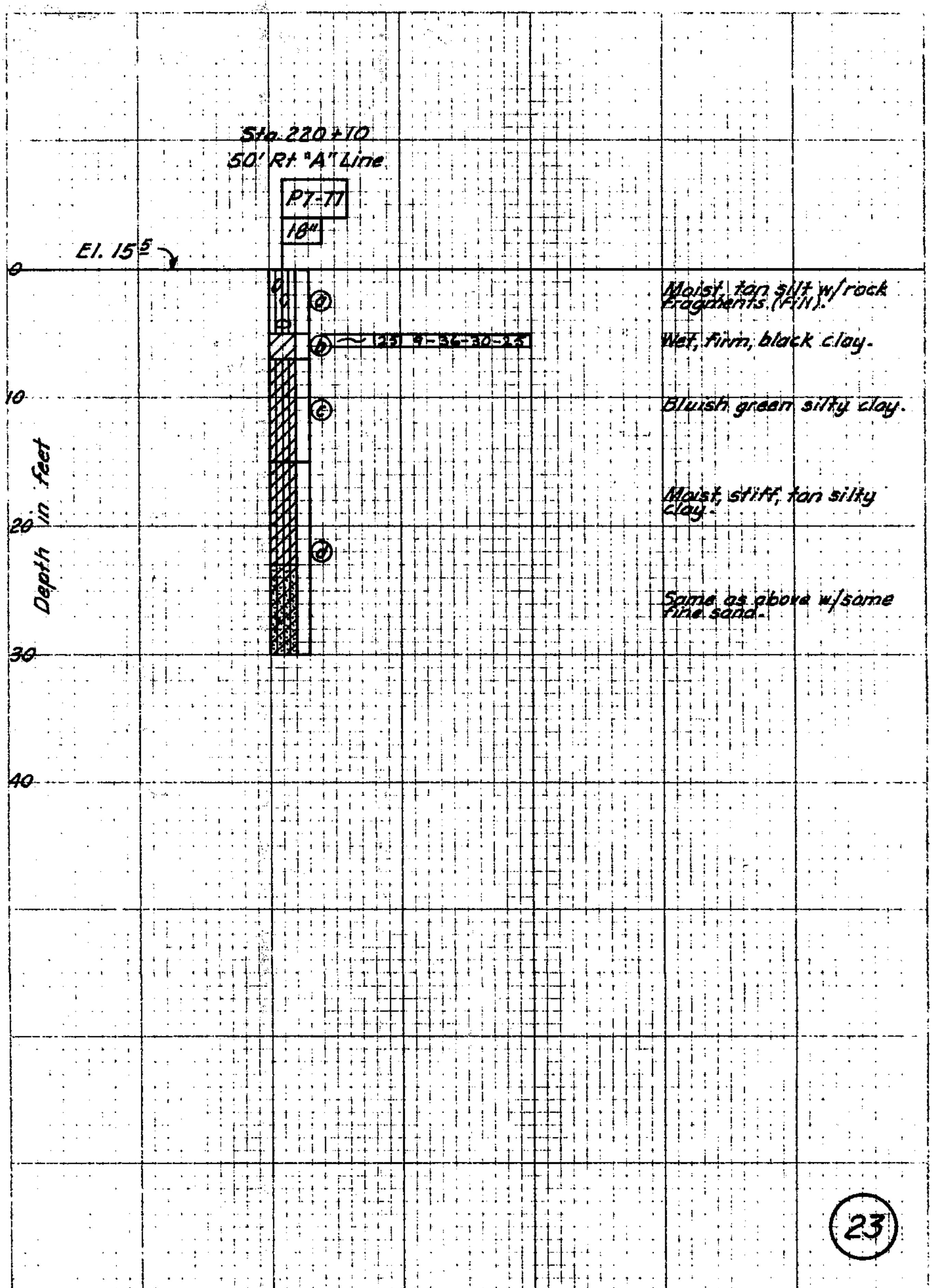


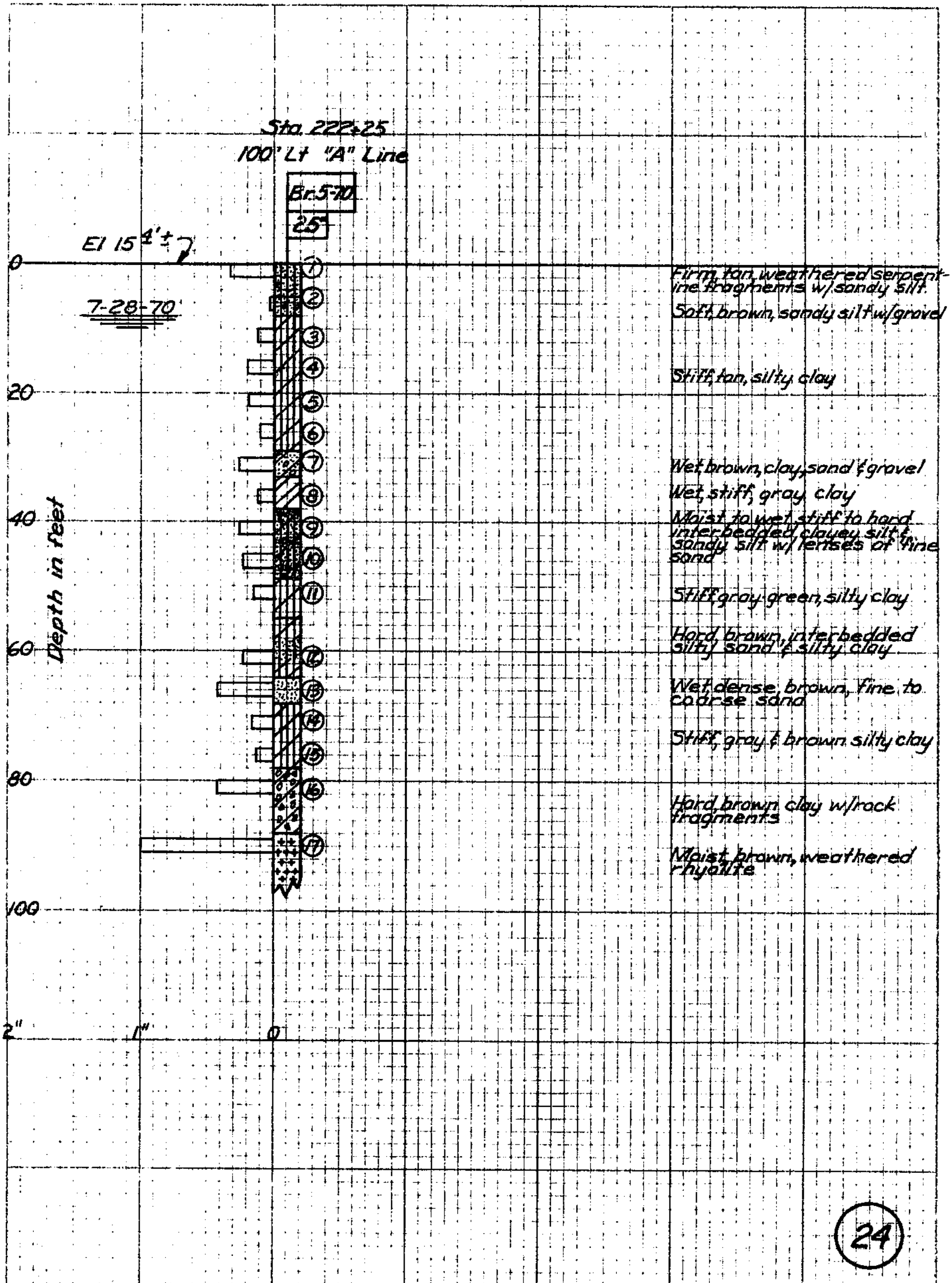
Sta 219+60
100' Rt. "A" Line
B1-71
2"
Elev. 17.0
Depth in feet
0
10
20
30
40
50
60
70
80
M 5.2
U=0,0,2
U=0,1,2
←105
U=0,0,1
←90
U=0,1,1
←130
←120
U=0,0,1
U=0,0,1
① 132
② 127
③ 127 24 1-24-31-44 qu=2.02
④ 123
⑤ 125 26 0-43-27-30 qu=2.58
⑥ 123
⑦ 131 20 0-28-42-30 qu=3.68
132 ~
⑧ 122 25 0-41-38-21 qu=1.18
⑨ 125 27 1-44-42-13
⑩
⑪ ~ 19 4.3-46-4-1
⑫ 122 27 0-26-47-27
118
⑬ 126
⑭ 127 27 2-61-25-12
⑮ 128 23 4-21-32-43
⑯ 128
⑰ 137
⑱ 128 23 0-11-62-27
⑲ ~ 24 1-84-8-7
⑳ 128 19 0-33-33-12
㉑
㉒ ~ 24 16-60-8-16
Moist, firm, brown clayey gravel. (Fill material).
Moist to wet soft to stiff gray silty clay w/trace of peat.
Moist, hard, tan & gray sandy clay.
Wet, firm, tan, sandy, clayey silt.
Wet, loose, brown, coarse sand & gravel.
Moist, stiff, tan, silty clay.
Wet, dense, gray, silty sand w/some clay binder.
Moist, stiff, gray clay w/thin lenses of silty sand.
Wet, dense tan sand.
21
2"
1"
0

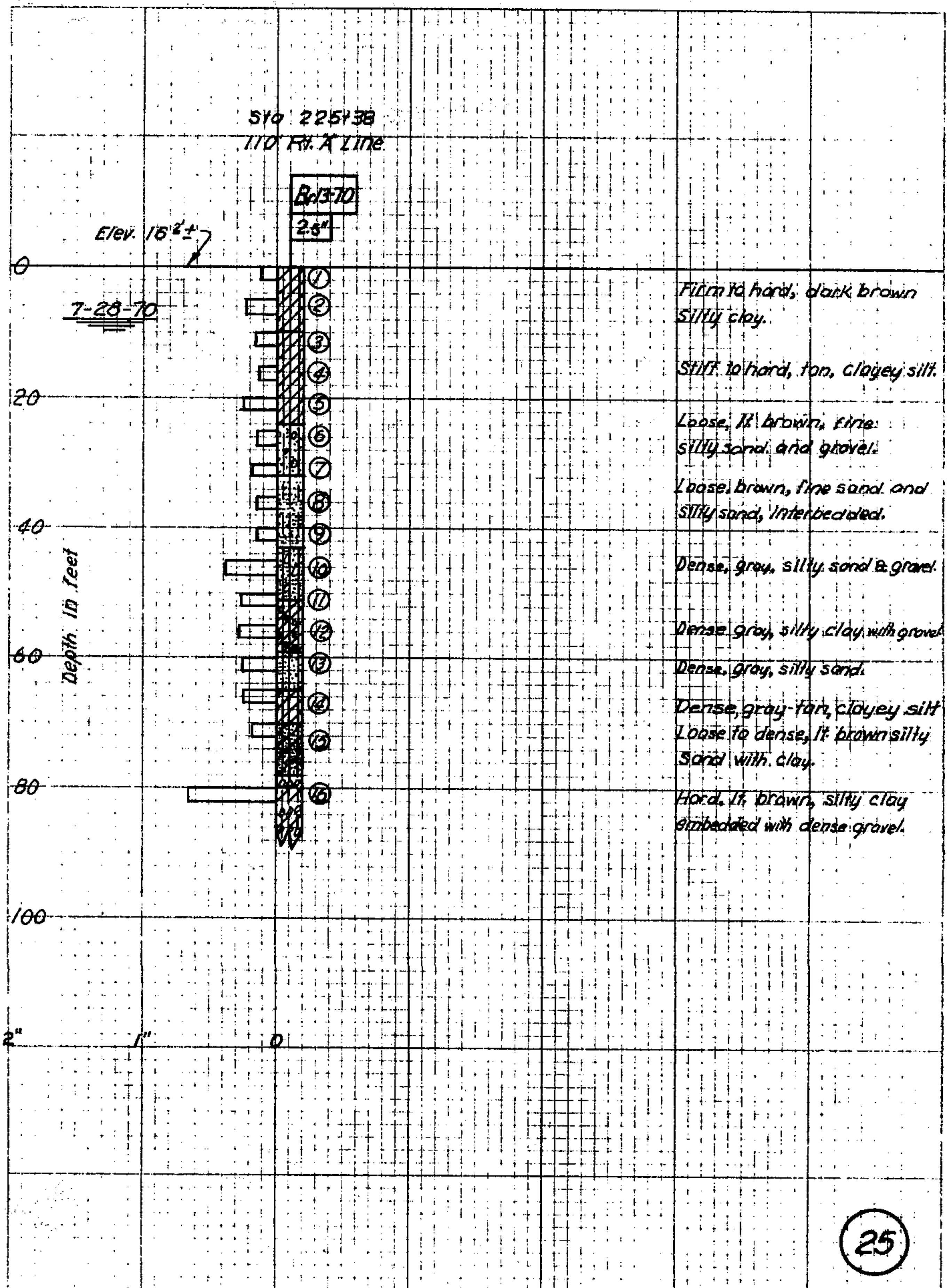
A22



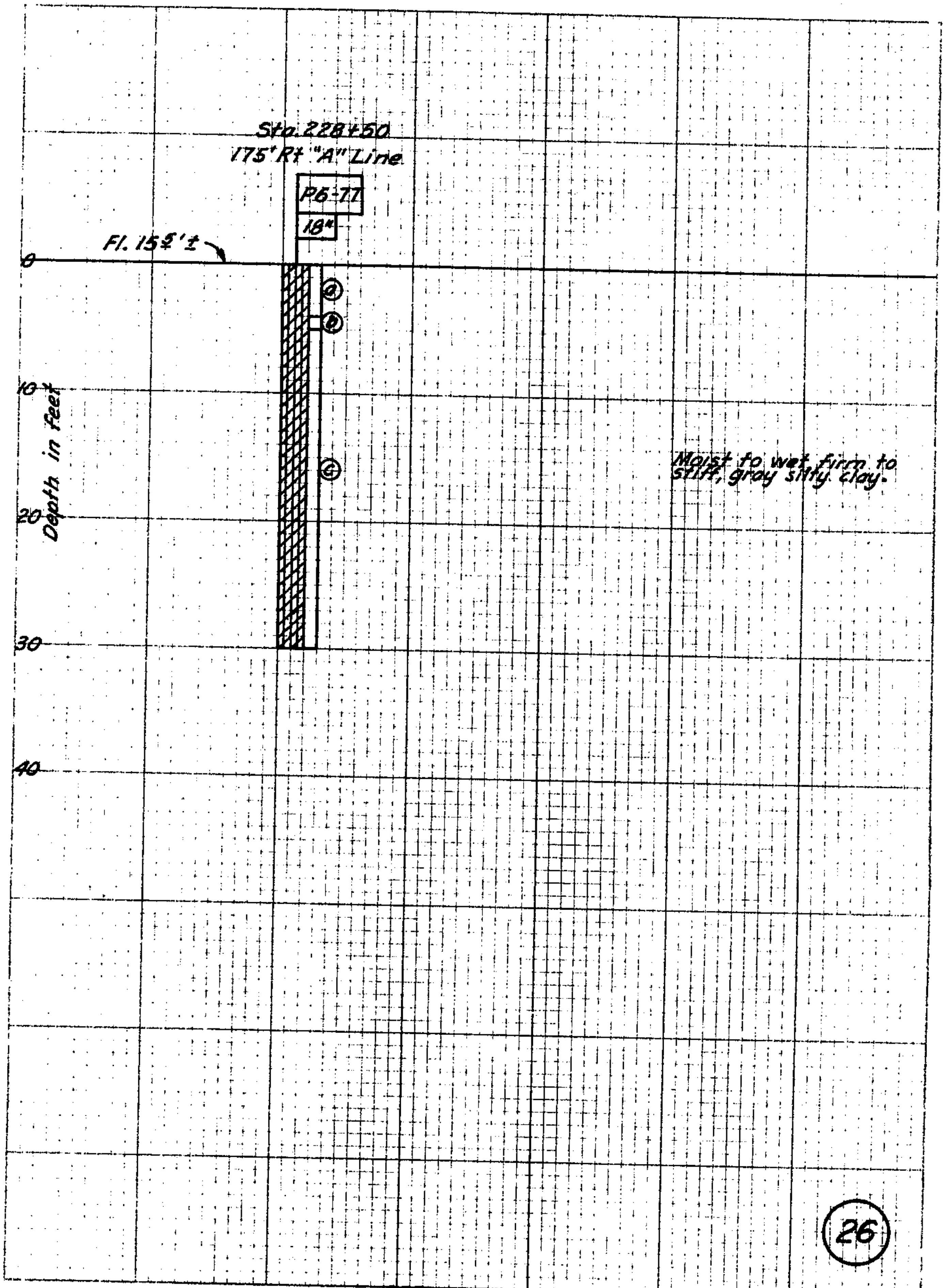
Sta. 219+80
110' Rt. "A" Line
P5-70
18"
Elev. 15'±
5-29-70
0
10
20
30
40
Depth in feet
a
b
c
d
e
f
g
h
i
Dry, brown, rocky fill.
Soft, black, peaty clayey topsoil.
Moist brown to tan, clayey silty sand.
Moist firm lt. brown streaked w/ blue-gray, sandy, silty clay.
Wet, soft, friable, brown silty sand w/clay binder.
Wet, soft brown-tan sand w/ trace of silt, clay & gravel
Wet. Clean sand and gravel.
Wet, soft brown, clayey silty sand with gravel.
22


