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GROUNDWATER INVESTIGATIONS & CANAL SEEPAGE STUDIES

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Issued under grant from Special Research Project 102 in cooperation with the Soil and Water Conservation Research Division, Agricultural Research Service, USDA and Region 1 Bureau of Reclamation, U.S. Deaprtment of the Interior.

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GROUNDWATER INVESTIGATIONS AND CANAL SEEPAGE STUDIES

by

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U. S. Department of Agriculture

In cooperation with
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ABSTRACT

Progress in 1966 on Special Research Project 102 of the University of Idaho Engineering Experiment Station in cooperation with the Bureau of Reclamation and the Agricultural Research Service, ARS, is presented. Average seepage rates from 86 seepage meter tests with the ARS meter are compared with ponding rates and inflow-outflow loss measurements. The average seepage meter rates are 23 percent higher than ponded rates. A method for determining the required number of seepage meter tests is outlined. Seasonal variation of soil moisture tension in the soil prism beneath an operating canal was measured, using two methods of tensiometer installation and readout. Increases in soil moisture tension from 0 to 5.5 feet of water over the irrigation season is indicative of the gradual sealing of the canal bottom and corresponding decreases in seepage rates. Field and laboratory conductivity measurements indicate the impeding layer in the canal bottom has a saturated hydraulic conductivity of about 1/50 that of the natural soil beneath the layer at the end of the irrigation season. A new method for securing and testing undisturbed soil cores using shrinkable plastic tubing is outlined. Good agreement between field estimates of the conductivity of the impeding layer and laboratory tests was achieved.

DESCRIPTORS -- Seepage/ canal seepage/ seepage losses/ meters/ ponding tests/ unsaturated flow/ tensiometers/ permeability/ hydraulic conductivity/ hydraulic gradients/ lower cost canal linings.

IDENTIFIERS -- University of Idaho/ seepage meters/ inflow-outflow.

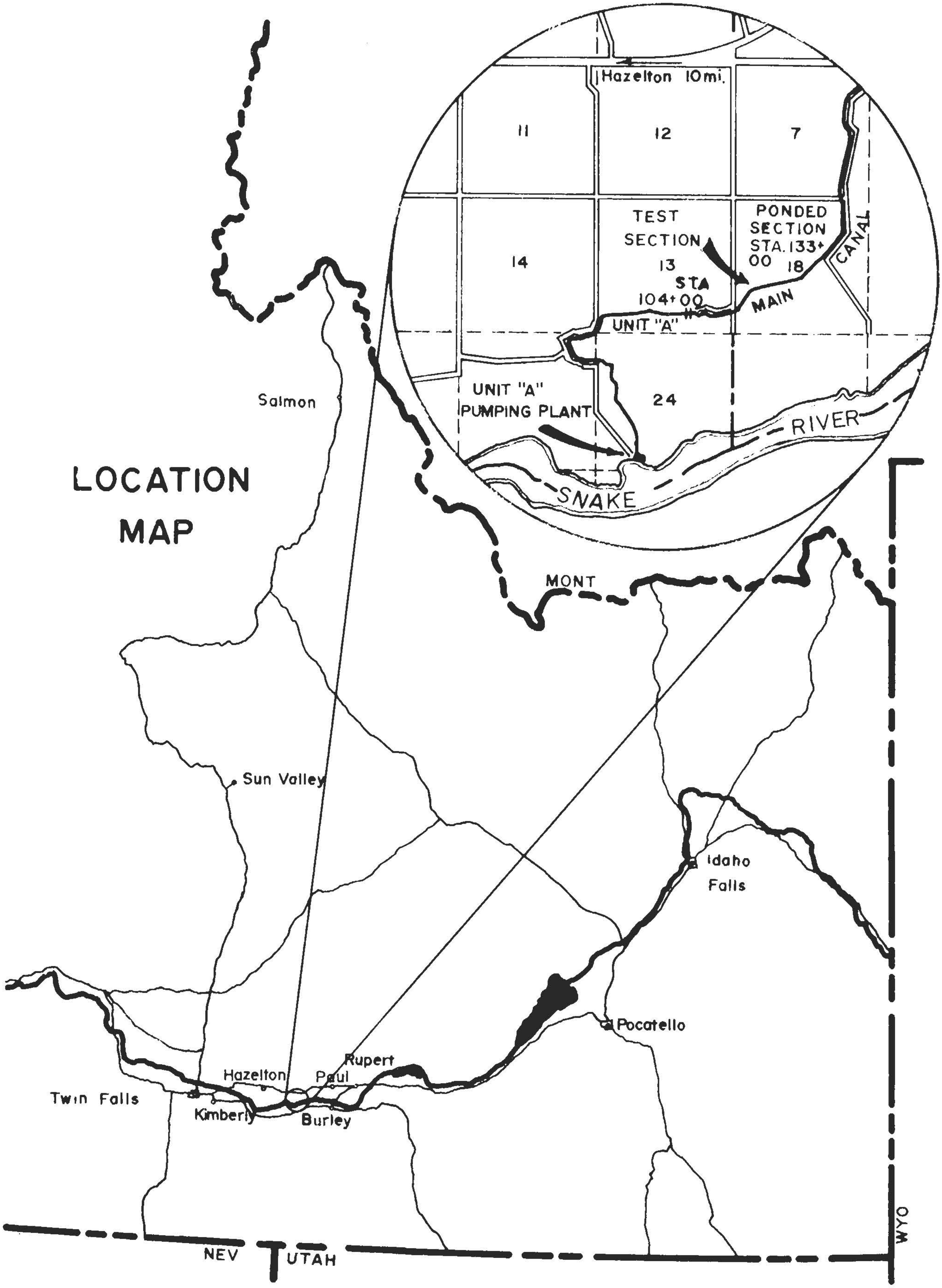
INTRODUCTION

This progress report includes information on field and laboratory research programs pursued during 1966 by the University of Idaho Engineering Experiment Station in the field of canal seepage and groundwater. The sutdies outlined are being conducted as a joint effort by the University and the Agricultural Research Service and constitute fulfillment of the obligations of the Engineering Experiment Station for the second year of a three-year research contract with the U. S. Bureau of Reclamation under the Lower Cost Canal Lining Program. The field investigations were carried out on the Unit 'A' Main Canal (Northside Pumping Canal) and laterals of the A & B Irrigation District near Paul, Idaho. A map showing the locations of the field test sections is shown on page 3. Laboratory studies were conducted at the Snake River Conservation Research Center of the Agricultural Research Service near Kimberly, Idaho.

Overall objectives of the project are to investigate the seepage from canals and reservoirs to include the mechanics of seepage flow from canals, the processes by which seepage water reaches local water tables, and the effect, either beneficial or detrimental, on the water table. An investigation as broad as this involves first of all the study of methods of measuring seepage with the ultimate aim of developing a measurement system which is sufficiently accurate, economical, and fast. Secondly, the determination of water movement to the groundwater table and in the groundwater flow involves both the use of traditional methods and devices and the investigation of new techniques.

The U. S. Bureau of Reclamation furnished material and labor for ponding tests and performed the inflow-outflow loss measurements on the Northside Pumping Canal. Personnel of the A & B Irrigation District assisted materially in these studies and their cooperation is appreciated.

Mr. Gary Clark and Mr. Bruce Wojcik assisted in the field and laboratory studies and Mr. C. D. Carpenter, Agricultural Research Service, assisted in the field experiments.



LOCATION MAP
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MAIN
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Idaho Falls
Pocatello
Rupert
Hazelton
Paul
Twin Falls
Kimberly
Burley
NEV
UTAH
WYO

PONDING TESTS
NORTHSIDE PUMPING CANAL

PROCEDURE

Ponding tests to evaluate seepage losses in a section of the Northside Pumping Canal were performed in October, 1966. These tests were performed in conjunction with Region 1, Bureau of Reclamation in their investigation of the Use of Water on Federal Irrigation Projects, Water Use Research Study. In the determination of farm and project irrigation efficiencies for this study it is necessary to evaluate operating losses from canals and laterals and the 1966 ponding tests are part of an attempt to determine losses from the 4.5 mile long Northside Pumping Canal.