Sta. 220+10
50' Rt "A" Line
PT-T1
18"
El. 15.5
0
10
20
30
40
Depth in feet
a
b
c
d
(25) 9-36-30-23
Moist, tan silt w/rock fragments. (Fill).
Wet, firm, black clay.
Bluish green silty clay.
Moist, stiff, tan silty clay.
Same as above w/some fine sand.
23

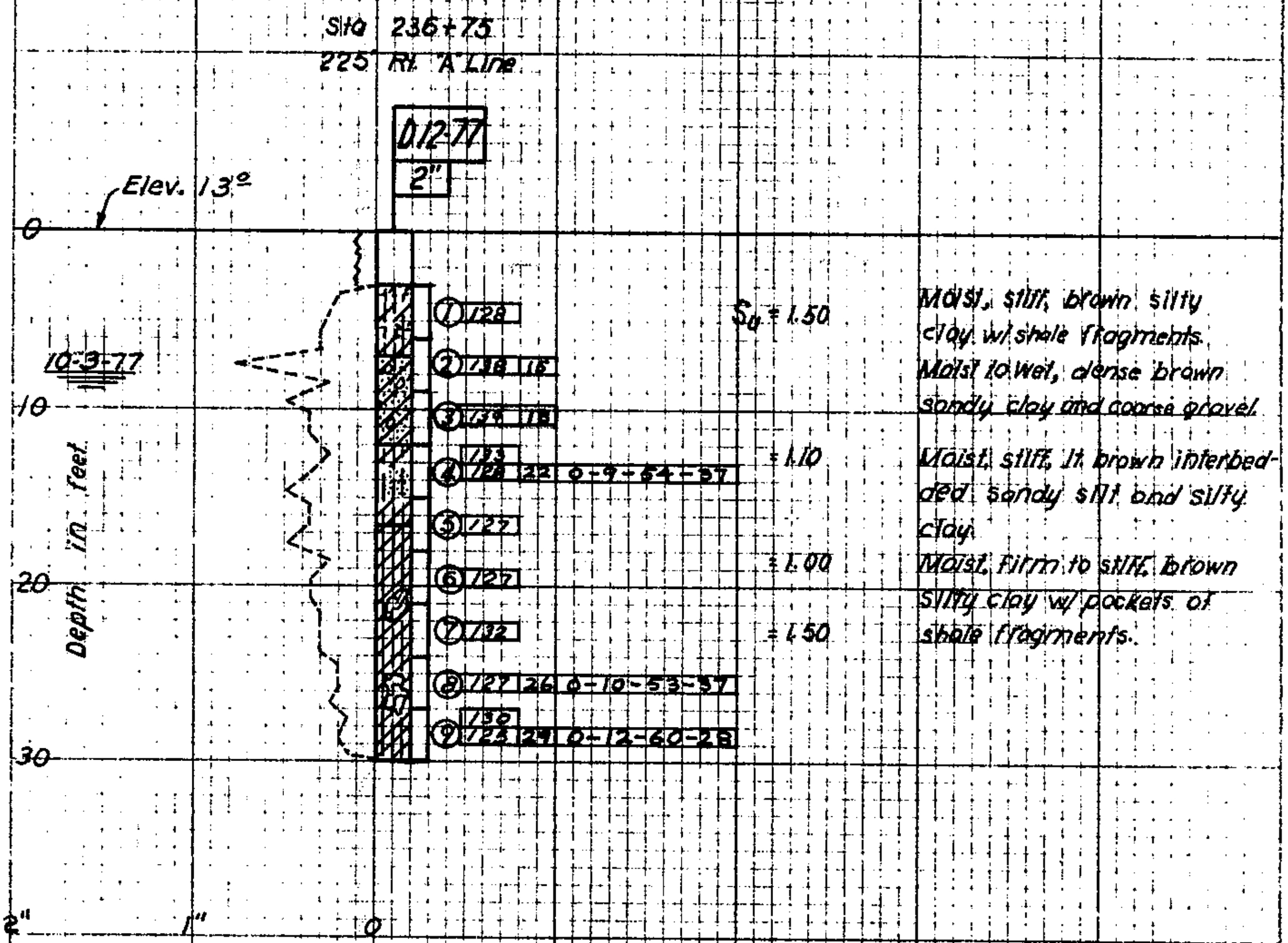

Sta. 222+25
100' Lt "A" Line
Br.5-70
25
El 15 4'±
7-28-70
Depth in feet
0
20
40
60
80
100
2"
1"
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Firm, tan, weathered serpentine fragments w/sandy silt.
Soft, brown, sandy silt w/gravel
Stiff, tan, silty clay
Wet brown, clay, sand & gravel
Wet, stiff, gray clay
Moist to wet, stiff to hard interbedded clayey silt & sandy silt w/ lenses of fine sand
Stiff gray-green, silty clay
Hard, brown, interbedded silty sand & silty clay
Wet dense, brown, fine to coarse sand
Stiff, gray & brown silty clay
Hard, brown clay w/rock fragments
Moist brown, weathered rhyolite
24



Sta 225+38
110' Rt. 'A' Line
Br13-70
2.5"
Elev. 16'±
7-28-70
0
20
40
60
80
100
Depth In Feet
2"
1"
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Firm to hard, dark brown silty clay.
Stiff to hard, tan, clayey silt.
Loose, lt. brown, fine silty sand and gravel.
Loose, brown, fine sand and silty sand, interbedded.
Dense, gray, silty sand & gravel.
Dense gray, silty clay with gravel
Dense, gray, silty sand.
Dense, gray-tan, clayey silt
Loose to dense, lt. brown silty sand with clay.
Hard, lt. brown, silty clay embedded with dense gravel.

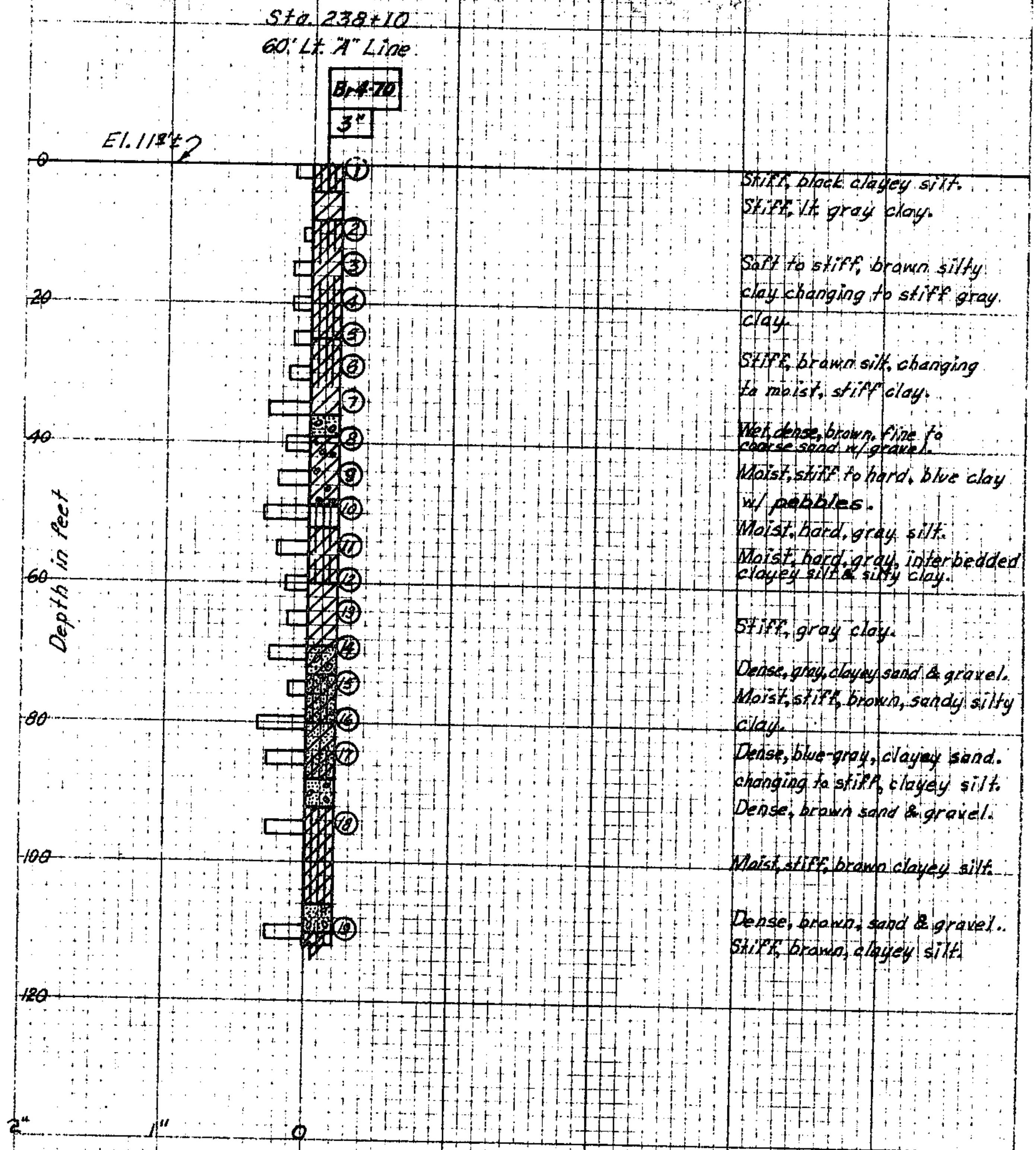


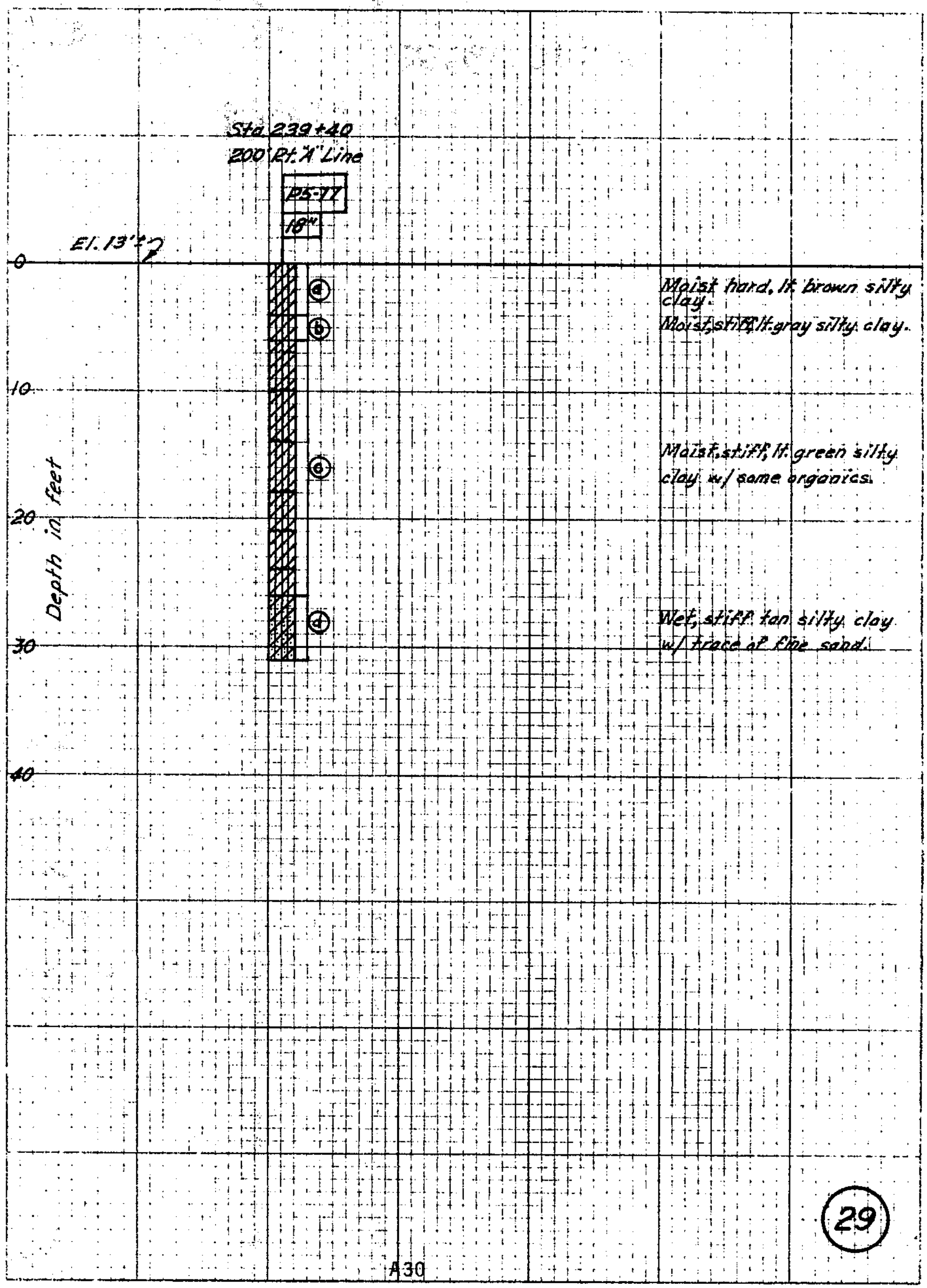
Sta. 228+50
175' Rt "A" Line
P6-77
18"
El. 15½'±
0
10
20
30
40
Depth in feet
a
b
c
Moist to wet firm to stiff, gray silty clay.
26

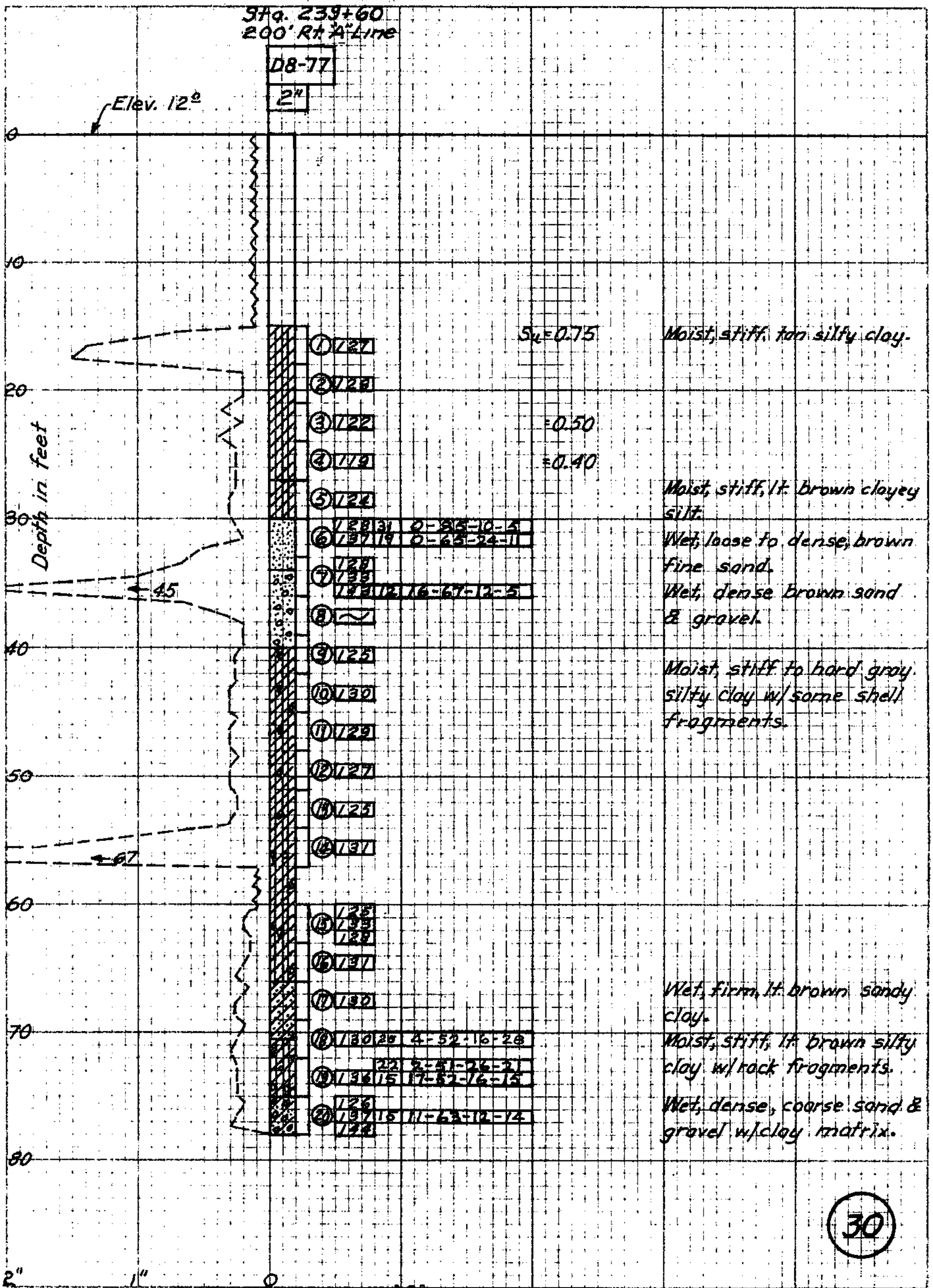


Sta 236+75
225' Rt. 'A' Line
D12-77
2"
Elev. 13º
0
10
20
30
10-3-77
Depth in feet
1 128
2 138 15
3 139 18
4 133 128 22 0-9-54-37
5 127
6 127
7 132
8 127 26 0-10-53-37
9 130 123 24 0-12-60-28
S_u = 1.50
= 1.10
= 1.00
= 1.50
Moist, stiff, brown silty clay w/ shale fragments.
Moist to wet, dense brown sandy clay and coarse gravel.
Moist, stiff, lt brown interbedded sandy silt and silty clay.
Moist, firm to stiff, brown silty clay w/ pockets of shale fragments.
2"
1"
0