A one-mile reach of canal was chosen immediately downstream from the location of ponding tests performed in the fall of 1965.(1) The ponded reach corresponds approximately to canal design stations 139+20 to 206+80. In order to have constructed earthen dikes as used in the 1965 ponding tests, it would have been necessary to haul the earth about one mile and it was therefore decided to attempt the use of wooden bulkheads for dike construction. To facilitate handling and for reuse in future tests, the bulkheads were constructed in sections with 3/4 inch marine plywood and 2 X 6 studs with the sections being bolted together during installation, Figure 1A. Five or more 4" X 4" timber braces were used on the downstream side of each bulkhead and the bulkhead was covered with 8 mil. thick polyethelene plastic, Figure 1B. A twelve foot wide overflow section allowed a flow of about 15 cfs to pass safely over the crest to fill the ponds. Water surface elevations were measured at both ends of the pond with water stage recorders and hook-gages mounted in 12 inch diameter stilling wells.

A. Construction of plywood panel bulkhead, looking upstream.

B. Bulkhead with plastic covering and bracing during filling of pond.
FIGURE 1 BULKHEAD CONSTRUCTION AND OPERATION
PONDING TESTS - NORTHSIDE PUMPING CANAL

Because of difficulty in regulating the canal discharge, the plastic on the canal bank at the upstream dike was overtopped and allowed piping to occur around one side of the bulkhead. Erosion of the soil around the side of the bulkhead caused the plastic to tear and further erosion to take place. An effective seal was achieved on the upstream bulkhead when the water surface dropped below the tear in the plastic. Because of the bulkhead failure and the impossibility of obtaining water at the late date in the irrigation season, a partial ponding test, about 86 hours duration, was achieved on the upstream pond only.

RESULTS

The ponded section of the canal had been dry or flowing at a shallow depth for 14 days prior to the ponding tests while seepage meter tests were run and bulkheads constructed. Seepage loss rates measured for the single pond filling are higher than normal loss rates because the bank storage was not satisfied. Figure 2 shows the wetted surface area and pond volume for each elevation in the ponded reach, and measured seepage rates corresponding to water surface elevations are shown in Figure 3. All wetted surface, volume, and seepage rate calculations were performed by digital computer. The operating depth for the canal in this reach is 4.65 feet, however, the maximum water depth for which the ponded rate was measured is 2.65 feet. The seepage rate for a depth of 2.65 feet was .73 cubic feet feet per square foot per day (cfd.). An extrapolation of the curve in Figure 3 to the elevation of operating depth shows an estimated seepage rate in the neighborhood of .82 cfd. for the first pond filling. Similar curves from ponding tests performed in 1965 on adjacent portions of the canal show that the ponded seepage rate at operating depth on the second filling is about