27

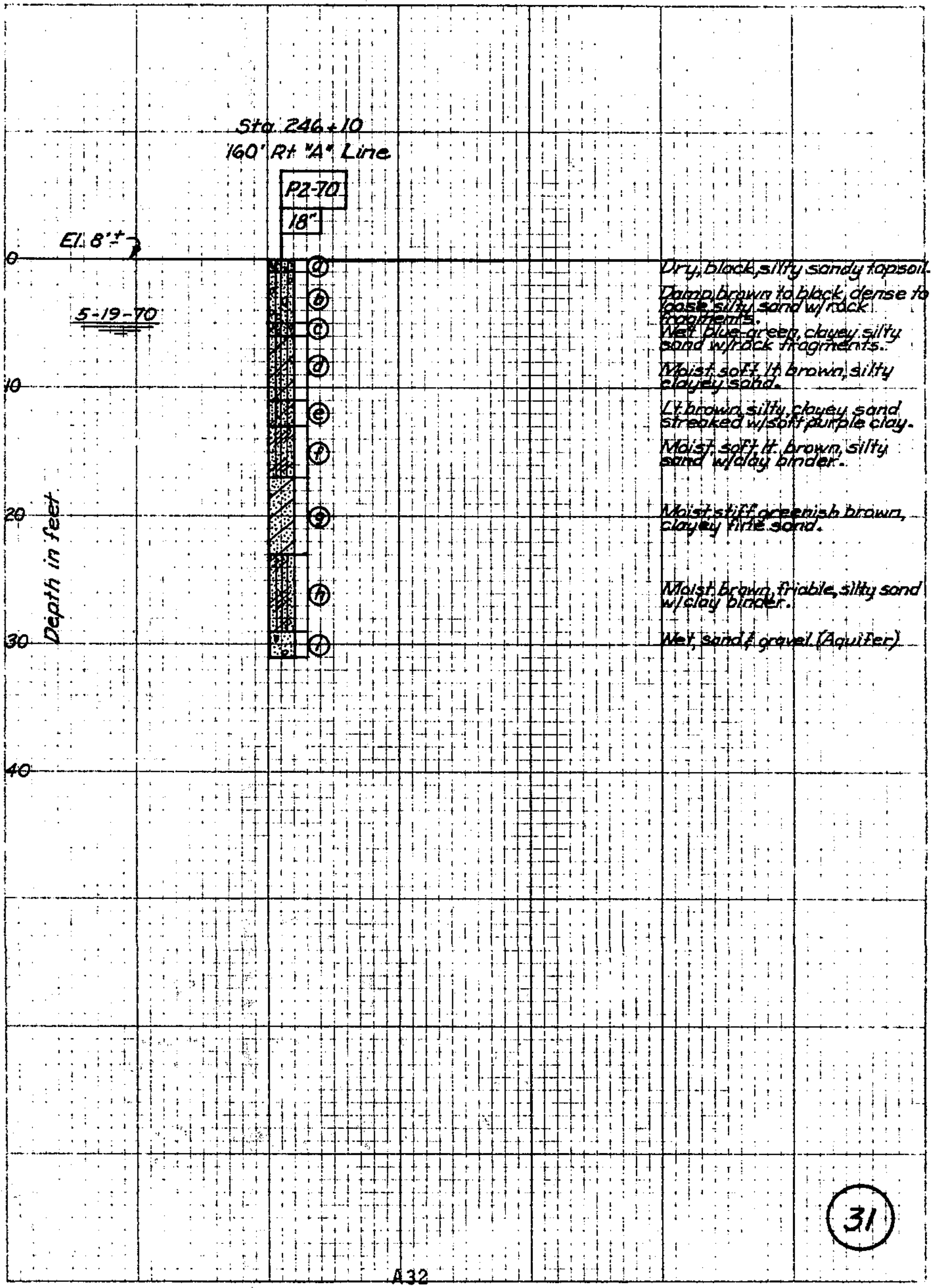

Sta. 238+10
60' Lt. "A" Line
Br.4-70
3"
El. 118'±
0
20
40
60
80
100
120
Depth in feet
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
Stiff, black clayey silt.
Stiff, lt. gray clay.
Soft to stiff, brown silty clay changing to stiff gray clay.
Stiff, brown silt, changing to moist, stiff clay.
Wet, dense, brown, fine to coarse sand w/ gravel.
Moist, stiff to hard, blue clay w/ pebbles.
Moist, hard, gray silt.
Moist, hard, gray, interbedded clayey silt & silty clay.
Stiff, gray clay.
Dense, gray, clayey sand & gravel.
Moist, stiff, brown, sandy silty clay.
Dense, blue-gray, clayey sand. changing to stiff, clayey silt.
Dense, brown sand & gravel.
Moist, stiff, brown clayey silt.
Dense, brown, sand & gravel..
Stiff, brown, clayey silt.
2"
1"
0

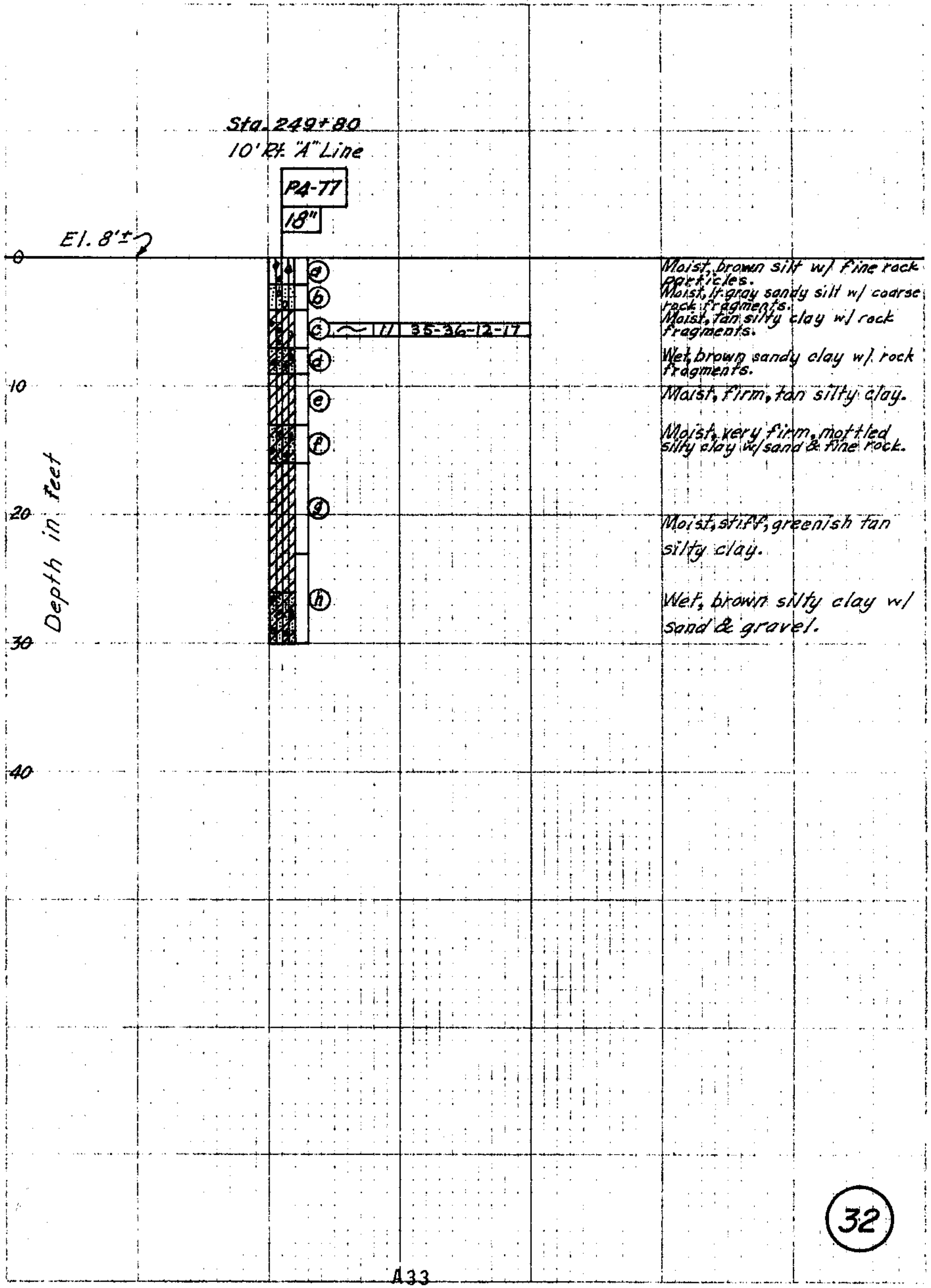

Sta 239+40
200' Rt. "A" Line
PS-77
18"
El. 13'±
0
10
20
30
40
Depth in feet
a
b
c
d
Moist hard, lt brown silty clay.
Moist, stiff, lt. gray silty clay.
Moist, stiff, lt. green silty clay w/ some organics.
Wet, stiff tan silty clay w/ trace of fine sand.
29

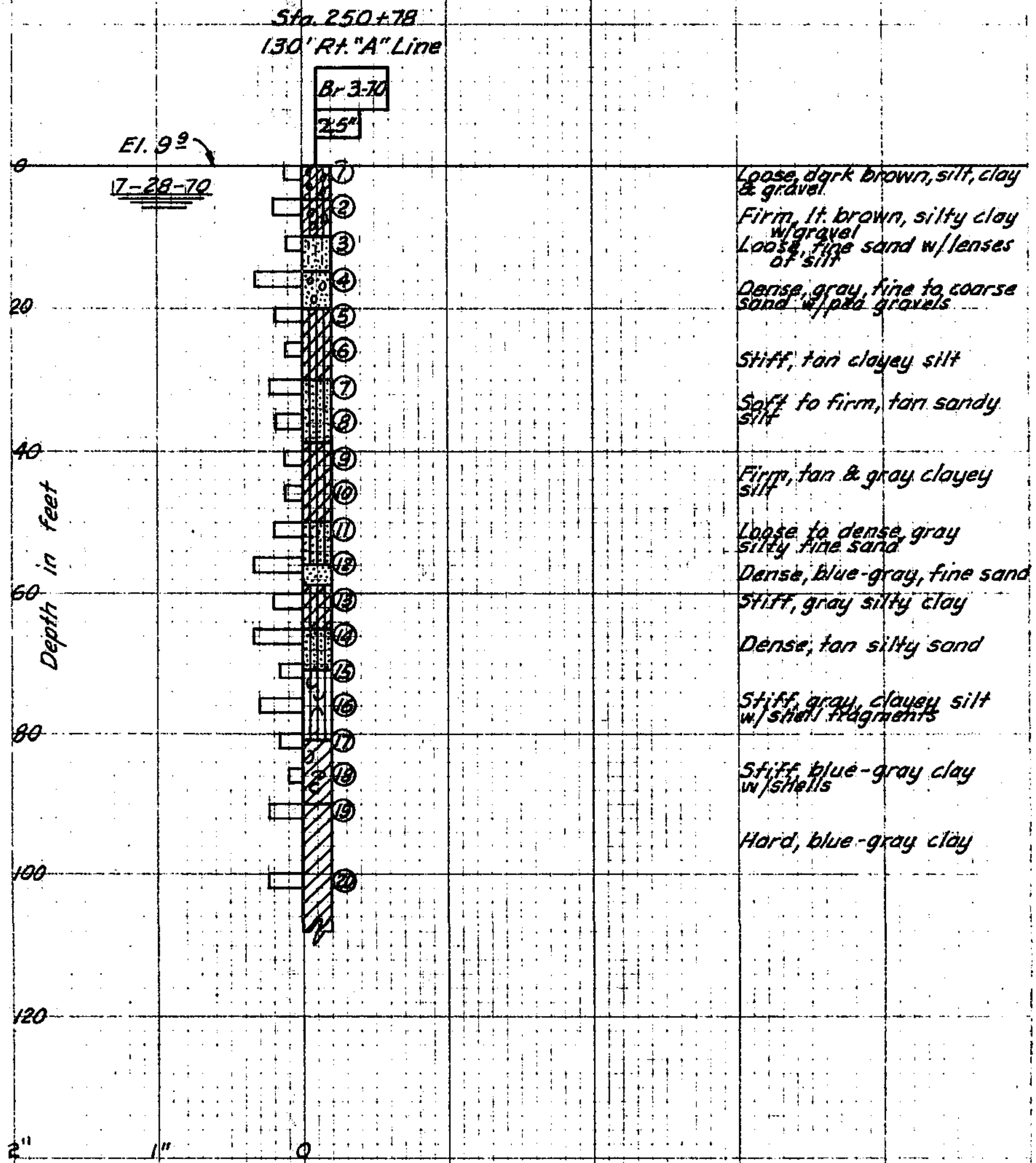


Sta. 239+60
200' Rt "A" Line
D8-77
2"
Elev. 12.0
Depth in feet
0
10
20
30
40
50
60
70
80
2"
1"
0
Su=0.75
=0.50
=0.40
45
67
Moist, stiff, tan silty clay.
Moist, stiff, lt. brown clayey silt.
Wet, loose to dense, brown fine sand.
Wet, dense brown sand & gravel.
Moist, stiff to hard gray silty clay w/some shell fragments.
Wet, firm, lt. brown sandy clay.
Moist, stiff, lt. brown silty clay w/rock fragments.
Wet, dense, coarse sand & gravel w/clay matrix.

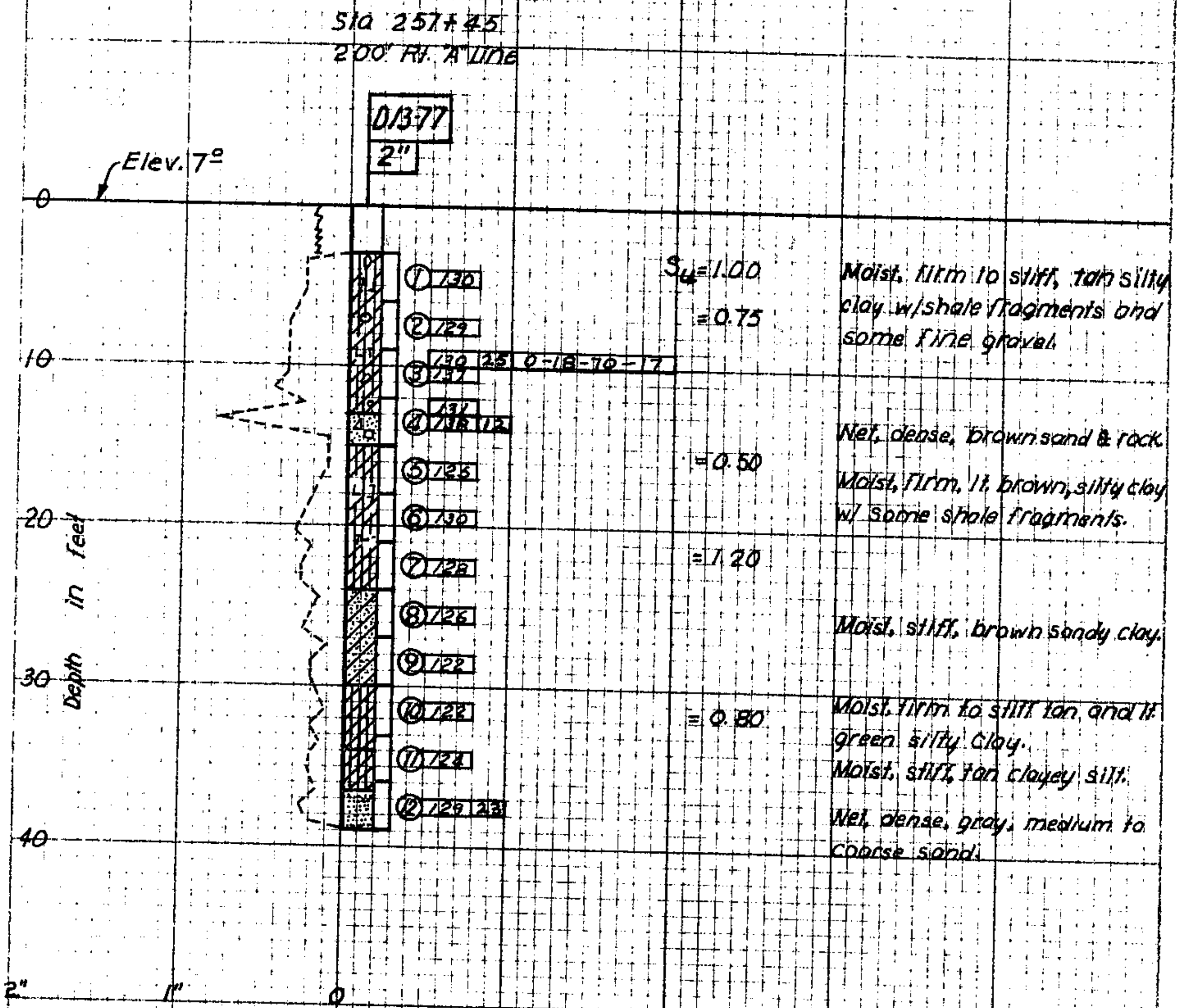
30


Sta. 246+10
160' Rt "A" Line
PZ-70
18"
El. 8'±
5-19-70
0
10
20
30
40
Depth in feet
a
b
c
d
e
f
g
h
i
Dry, black, silty sandy topsoil.
Damp brown to black, dense to loose silty sand w/rock fragments.
Wet blue-green, clayey silty sand w/rock fragments.
Moist, soft, lt. brown, silty clayey sand.
Lt. brown, silty, clayey sand streaked w/soft purple clay.
Moist, soft, lt. brown, silty sand w/clay binder.
Moist, stiff, greenish brown, clayey fine sand.
Moist brown, friable, silty sand w/clay binder.
Wet, sand & gravel. (Aquifer)
31

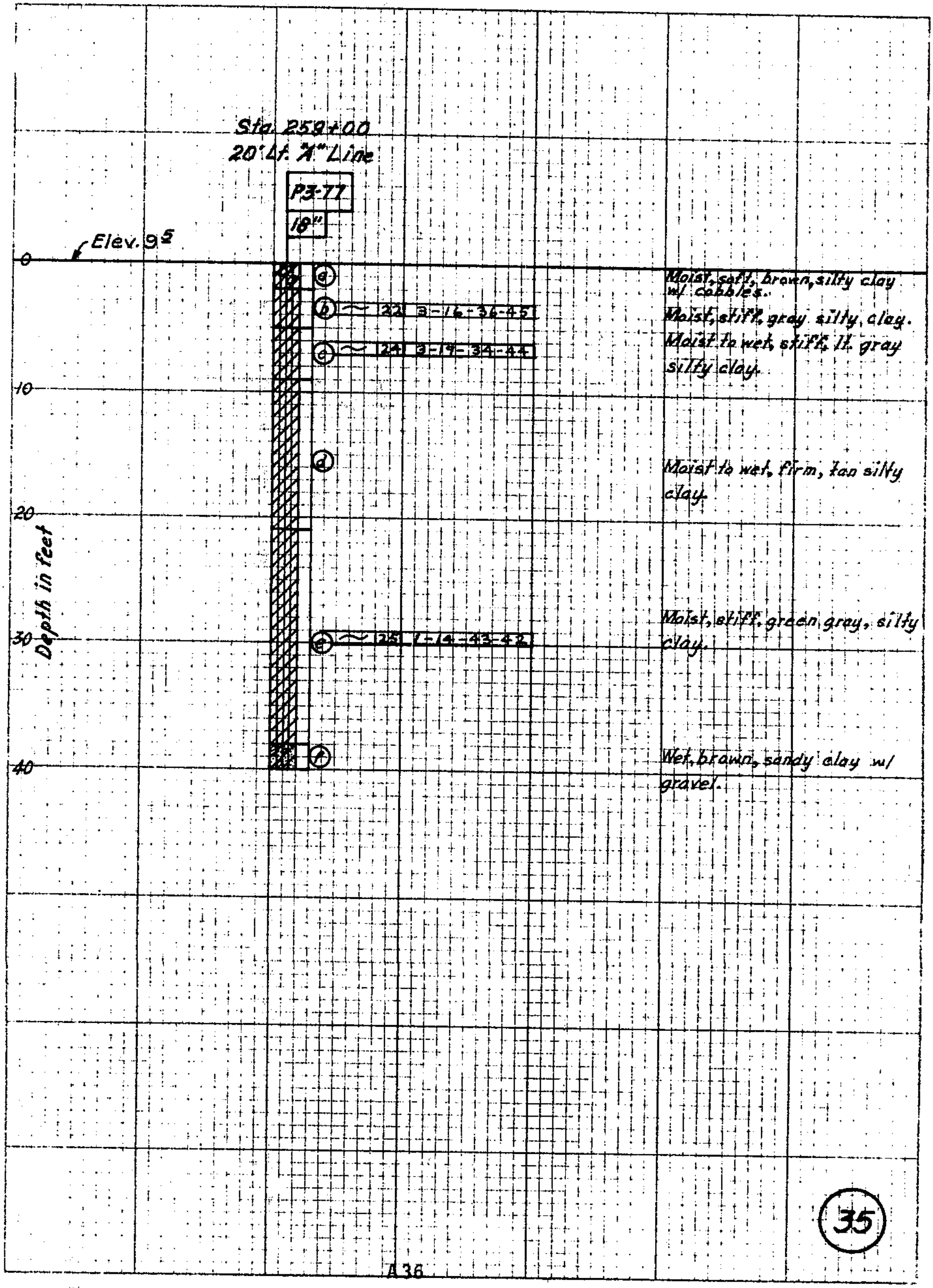

Sta. 249+80
10' Rt. "A" Line
P4-77
18"
El. 8'±
0
10
20
30
40
Depth in feet
a
b
c
d
e
f
g
h
11 35-36-12-17
Moist, brown silt w/ fine rock particles.
Moist, lt. gray sandy silt w/ coarse rock fragments.
Moist, tan silty clay w/ rock fragments.
Wet, brown sandy clay w/ rock fragments.
Moist, firm, tan silty clay.
Moist, very firm, mottled silty clay w/ sand & fine rock.
Moist, stiff, greenish tan silty clay.
Wet, brown silty clay w/ sand & gravel.


Sta. 250+78
130' Rt. "A" Line
Br-3-70
2.5"
El. 9.9
7-28-70
0
20
40
60
80
100
120
Depth in feet
2"
1"
0
Loose, dark brown, silt, clay & gravel
Firm, lt. brown, silty clay w/gravel
Loose, fine sand w/lenses of silt
Dense, gray, fine to coarse sand w/pea gravels
Stiff, tan clayey silt
Soft to firm, tan sandy silt
Firm, tan & gray clayey silt
Loose to dense, gray silty fine sand
Dense, blue-gray, fine sand
Stiff, gray silty clay
Dense, tan silty sand
Stiff, gray, clayey silt w/shell fragments
Stiff, blue-gray clay w/shells
Hard, blue-gray clay