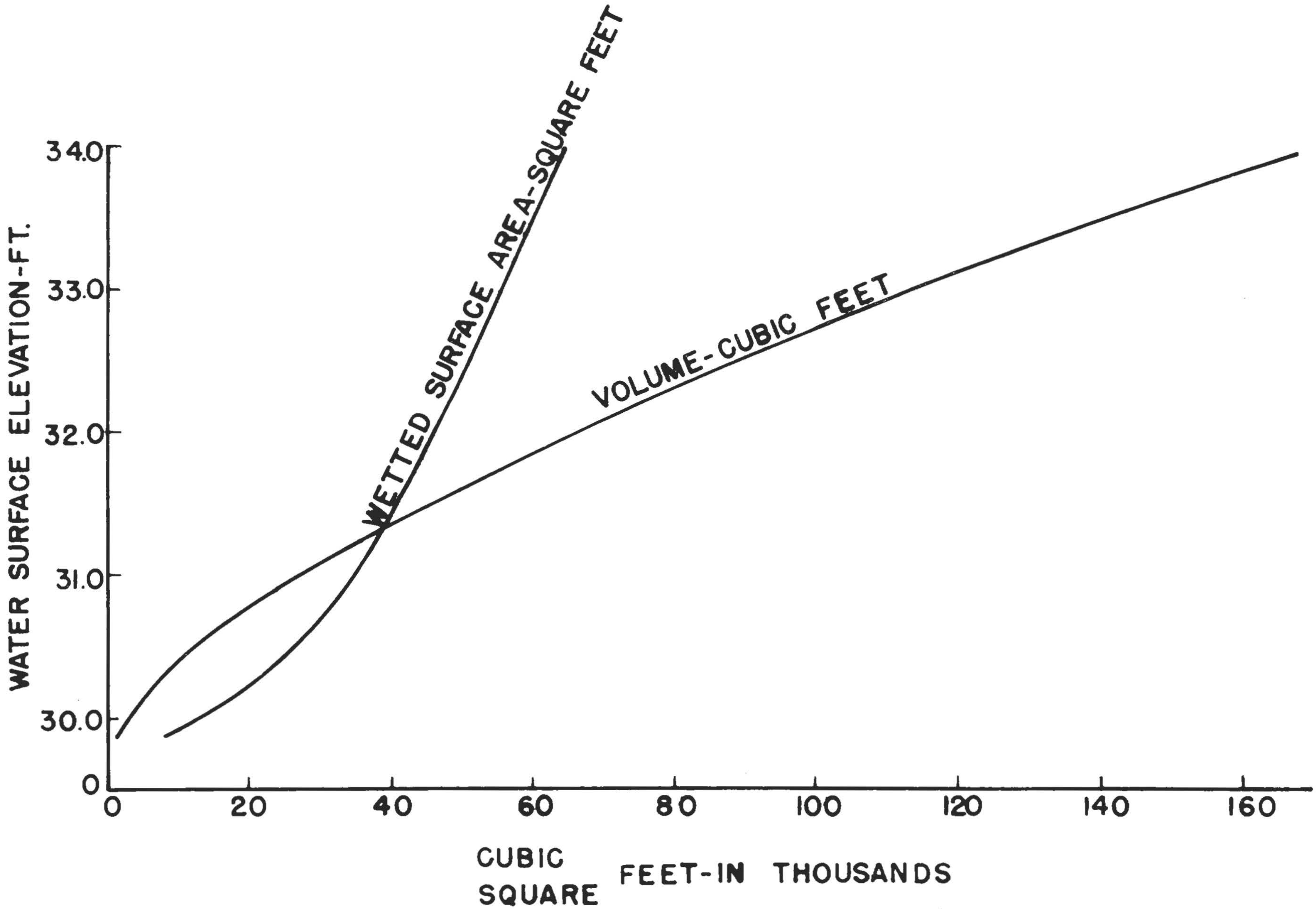


FIGURE 2 VOLUME AND WETTED AREA – UPPER TEST SECTION, NORTHSIDE PUMPING CANAL, NOVEMBER 1966

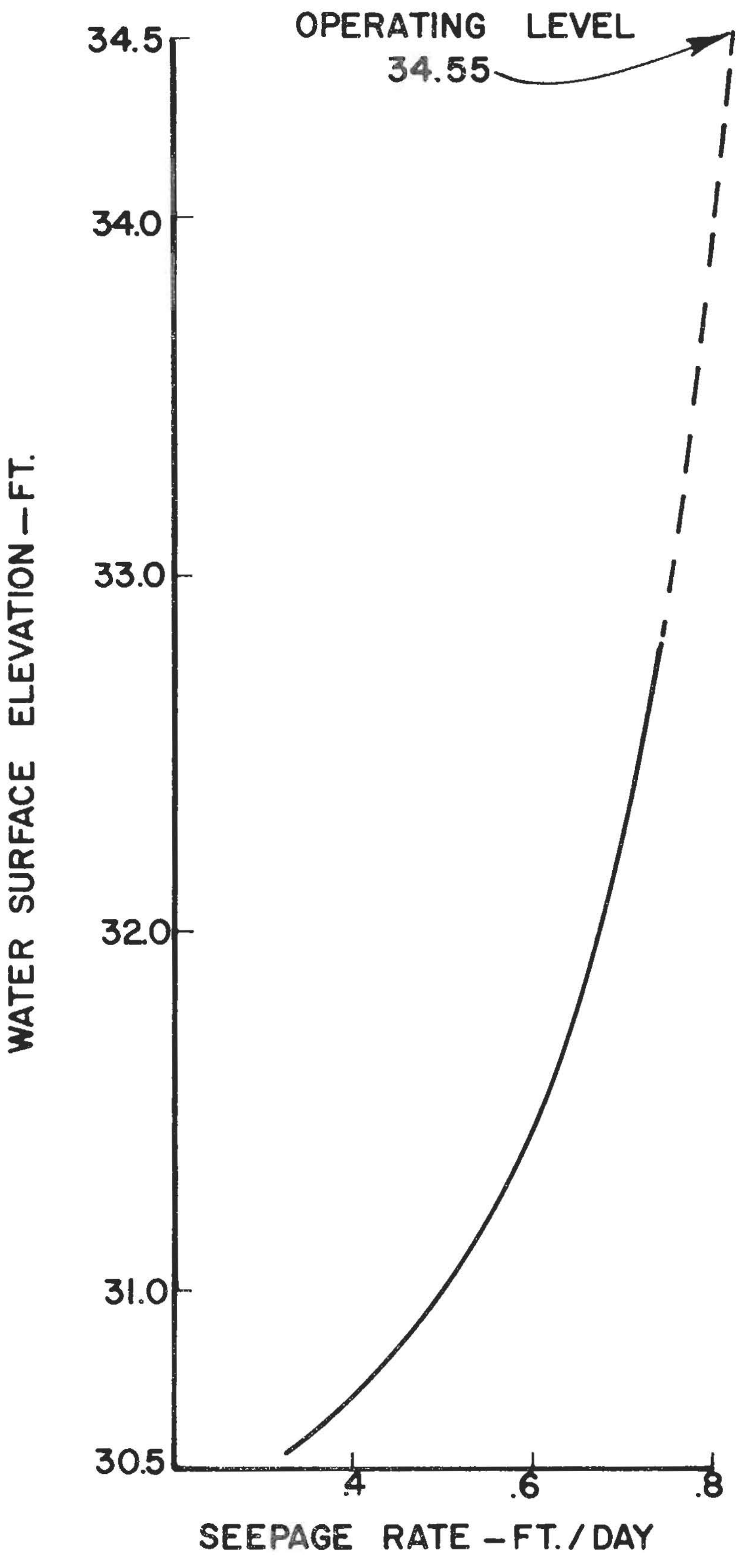


FIGURE 3 SEEPAGE VS. WATER SURFACE ELEVATION
NORTHSIDE PUMPING CANAL NOVEMBER 1966

92% of that for the first filling. For this same percentage, the seepage rate at operating depth for the 1966 ponded reach is about .75 cfd.

DISCUSSION

The seepage rate of .75 cfd is larger than the average value of .60 cfd measured in the one-mile ponded reach in the fall of 1965. Prior to the 1966 tests, Irrigation District personnel had 'sloped' the canal banks to remove the berm and 2-4 inches of the soil on the canal banks. The bottom of the canal was not disturbed except for the soil which rolled down during the 'sloping' operation. The 'sloping' operation did remove from the banks the thin impeding sediment layer which had developed and could have caused the increased seepage rate to occur. The curve in Figure 3 is a characteristic curve for this canal, and shows that the increase in seepage rate with water depth is quite small. The cross-section in Figure 8 is typical of this canal and shows the intersection of a consolidated silt layer with the canal bank. The consolidated layer was present in the entire ponded reach and because it has a lower hydraulic conductivity, could have caused the seepage vs. elevation curve to have the shape as shown in Figure 3.