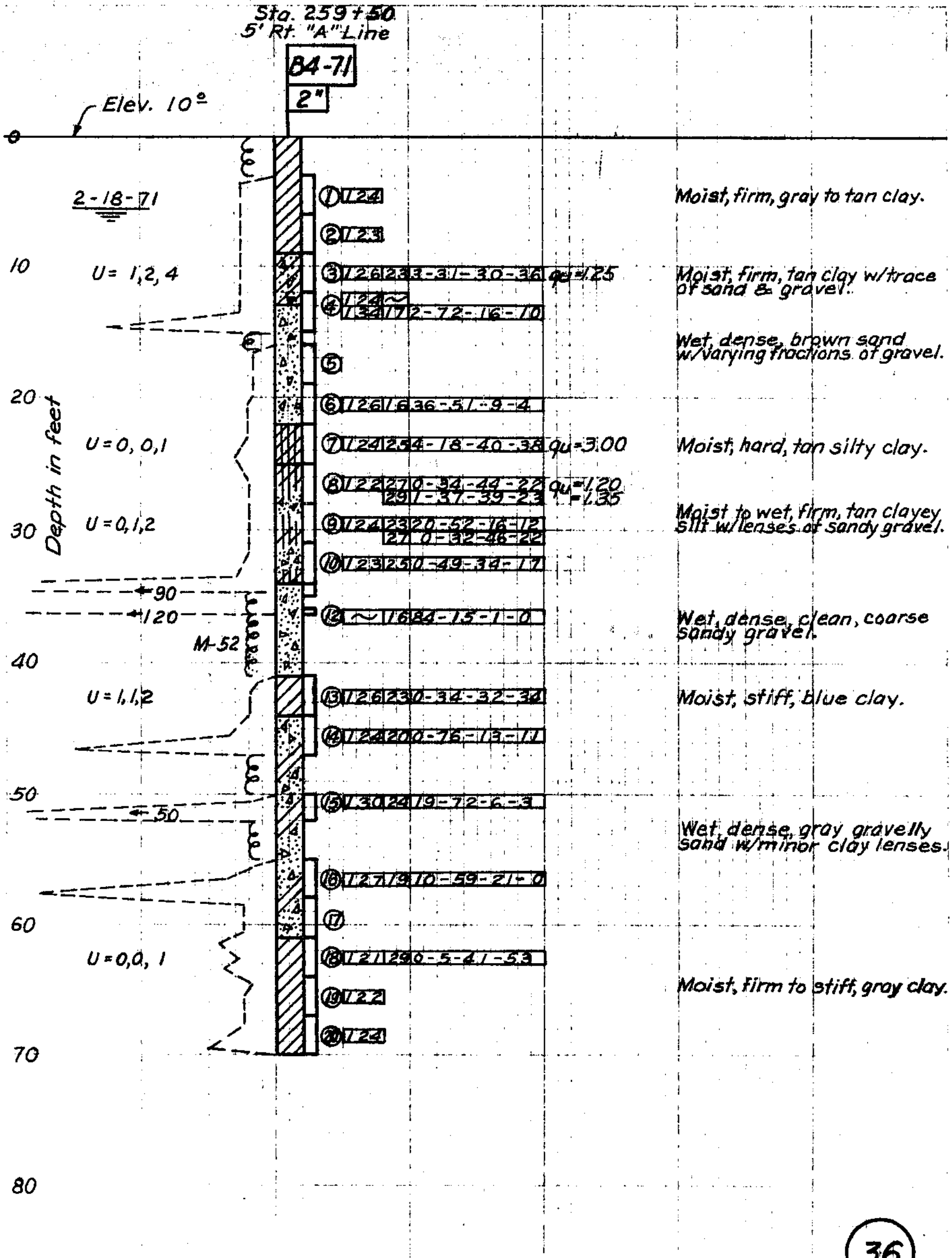
33

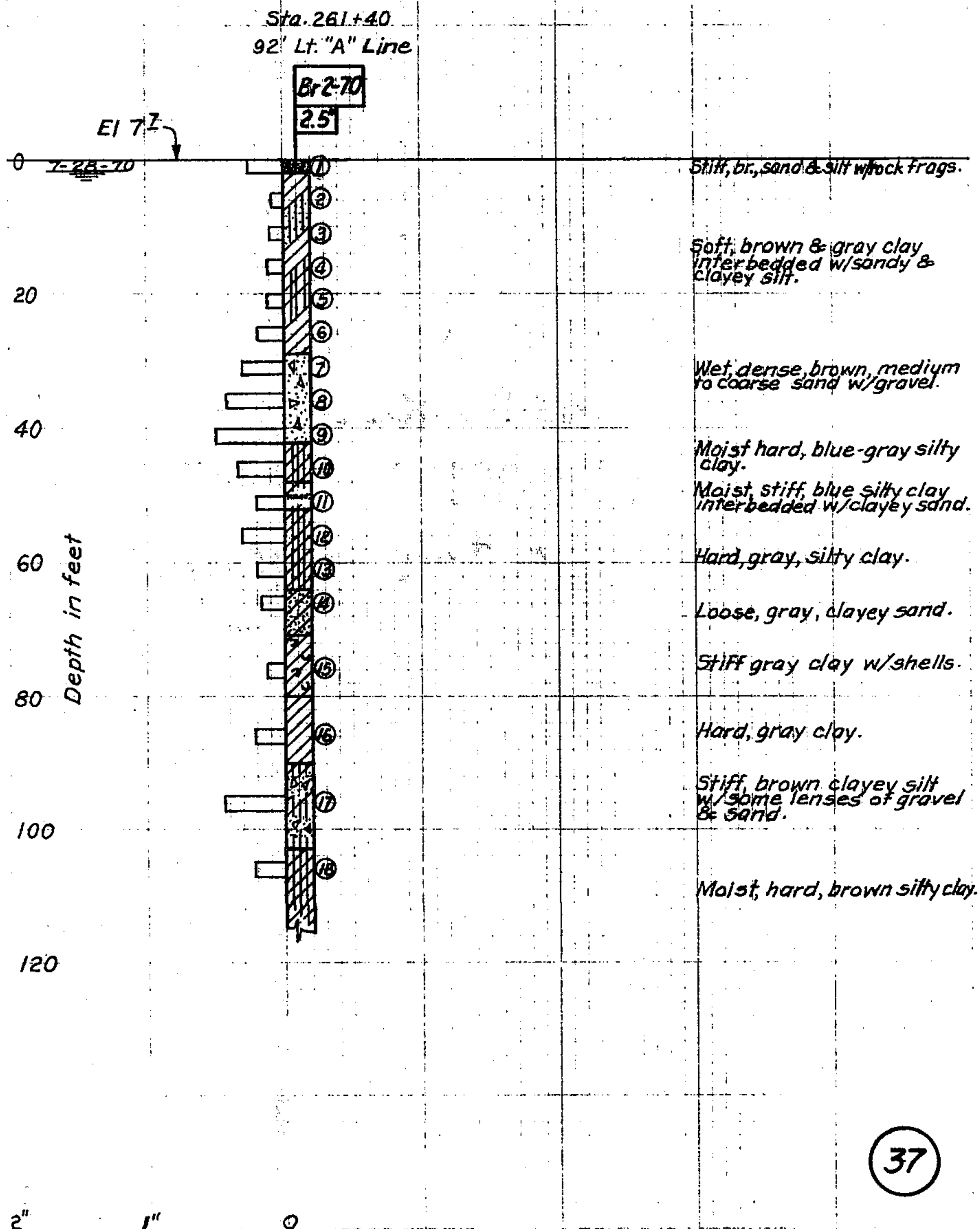


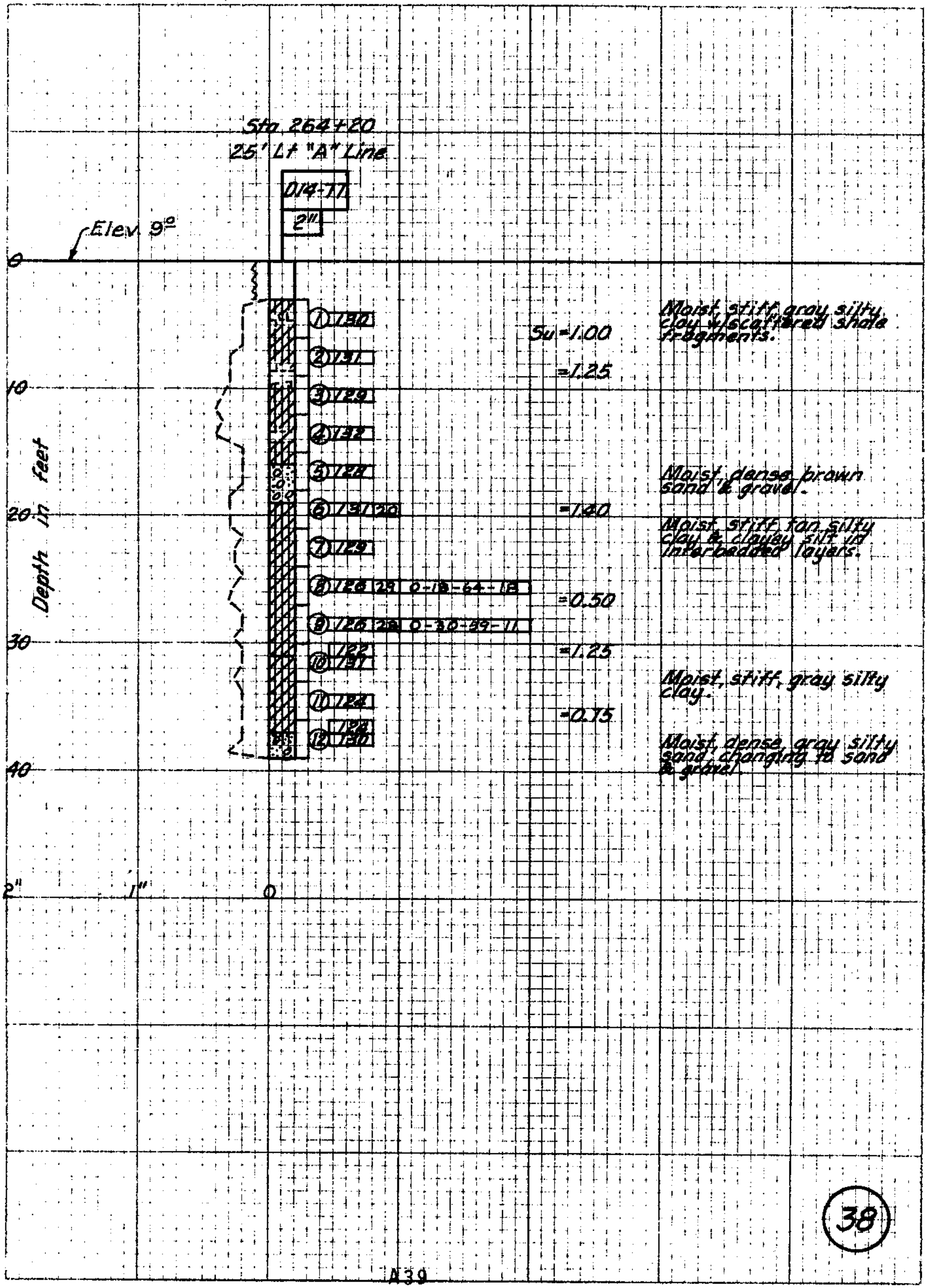
Sta 257+45
200' Rt. "A" Line
D/377
2"
Elev. 7º
0
10
20
30
40
Depth in feet
2"
1"
0
① 130
② 124
③ 130 25 0-18-70-17
131
④ 131
118 112
⑤ 125
⑥ 130
⑦ 128
⑧ 126
⑨ 122
⑩ 122
⑪ 124
⑫ 129 23
S_u = 1.00
= 0.75
= 0.50
= 1.20
= 0.80
Moist, firm to stiff, tan silty clay w/shale fragments and some fine gravel.
Wet, dense, brown sand & rock.
Moist, firm, lt brown, silty clay w/ some shale fragments.
Moist, stiff, brown sandy clay.
Moist, firm to stiff tan and lt green silty clay.
Moist, stiff, tan clayey silt.
Wet, dense, gray, medium to coarse sand.

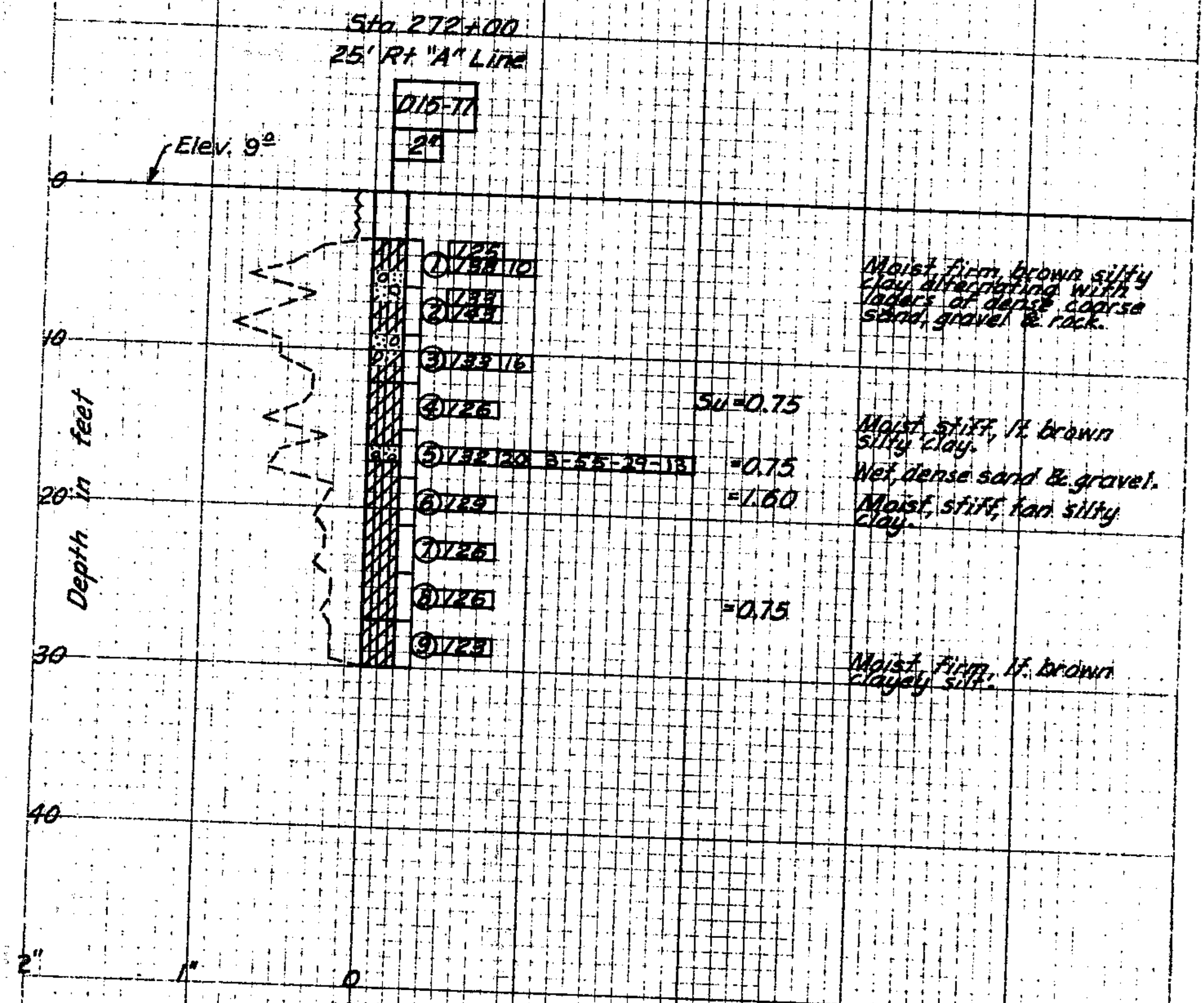


Sta. 259+00
20' Lt. "A" Line
P3-77
18"
Elev. 9.5
0
10
20
30
40
Depth in feet
a
b
22
3-16-36-45
c
24
3-19-34-44
d
e
25
7-14-43-42
f
Moist, soft, brown, silty clay w/ cobbles.
Moist, stiff, gray, silty, clay.
Moist to wet, stiff, lt. gray silty clay.
Moist to wet, firm, tan silty clay.
Moist, stiff, green gray, silty clay.
Wet, brown, sandy clay w/ gravel.
35

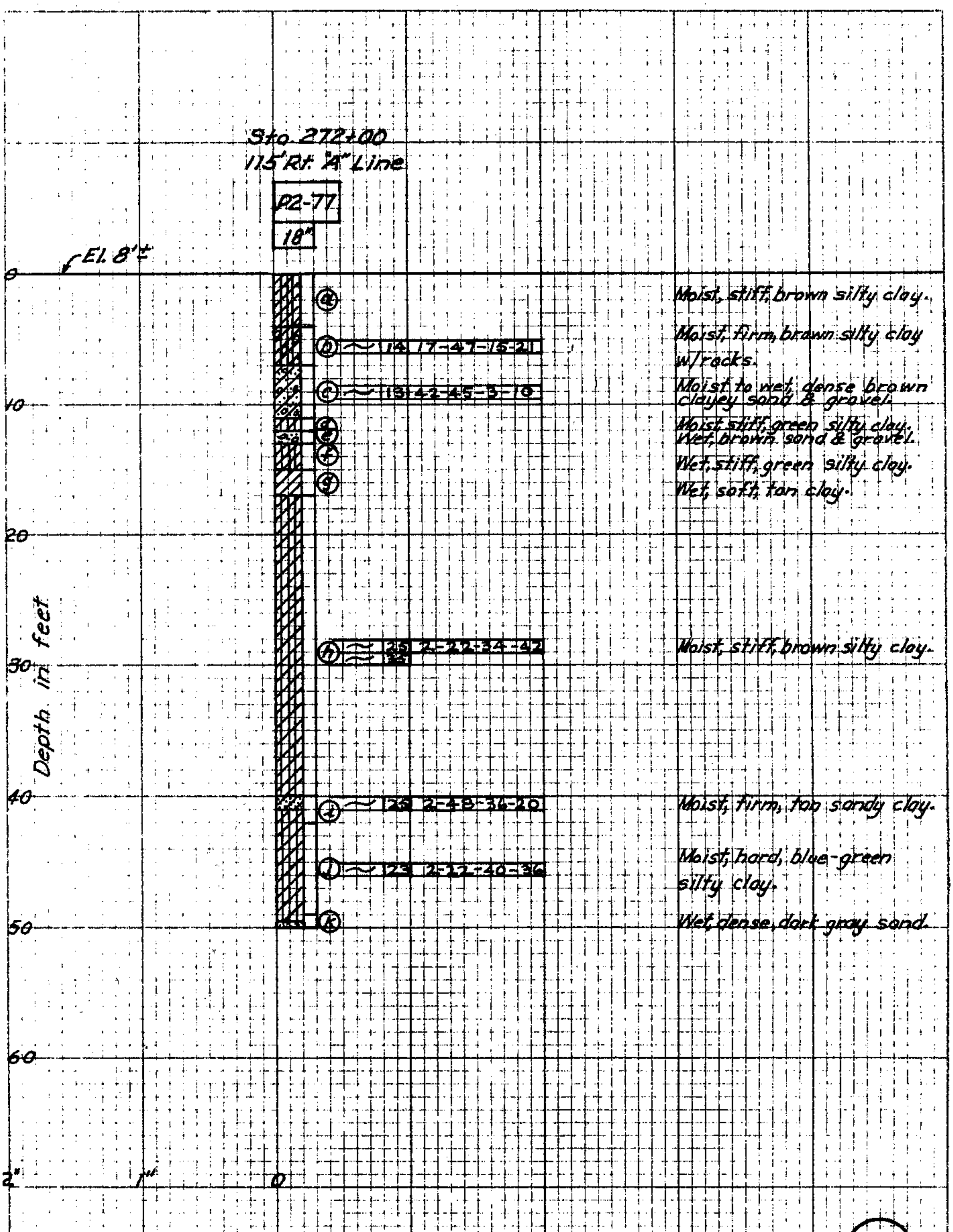

Sta. 259+50
5' Rt. "A" Line
B4-71
2"
Elev. 10°
2-18-71
Depth in feet
10
20
30
40
50
60
70
80
U= 1,2,4
U=0, 0,1
U=0,1,2
U=1,1,2
U=0,0, 1
90
120
50
M-52
① 1 24
② 1 23
③ 1 26 23 3-31-30-36 qu=1.25
④ 1 24 ~ 1 34 17 2-72-16-10
⑤
⑥ 1 26 16 36-51-9-4
⑦ 1 24 25 4-18-40-38 qu=3.00
⑧ 1 22 27 0-34-44-22 qu=1.20 29 1-37-39-23 =1.35
⑨ 1 24 23 20-52-16-12 27 0-32-46-22
⑩ 1 23 25 0-49-34-17
⑫ ~ 16 84-15-1-0
⑬ 1 26 23 0-34-32-34
⑭ 1 24 20 0-76-13-11
⑮ 1 30 24 19-72-6-3
⑯ 1 27 19 10-59-21-0
⑰
⑱ 1 21 29 0-5-41-53
⑲ 1 22
⑳ 1 24
Moist, firm, gray to tan clay.
Moist, firm, tan clay w/trace of sand & gravel.
Wet, dense, brown sand w/varying fractions of gravel.
Moist, hard, tan silty clay.
Moist to wet, firm, tan clayey silt w/lenses of sandy gravel.
Wet, dense, clean, coarse sandy gravel.
Moist, stiff, blue clay.
Wet, dense, gray gravelly sand w/minor clay lenses.
Moist, firm to stiff, gray clay.
2"
1"
0


Sta. 261+40
92' Lt. "A" Line
Br 2-70
2.5'
El 7.7
7-28-70
0
20
40
60
80
100
120
Depth in feet
Stiff, br., sand & silt w/rock frags.
Soft, brown & gray clay interbedded w/sandy & clayey silt.
Wet, dense, brown, medium to coarse sand w/gravel.
Moist hard, blue-gray silty clay.
Moist, stiff, blue silty clay interbedded w/clayey sand.
Hard, gray, silty clay.
Loose, gray, clayey sand.
Stiff gray clay w/shells.
Hard, gray clay.
Stiff, brown clayey silt w/some lenses of gravel & sand.
Moist, hard, brown silty clay.
2"
1"
0
37

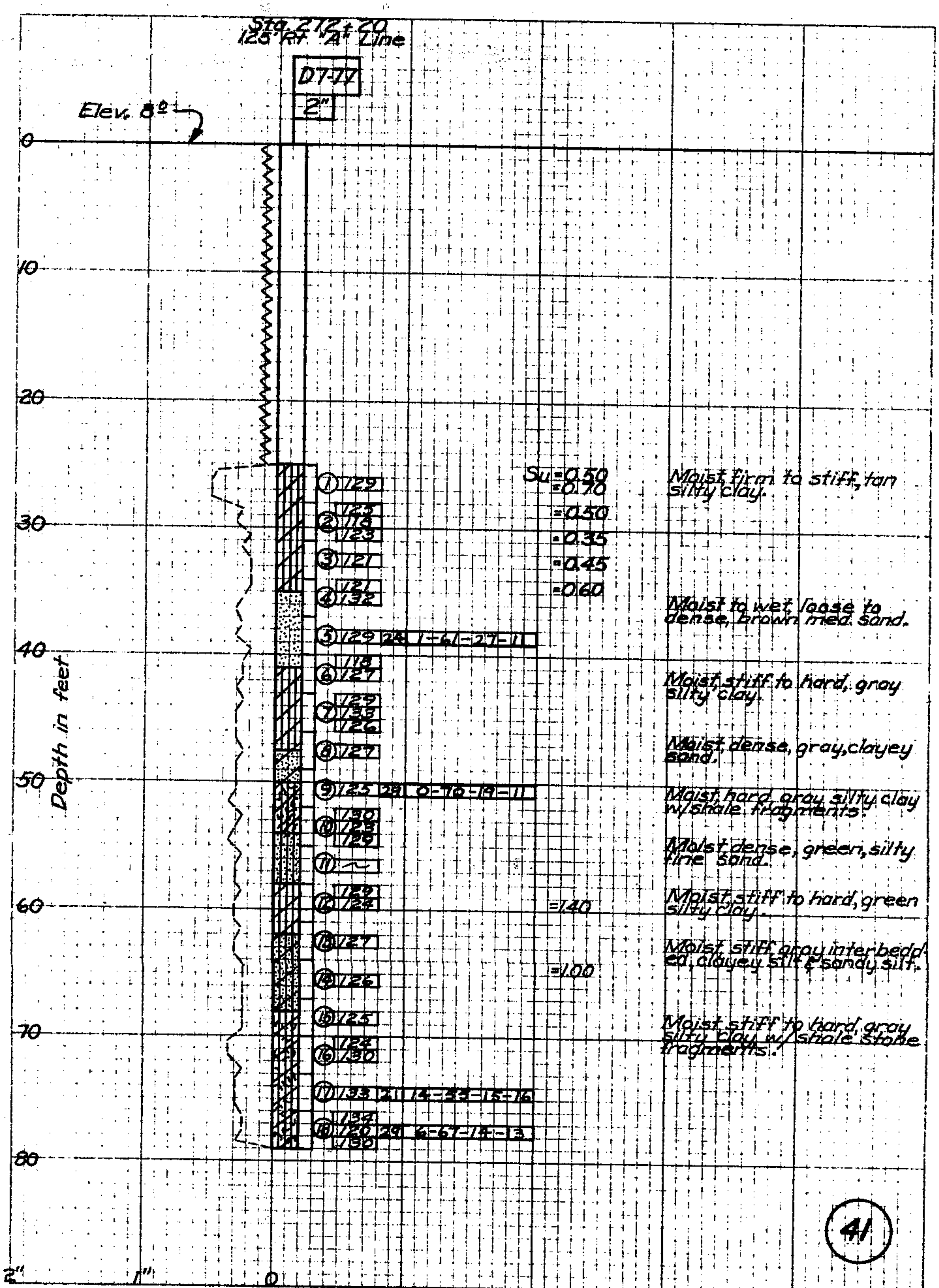

Sta 264+20
25' Lt "A" Line
D14-77
2"
Elev. 9.0
0
10
20
30
40
Depth in feet
1 130
2 131
3 129
4 132
5 128
6 131 20
7 129
8 128 23 0-19-64-18
9 126 28 0-30-59-11
122
10 131
11 124
128
12 130
Su=1.00
=1.25
=1.40
=0.50
=1.25
=0.75
Moist, stiff, gray silty clay w/scattered shale fragments.
Moist, dense brown sand & gravel.
Moist, stiff tan silty clay & clayey silt in interbedded layers.
Moist, stiff, gray silty clay.
Moist, dense, gray silty sand, changing to sand & gravel.
2"
1"
0
38