SEEPAGE METER TESTS

PROCEDURE

Further tests to evaluate the effectiveness of seepage meters for canal seepage measurement were made in October 1966. Using two variable head seepage meters developed by the Agricultural Research Service[2] a total of 60 tests were run in the one-half mile section of the Northside Pumping Canal on which a ponding test was subsequently performed. The 60 measurements were run using the procedure followed in

1965.[1,3] Ten tests, in groups of five across the canal, were run at each of six stations about 400 feet apart. The centerline water depth during the tests averaged 18 inches with a flow velocity of about .75 feet per second.

Twenty-six seepage meter measurements were performed on the one-half mile section immediately downstream of the ponded reach. The tests were made in groups of about seven at four stations, 800 feet apart. The average centerline water depth was 22 inches with a flow velocity of about .6 feet per second.

RESULTS

Table 1 is a summary of test results for the 1966 tests in the upper test section. The measured seepage rates varied from .024 cfd to 2.790 cfd with an average seepage rate of 0.692 cfd. Standard deviation of the 60 tests was .869 cfd. Water depths at meter locations ranged from 6 inches to 25 inches and averaged 13.3 inches.

Measured seepage rates in the lower test as shown in Table 2 varied from .038 cfd to 2.491 cfd and averaged .319 cfd. Standard deviation of the 26 tests was .518 cfd. Water depths at meter locations varied from 7 inches to 25 inches and averaged 16.5 inches.

Computed seepage was determined using a digital computer program developed for the 1965 series of tests. The program, and information for use is included in Appendix 1.

DISCUSSION

Results of the 86 seepage meter tests indicate that the seepage rate for the upper test section is about twice the rate for the lower test section. Two reasons may account for the difference in rates. First, in the downstream section, about two-thirds of the 1/2 mile reach

TABLE 1
SEEPAGE METER TESTS
ARS SEEPAGE METER

Northside Pumping Canal
Minidoka Project-Idaho

October, 1966

Upper Test Section

Station	Test No.	Penetration Inches d	Water Depth Inches h_w	Balanced Head, In. H_b	H_b/h_w	Seepage ft/day I_s	*** P In.	** R_a Hours
149+00	1-1	.75	7.0	20.59	2.94	2.010	-13.59	20.48
	1-2	.25	7.0	35.63	5.09	1.535	-28.63	46.42
	1-3	.75	18.0	25.00	1.39	.439	- 7.00	113.89
	1-4	.25	18.0	6.30	.35	2.372	-	-
	1-5	1.25	16.0	30.12	1.88	.443	-14.12	135.97
	1-6	1.00	8.0	8.86	1.11	.733	- 0.86	24.17
	1-7	.75	17.5	25.20	1.44	.659	- 7.70	76.47
	1-8	.50	6.5	12.60	1.94	.422	- 6.10	59.71
	1-9	.50	18.0	20.67	1.15	2.349	- 2.67	17.60
	1-10	.75	15.0	5.24	.35	.170	-	-
Total		6.75	131.0			11.132	-80.66	494.73
Average		.675	13.10			1.113	-10.08	61.84
145+00	2-1	.75	25.0	30.31	1.21	.176	- 5.31	344.49
	2-2	1.25	22.0	20.08	.91	.141	+ 1.92	284.70
	2-3	1.50	21.0	15.75	.75	.437	-	-
	2-4	1.75	12.0	24.02	2.00	.149	-12.02	315.27
	2-6	1.25	10.0	31.50	3.15	.453	-21.50	139.06
	2-7	1.25	7.5	11.81	1.57	.869	- 4.31	20.54
	2-8	1.00	19.0	-	-	.062	-	-
	2-9	1.00	12.0	3.94	.33	.024	-	-
Total		9.75	128.5			2.311	-41.22	1104.06
Average		1.22	16.06			.288	8.24	220.81
141+00	3-1	.75	6.0	3.15	.52	.387	-	401.78
	3-2	.50	13.0	15.67	1.21	.078	- 2.67	79.68
	3-3	.75	16.0	20.08	1.25	.504	- 4.08	47.63
	3-4	.50	19.0	19.69	1.04	1.653	- 0.69	494.50
	3-5	.75	17.0	21.26	1.25	.092	- 4.26	10.98
	3-6	1.00	8.0	12.80	1.60	2.330	- 4.80	87.95
	3-7	.50	14.0	16.54	1.18	.376	- 2.54	588.12
	3-8	.50	18.0	23.82	1.32	.081	- 5.82	432.01
	3-9	-	18.0	15.98	.89	.074	+ 2.02	15.71
	3-10	1.00	10.0	14.17	1.42	1.804	- 4.17	-
Total		6.25	139.0			7.379	-27.00	2158.36
Average		.694	13.90			.738	- 3.00	239.81