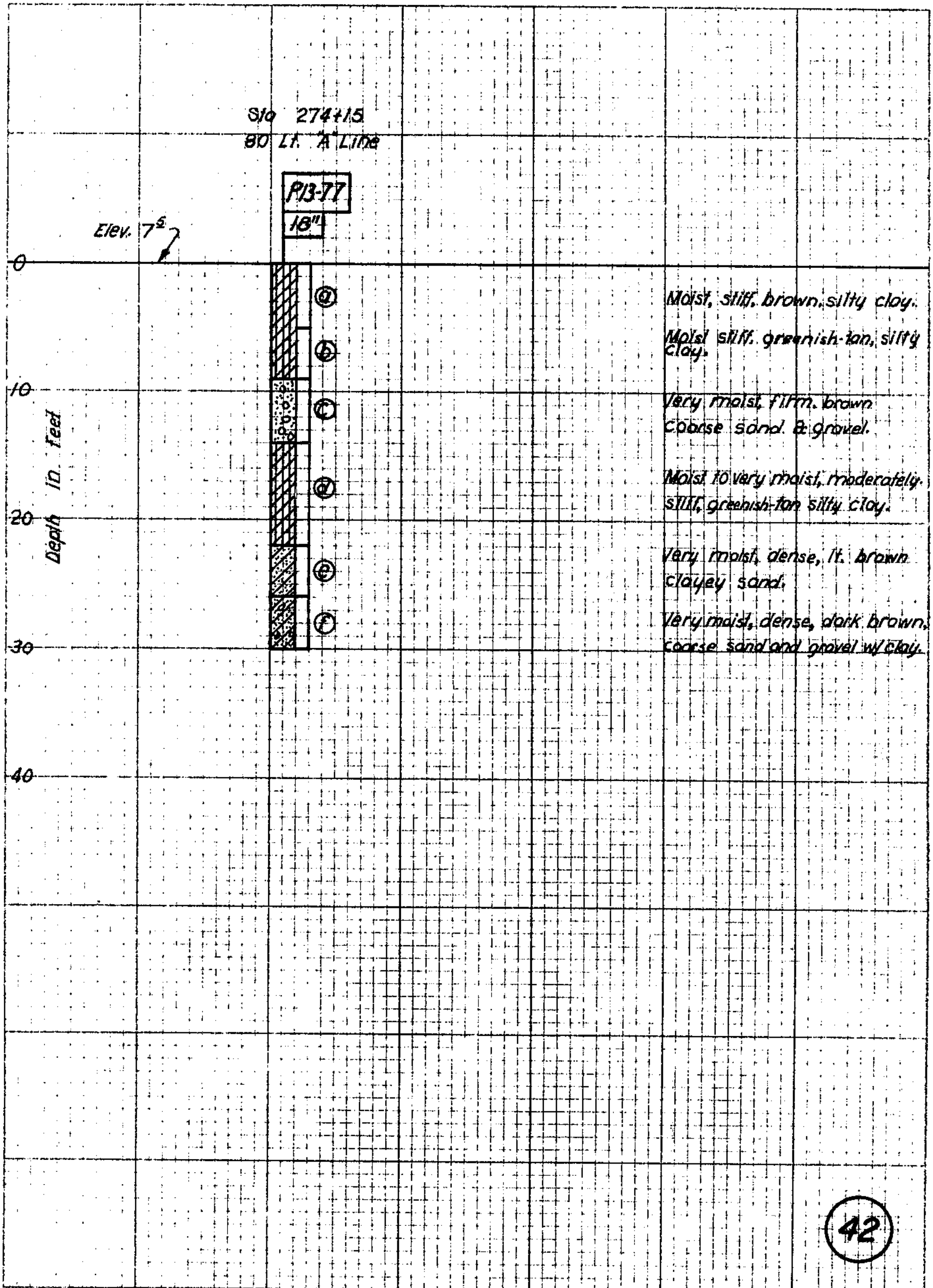

Sta. 272+00
25' Rt "A" Line
D15-77
2"
Elev. 9º
0
10
20
30
40
Depth in feet
① /25
/38 10
② /33
/43
③ /33 16
④ /26
⑤ /32 20 3-55-29-13
⑥ /29
⑦ /26
⑧ /26
⑨ /23
Su=0.75
=0.75
=1.60
=0.75
Moist, firm, brown silty clay alternating with layers of dense coarse sand, gravel & rock.
Moist, stiff, lt. brown silty clay.
Wet, dense sand & gravel.
Moist, stiff, tan silty clay.
Moist, firm, lt. brown clayey silt.
2"
1"
0

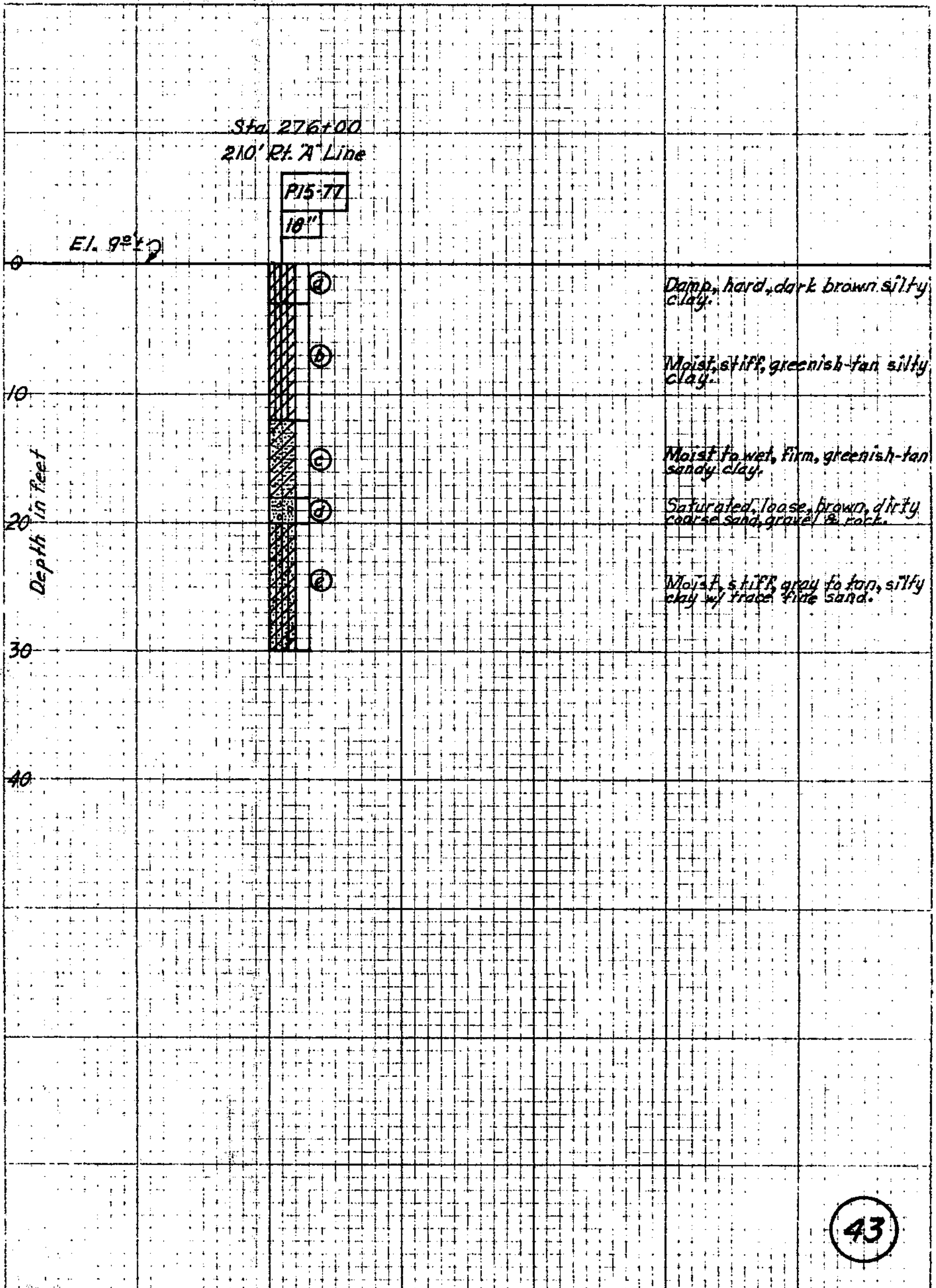
39

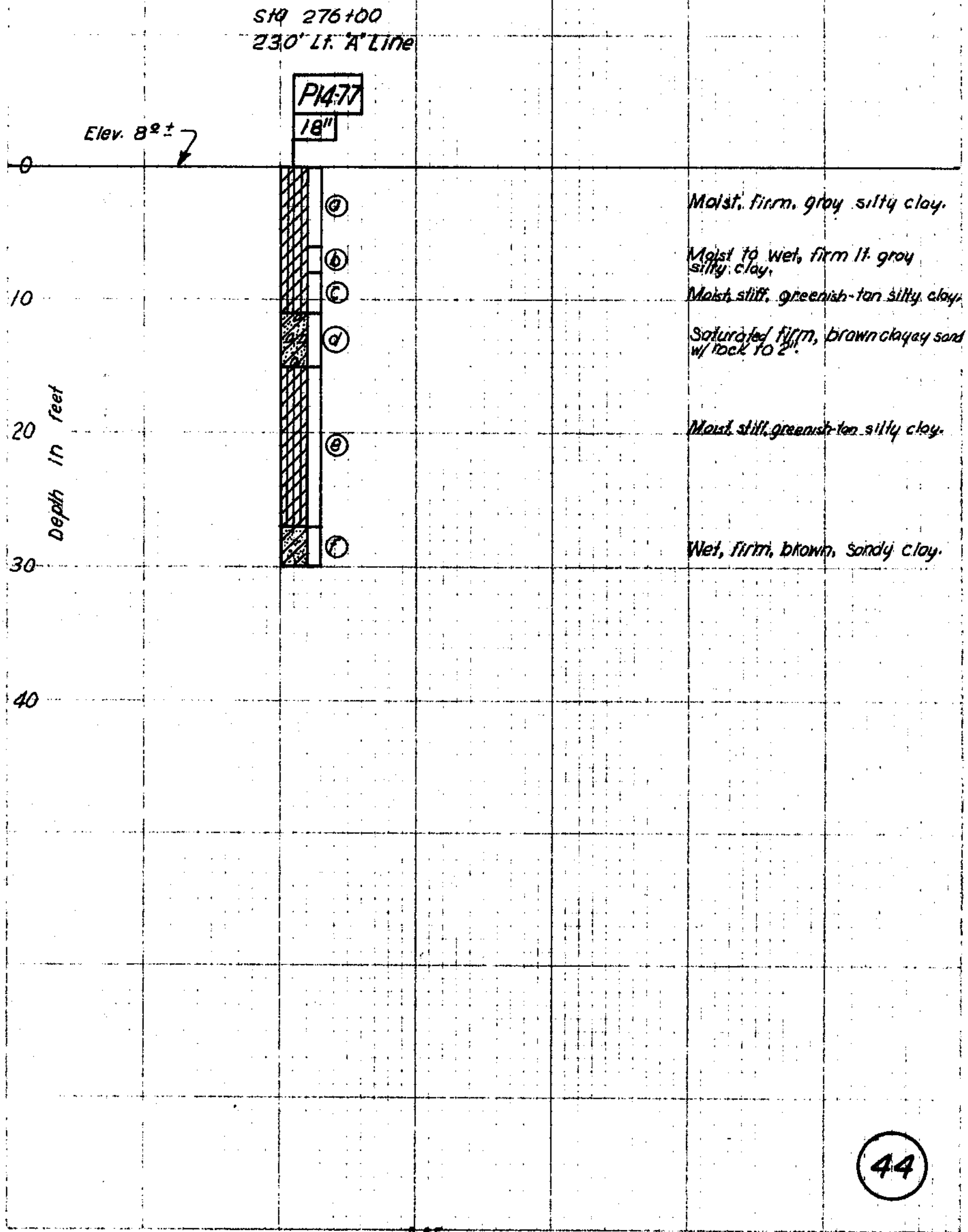


Sta. 272+00
115' Rt. "A" Line
P2-77
18"
El. 8'±
0
10
20
30
40
50
60
Depth in feet
2"
1"
0
a
b 114 17-47-15-21
c 113 42-45-3-10
d
e
f
g
h 25 2-22-34-42
i 25 2-48-34-20
j 123 2-22-40-36
k
Moist, stiff, brown silty clay.
Moist, firm, brown silty clay w/rocks.
Moist to wet, dense brown clayey sand & gravel.
Moist, stiff, green silty clay.
Wet, brown sand & gravel.
Wet, stiff, green silty clay.
Wet, soft, tan clay.
Moist, stiff, brown silty clay.
Moist, firm, tan sandy clay.
Moist, hard, blue-green silty clay.
Wet, dense, dark gray sand.

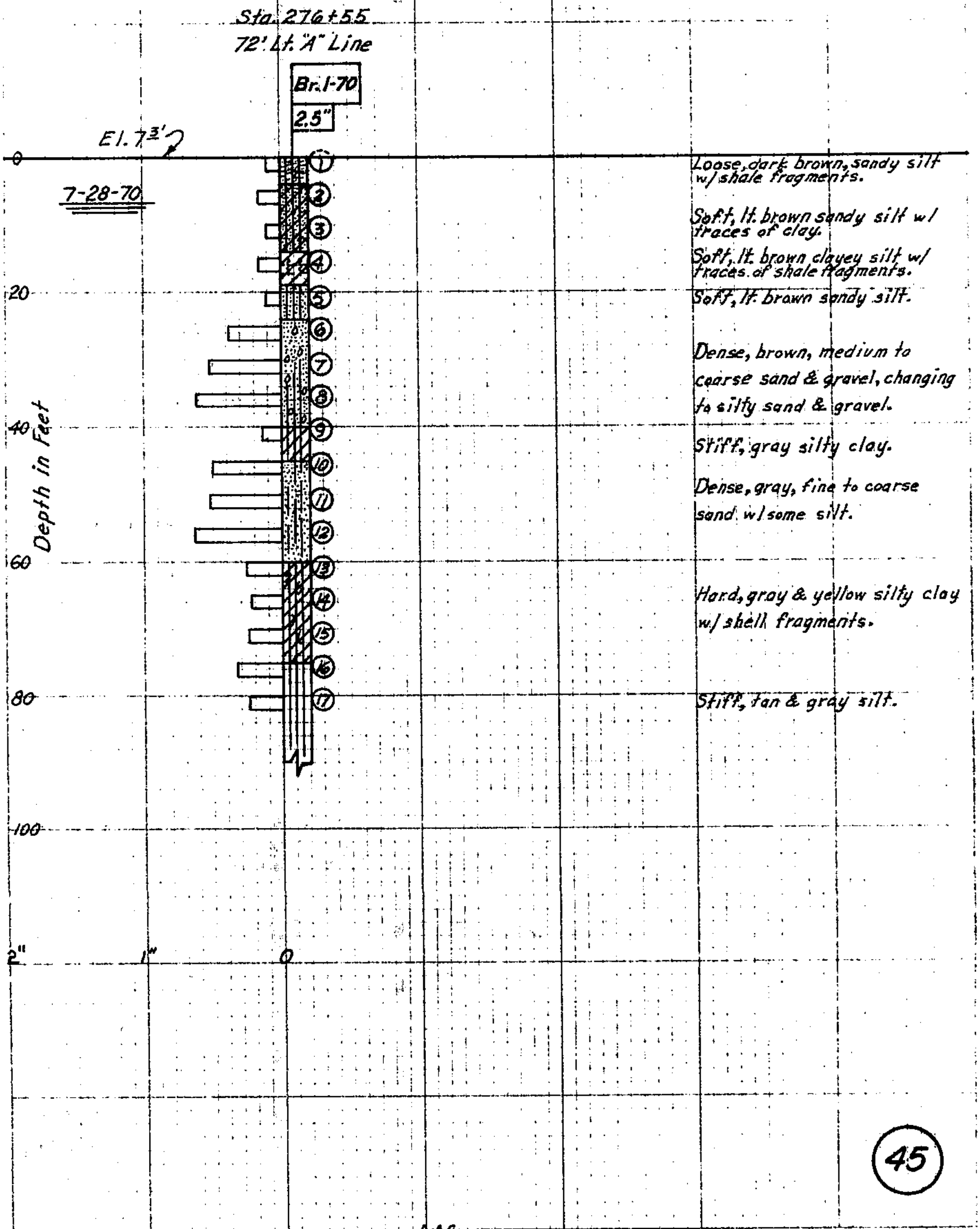

Sta. 212+20
125' Rt. "A" Line
D7-77
-2"
Elev. 8±
0
10
20
30
40
50
60
70
80
Depth in feet
① 129
② 125 116 123
③ 127
④ 121 132
⑤ 129 28 1-61-27-11
⑥ 118 127
⑦ 129 133 126
⑧ 127
⑨ 125 28 0-70-19-11
⑩ 130 123 129
⑪ ~
⑫ 129 124
⑬ 127
⑭ 126
⑮ 125
⑯ 124 130
⑰ 133 21 14-53-15-16
⑱ 134 120 130 29 6-67-14-13
Su=0.50
=0.70
=0.50
=0.35
=0.45
=0.60
=1.40
=1.00
Moist, firm to stiff, tan silty clay.
Moist to wet, loose to dense, brown med. sand.
Moist, stiff to hard, gray silty clay.
Moist, dense, gray, clayey sand.
Moist, hard, gray silty clay w/shale fragments.
Moist, dense, green, silty fine sand.
Moist, stiff to hard, green silty clay.
Moist, stiff, gray interbedded, clayey silt & sandy silt.
Moist, stiff to hard, gray silty clay w/shale stone fragments.
41
2"
1"
0

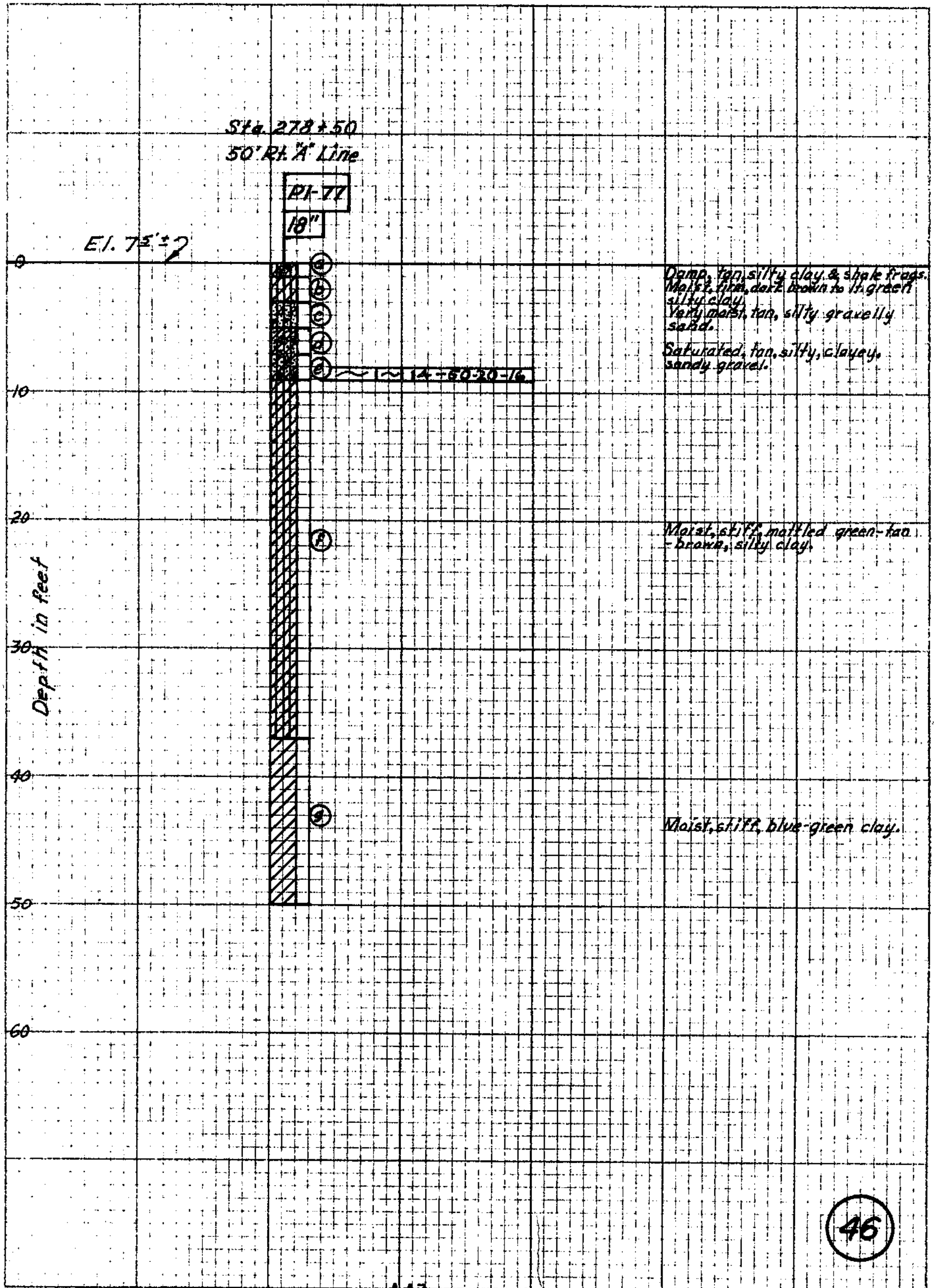

Sta 274+15
80 Lt. "A" Line
P13-77
18"
Elev. 7.5
0
10
20
30
40
Depth in feet
a
b
c
d
e
f
Moist, stiff, brown, silty clay.
Moist stiff, greenish-tan, silty clay.
Very moist, firm, brown coarse sand & gravel.
Moist to very moist, moderately stiff, greenish-tan silty clay.
Very moist, dense, lt. brown clayey sand.
Very moist, dense, dark brown, coarse sand and gravel w/clay.
42


Sta. 276+00
210' Rt. "A" Line
P15-77
18"
El. 9±?
0
10
20
30
40
Depth in feet
a
b
c
d
e
Damp, hard, dark brown silty clay.
Moist, stiff, greenish-tan silty clay.
Moist to wet, firm, greenish-tan sandy clay.
Saturated, loose, brown, dirty coarse sand, gravel & rock.
Moist, stiff, gray to tan, silty clay w/ trace fine sand.
43

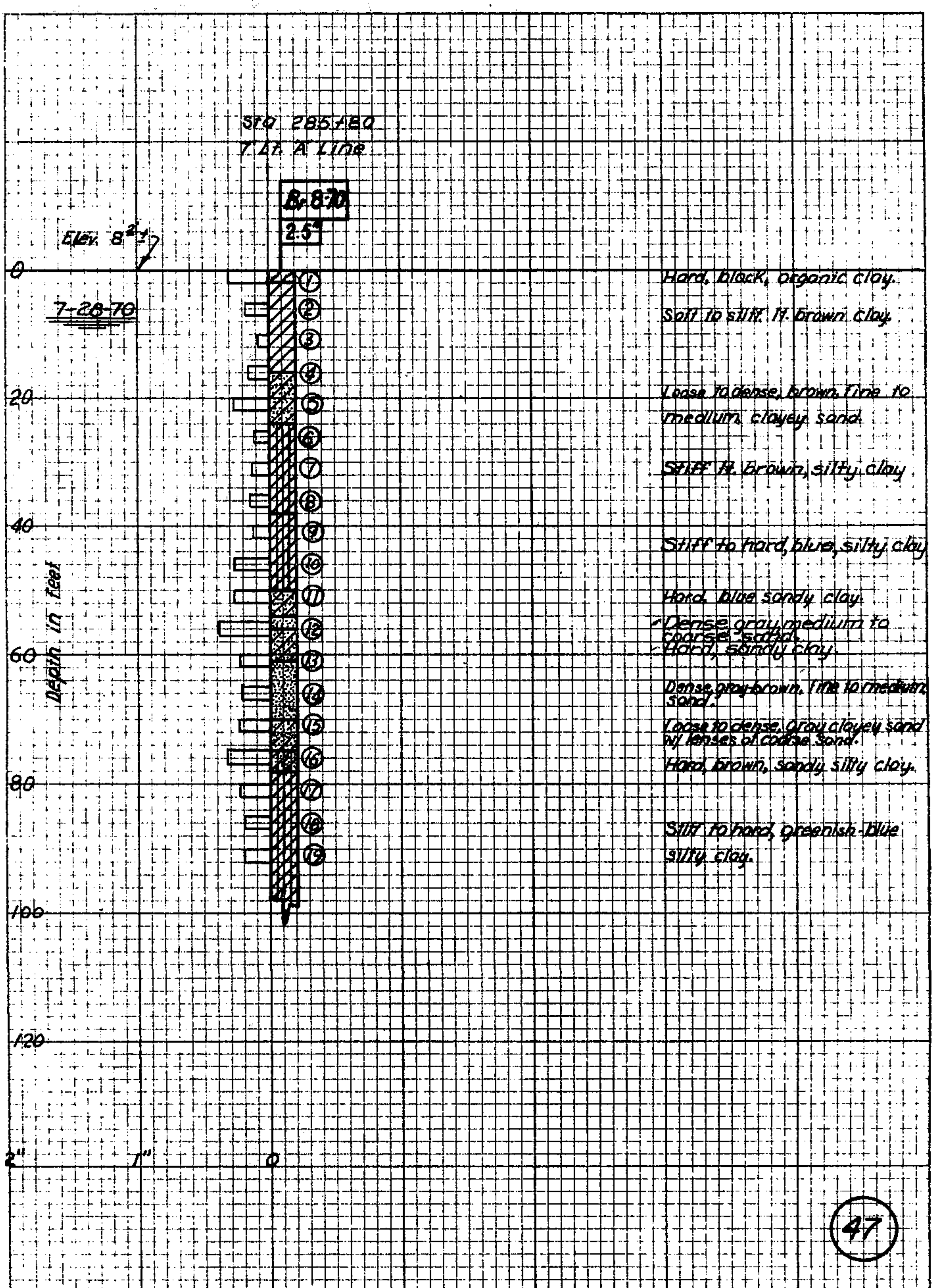


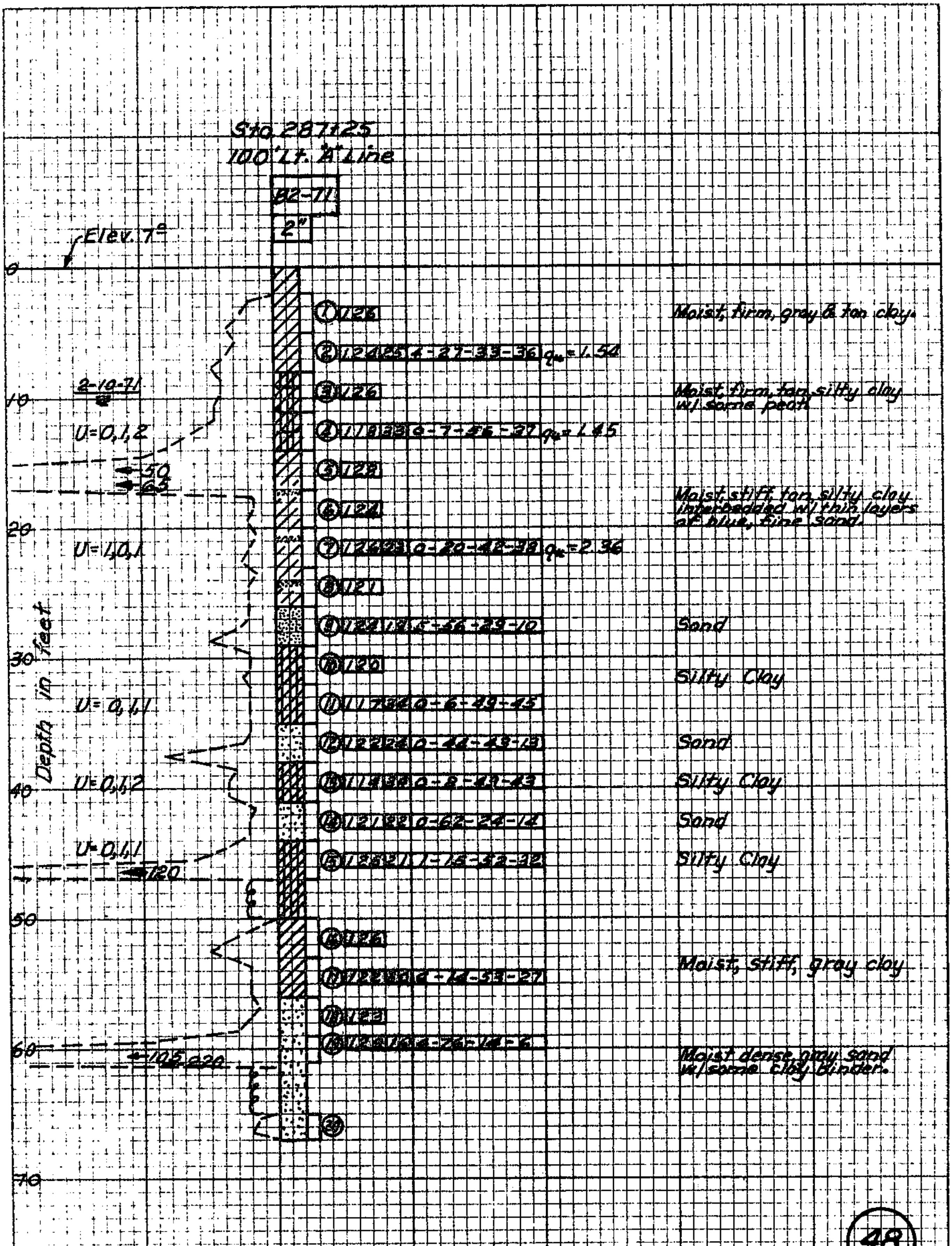
Sta 276+00
230' Lt. "A" Line
P14-77
18"
Elev. 8±
0
10
20
30
40
Depth in feet
a
b
c
d
e
f
Moist, firm, gray silty clay.
Moist to wet, firm lt. gray silty clay.
Moist, stiff, greenish-tan silty clay.
Saturated firm, brown clayey sand w/ rock to 2".
Moist stiff, greenish-tan silty clay.
Wet, firm, brown, sandy clay.
44


Sta. 276+55
72' Lt. "A" Line
Br. I-70
2.5"
El. 7.3'
7-28-70
Depth in Feet
0
20
40
60
80
100
2"
1"
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Loose, dark brown, sandy silt w/ shale fragments.
Soft, lt. brown sandy silt w/ traces of clay.
Soft, lt. brown clayey silt w/ traces of shale fragments.
Soft, lt. brown sandy silt.
Dense, brown, medium to coarse sand & gravel, changing to silty sand & gravel.
Stiff, gray silty clay.
Dense, gray, fine to coarse sand w/ some silt.
Hard, gray & yellow silty clay w/ shell fragments.
Stiff, tan & gray silt.

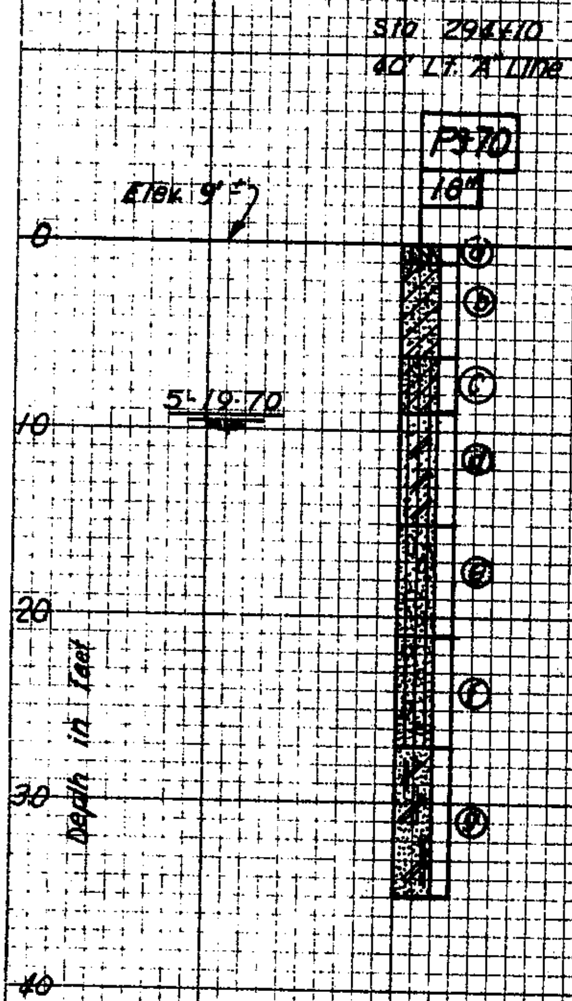


Sta. 278+50
50' Rt. "A" Line
P1-77
18"
El. 7±?
0
10
20
30
40
50
60
Depth in feet
a
b
c
d
e
14-60-20-16
f
g
Damp, tan, silty clay & shale frags.
Moist, firm, dark brown to lt. green silty clay.
Very moist, tan, silty gravelly sand.
Saturated, tan, silty, clayey, sandy gravel.
Moist, stiff, mottled green-tan-brown, silty clay.
Moist, stiff, blue-green clay.
46


Sta. 285+80
7' Lt. A Line
B-8-70
2.5"
Elev. 8.2±
7-28-70
0
20
40
60
80
100
120
Depth in Feet
2"
1"
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
Hard, black, organic clay.
Soft to stiff, lt. brown clay.
Loose to dense, brown, fine to medium, clayey sand.
Stiff lt. brown, silty clay.
Stiff to hard, blue, silty clay.
Hard, blue sandy clay.
Dense gray medium to coarse sand.
Hard, sandy clay.
Dense gray-brown, fine to medium sand.
Loose to dense, gray clayey sand w/ lenses of coarse sand.
Hard, brown, sandy silty clay.
Stiff to hard, greenish-blue silty clay.
47



Sta. 287+25
100' Lt. "A" Line
B2-71
2"
Elev. 7±
2-10-71
U=0,1,2
50
63
U=4,0,1
Depth in feet
U=0,1,1
U=0,1,2
U=0,1,1
120
105
220
0
10
20
30
40
50
60
70
2"
1"
0
① 126
② 124 105 4-27-33-36 q_u=1.54
③ 126
④ 118 123 0-7-66-27 q_u=1.45
⑤ 128
⑥ 124
⑦ 126 123 0-20-42-38 q_u=2.36
⑧ 121
⑨ 124 121 5-56-29-10
⑩ 120
⑪ 117 122 0-6-49-45
⑫ 122 124 0-44-43-13
⑬ 118 128 0-8-49-43
⑭ 121 122 0-62-24-14
⑮ 126 121 1-15-52-32
⑯ 126
⑰ 122 122 2-14-59-27
⑱ 123
⑲ 124 124 4-76-14-6
⑳
Moist, firm, gray & tan clay.
Moist, firm, tan, silty clay w/ some peat.
Moist, stiff, tan, silty clay interbedded w/ thin layers of blue, fine sand.
Sand
Silty Clay
Sand
Silty Clay
Sand
Silty Clay
Moist, stiff, gray clay
Moist, dense, gray sand w/ some clay binder.
48



Dry, brown, silty, sandy topsoil.

Moist, soft, black, sandy clay w/ flecks of peat.

Moist, soft lt. brown, clayey, silty sand.

Wet, soft lt. brown silty sand w/ clay binder.

Moist, firm lt. brown, silty sand w/ rock fragments.

Moist, firm to friable, silty sand with rock fragments.

Moist, soft lt. brown, sand with traces of silt and clay.

49

APPENDIX B

AGENCIES CONTACTED DURING GROUND WATER RECONNAISSANCE SURVEY

APPENDIX B

AGENCIES CONTACTED DURING GROUND WATER RECONNAISSANCE SURVEY

- Department of Public Works, City of Richmond
- Richmond Unified School District
- University of California, Richmond Field Station
- Contra Costa County Health Department, Richmond Division
- U. S. General Services Division, Richmond Office
- Pacific Gas and Electric Company, Richmond and Oakland
- City of Richmond Planning Department
- Allied Propane Co., Richmond
- Blair Excavators, Inc., Richmond
- Buildings Regulations Division, City of Richmond
- Pacific Cryogenics, Richmond
- Port Director, City of Richmond
- City of El Cerrito Planning Department
- Department of Community Development, City of San Pablo
- Sanitary District, City of San Pablo
- Contra Costa County Planning Department, Martinez
- East Bay Municipal Utility District, Oakland
- San Francisco Bay Region Office, Regional Water Quality Control Board, Oakland
- Sanitary Engineering Section, State Department of Health, Berkeley
- Department of Water Resources, Central District Office, Sacramento

- Sacramento Municipal Utility District
- Bureau of Reclamation, Sacramento
- U. S. Geological Survey, Menlo Park
- U. S. Geological Survey, San Francisco
- Bureau of Reclamation, Denver

APPENDIX C

PRIVATE WELLS IN PROJECT VICINITY

APPENDIX C

LIST OF PRIVATE WELLS IN PROJECT VICINITY

Address	Operational	Address	Operational
131 S. 2nd St.	No	635 S. 29th St.	
231 S. 5th St.	Yes	608 S. 30th St.	-
229 S. 9th St.	-	654 S. 30th St.	No
148 S. 11th St.	-	Foot of S. 32nd St.	-
203 S. 13th St.	No	2016 Cutting Blvd.	No
141 S. 14th St.	Yes	719 Florida Ave.	-
728 S. 14th St.	No	901 Florida Ave.	No
332 S. 15th St.	-	2924 Johnson	No
316 S. 16th St.	No	718 Main	No
725 S. 16th St.	No	825 Maine	No
401 S. 18th St.	-	1900 Maine	-
338 S. 19th St.	-	2029 Maine	No
425 S. 19th St.	Yes	326 Ohio	-
433 S. 19th St.	-	1933 Ohio	-
200 S. 20th St.	No	1420 Potrero	-
331 S. 20th St.	No	1910 Potrero	-
425 S. 20th St.	-	614 Virginia	No
253 S. 21st St.	No	1829 Virginia	Yes
167 S. 22nd St.	-	1314 Wright	Yes
315 S. 23rd St.	-	1431 Wright	-
416 S. 23rd St.	No	555 S. 30th St.	No
546 S. 29th St.	Yes	3033 Florida	-
609 S. 29th St.	No		

Address	Operational	Address	Operational
4310 Florida	-	153 S. 45th St.	-
2621 Maine	-	220 S. 45th St.	-
2625 Maine	No	729 S. 45th St.	-
2508 Maine	-	832 S. 45th St.	No
3519 Wall	No	890 S. 45th St.	-
3815 Wall	-	900 S. 45th St.	-
3827 Wall	No	919 S. 45th St.	-
4604 Wall	-	923 S. 45th St.	-
4914 Wall	-	241 S. 46th St.	-
1339 Merced	-	700 S. 46th St.	-
601 S. 31st St.	No	736 S. 46th St.	-
381 S. 34th St.	-	701 S. 47th St.	No
384 S. 34th St.	No	1333 S. 47th St.	-
400 S. 34th St.	No	4321 Potrero	-
151 S. 37th St.	-	4827 Potrero	No
213 S. 37th St.	-	3812 Ohio *	Yes
355 S. 37th St.	-	4127 Ohio	No
380 S. 37th St.	Yes	4849 Cypress**	Yes
350 S. 38th St.	-	425 Stege	-
403 S. 38th St.	No	317 S. 49th St.	-
419 S. 38th St.	No	345 S. 49th St.	-
421 S. 38th St.	-	740 S. 49th St.	-
214 S. 41st St.	-		
743 S. 41st St.	-		
171 S. 42nd St.	-		

* Used for drinking until about 3 years ago.
** Being used for drinking.

NOTE:

<u>Source</u>:

(1) Public Health Dept. of Contra Costa Co.

(2) Richmond Civil Defense and Disaster Office, Dist. 1

APPENDIX D

CATHODIC PROTECTION WELLS

[illegible]

APPENDIX D

CATHODIC PROTECTION WELLS

The proposed Hoffman Freeway alignment in the City of Richmond will traverse in an industrial area bordering on San Francisco Bay. Corrosion protection of buried metal structures using electrical current (cathodic protection) is commonly practiced in the area. Users of such systems include Pacific Gas and Electric Company, Stauffer Chemical Company, Standard Oil Refinery, City of Richmond, East Bay Municipal Utilities District, and numerous small, independent companies. Moisture changes can affect the resistivity of the soil, a basic design parameter of these systems.

Ground Water Reconnaissance Survey

During the course of the ground water reconnaissance survey, a few well driller's logs were identified as being logs for bore holes drilled for installing cathodic protection "deep well anodes". Such well installations are commonly used for corrosion protection of buried metal structures by use of electrical current. It has been determined that cathodic protection systems have been used in the Richmond area for about ten years. Additional ones will be added with new construction in the near future.

Source of Information

Mr. Owen Uhlmeyer, Chief, Corrosion Control, DWR, Sacramento, was contacted by telephone, and he furnished information concerning the installation and use of cathodic protection systems, i.e., deep well anodes.

Several other corrosion engineers mentioned earlier in the report were contacted for their opinions as to whether construction of the semi-depressed section would interfere with cathodic protection systems currently in use. It was generally agreed that depressing the ground water table approximately 30 feet may affect the operating efficiency of some of these systems depending upon type and location. Whether there are any of these systems within the proposed excavation limits and/or within the areas influenced by ground water drawdown has not been determined.

Types of Cathodic Protection Wells

Section 13711 of the California Water Code defines a Cathodic Protection Well as:

> "Any artificial excavation in excess of 50 feet constructed by any method for the purpose of installing equipment or facilities for the protection electrically of metallic equipment in contact with the ground, commonly referred to as cathodic protection."

There are two types of cathodic protection wells, impressed current and sacrificial anode. Both may require deep, large diameter drilled wells. The impressed current type uses more current for operation which is usually supplied by the local utility company. The sacrificial anode requires very low current, and is self-generating. If either of these types of wells are to be abandoned, the California Water Code prescribes the procedure to be followed. It should be emphasized that these are not water producing wells.

Known Locations

The "Joint Committee for Protection of Underground Structures in Alameda and Contra Costa Counties" maintains a listing of cathodic protection systems. Mr. Francis R. Shoemaker, P.E., Bay Toll Authority, is the Secretary of the Committee. A map provided by Mr. Arnie Westerback of East Bay Municipal Utility District is attached showning locations of some of these cathodic protection installations in the vicinity of the Hoffman Freeway alignment. Mr. G. Dowd, also of East Bay Municipal Utility District, furnished a list of cathodic protection rectifiers in Contra Costa County. From this list, 6 rectifiers in the project vicinity were chosen and details pertaining to them are tabulated in Table D-1. It should be emphasized that this map (Figure D-1) and list are incomplete and were not verified by Caltrans personnel.

The Assistant Port Director, City of Richmond, Mr. Sal N. Bose, P.E., stated during interview that the City uses cathodic protection on sheet piling in port facilities. He expressed his opinion that due to the distance from the Hoffman Freeway alignment, there would be no interference with those systems when the ground water is depressed.

Effects of Dewatering

With the limited information available at this time relative to existing cathodic protection installations in the area of the proposed depressed section, it is not possible to fully assess the effects of dewatering on these systems. In some areas dewatering may enhance the efficiency of such systems.

The National Association of Corrosion Engineers Basic Corrosion Course notes(22), states that in tidal areas

TABLE D-1, DETAILS OF CATHODIC PROTECTION RECTIFIERS IN PROJECT VICINITY

NUMBER IN FIGURE	JOINT COMMITTEE NUMBER	OWNER	APPROXIMATE LOCATION	CAPACITY A	CAPACITY V	PROTECTED STRUCTURE
1	214	PT&T	Stege Ave & Cutting Blvd., Richmond	24	20	Telephone Cables
2	216	SPPL	Castro St. near Standard Oil Refinery	50	20	Pump Sta. Well Coated Gathering Lines
3	323	PT&T	S/S Potrero Ave. W/O 45th St.	12	12	Toll and Exchange Cable
4	324	PG&E	S/S Potrero Ave. W/O 54th St.	18	16	Toll and Exchange Cable
5	1164	PG&E	40th St and Nevin Ave.	5	20	Gas Dist.
6	1165	PG&E	Florida Ave W/O 17th St.	5	20	Gas Dist.

particularly, the resistivity range for water in the soil acting as the electrolyte varies greatly and is subject to change. This complicates the cathodic protection application. If the natural parameters upon which a cathodic protection design is based do not remain constant, then resistance of the anode in the electrolyte increases and the structure may be affected and cause current requirements to vary.

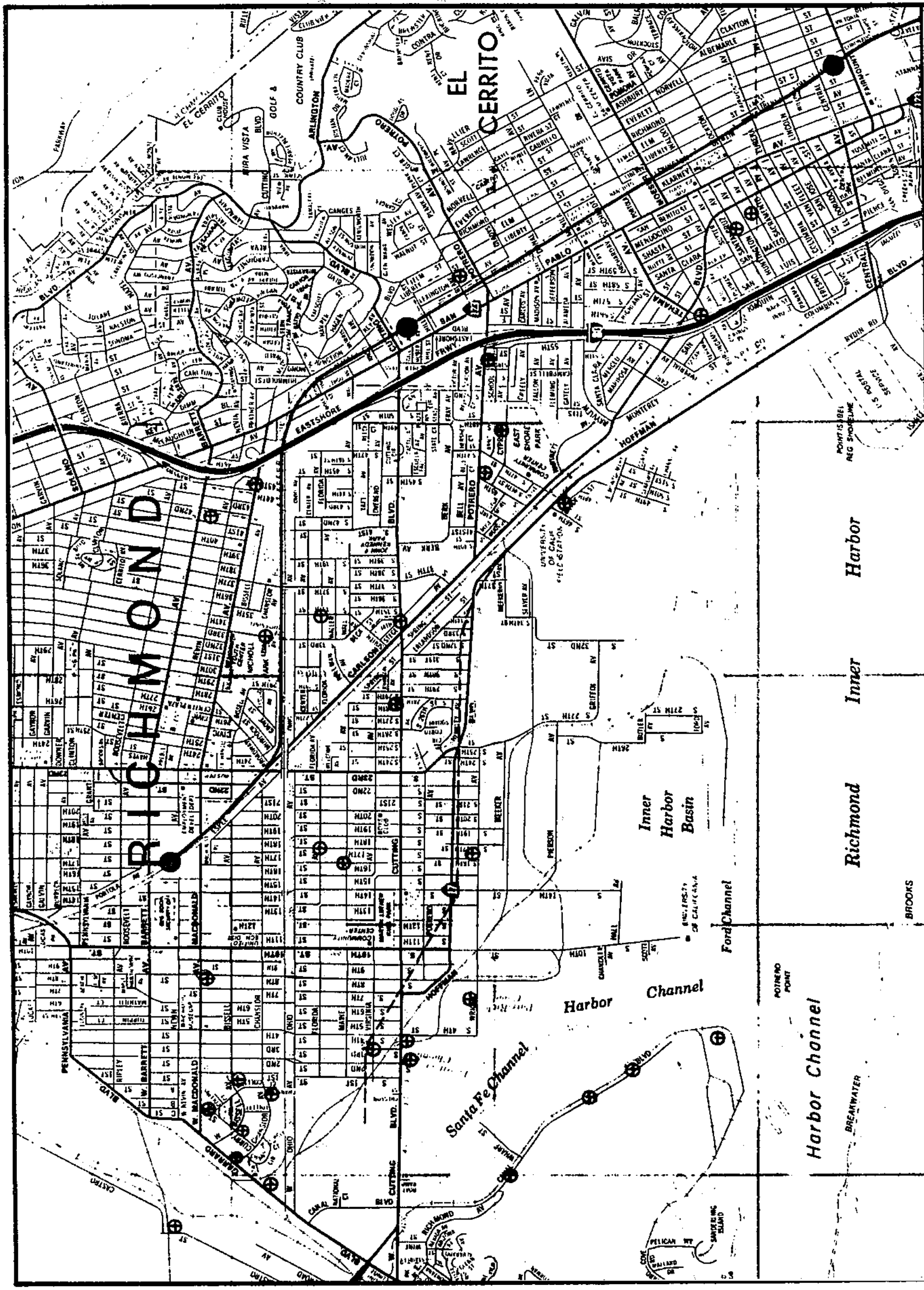
In soils which contain sulfate ions the magnesium anode operates very well when it is buried below ground water table due to good drainage into the anode. When ground water is depressed and if sulfates and chlorides are absent the magnesium anode tends to polarize and the soil tends to display high resistivity. The anode efficiency is then reduced by the low current density achieved.

Effects of Sea Water Intrusion

It is probable that sea water intrusion would not adversely affect cathodic protection deep well systems since magnesium anodes operate at peak performance in sea water. Sea water introduces an environment which has high chloride content and this contributes to high performance. Cathodic protection systems are designed after detailed, careful study of the environment in which they are placed. Monitoring facilities are an integral part of these systems and regular checks are made to detect changes.

Conclusions and Recommendations

Cathodic protection systems utilizing deep well anodes are prevalent in the vicinity of the proposed Hoffman semi-



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LOCATIONS OF CATHODIC PROTECTION WELLS

Figure D-1

depressed section and the adjacent areas which will be affected by the ground water drawdown. The exact location of many of these systems have not been verified by Caltrans personnel to date. Since there is a possibility that the proposed drawdown would affect these systems, we recommend a survey and cataloging of both drilled water wells and deep well anodes prior to construction. We further recommend soil resistivity measurements at designated sites in the six design study areas comprising the semi-depressed portion of the Hoffman Freeway.

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[illegible]