** R_a = hydraulic impedance of slowly permeable layer.

*** P = soil water pressure beneath slowly permeable layer - inches of water.

TABLE 1 (cont.)

Station	Test No.	Penetration Inches d	Water Depth Inches h_w	Balanced Head, In. H_b	H_b/h_w	Seepage ft/day I_s	*** P In.	** R_a Hours
137+00	4-1	1.00	6.0	7.87	1.31	2.790	- 1.87	5.64
	4-2	.75	7.0	11.10	1.58	2.305	- 4.10	9.63
	4-3	.75	21.0	23.62	1.12	.120	- 2.62	393.70
	4-4	.50	21.5	9.06	.42	.042	-	-
	4-5	.75	14.0	9.84	.70	.071	-	-
	4-6	1.00	7.5	14.96	1.99	2.870	- 7.46	10.43
	4-7	.50	9.0	21.30	2.37	.417	-12.30	102.15
	4-8	1.00	19.0	18.31	.96	.105	+ 0.69	348.70
	4-9	.50	17.0	28.82	1.70	.204	-11.82	282.53
	4-10	1.25	19.0	24.80	1.30	.078	- 5.80	635.97
Total		8.00	141.0			9.002	-52.74	1799.17
Average		.80	14.10			.900	- 5.86	199.91
133+50	5-1	1.00	16.5	16.06	.97	.066	+ 0.44	486.76
	5-2	2.00	6.5	13.98	2.15	1.829	- 7.48	15.28
	5-3	1.00	17.0	29.25	1.72	.262	-12.25	223.30
	5-4	1.50	6.5	6.89	1.05	2.613	- 0.39	5.27
	5-5	1.50	12.5	17.64	1.41	.106	- 5.14	332.79
	5-6A	1.25	11.5	28.94	2.52	.216	-17.44	267.94
	5-6B	1.25	11.5	28.94	2.51	.307	-17.44	188.51
	5-7	1.25	9.0	19.76	2.20	2.541	-10.76	15.55
	5-8	.50	16.0	17.32	1.08	.287	- 1.32	120.71
	5-9	1.38	12.5	18.58	1.49	.243	- 6.08	152.94
	5-10	1.00	16.0	20.08	1.25	.063	- 4.08	637.42
Total		13.63	135.5			8.533	-81.93	2446.47
Average		1.239	12.32			.776	- 7.45	222.10
129+00	6-1	1.13	15.0	21.10	1.41	.101	- 6.10	417.87
	6-2	1.50	7.0	12.52	1.79	.083	- 5.52	301.68
	6-3	.88	14.0	15.87	1.13	.064	- 1.87	495.82
	6-4	.75	18.0	4.92	.27	.072	-	-
	6-5	1.25	6.0	6.26	1.04	.091	- 0.26	137.58
	6-6	.75	15.0	19.69	1.31	.187	- 4.69	210.55
	6-7A	1.25	11.0	13.90	1.26	.332	- 2.90	83.72
	6-7B	.75	11.0	13.78	1.25	.082	- 2.78	336.09
	6-8	-	-	10.47	-	1.310	-	-
	6-9	.50	13.0	18.11	1.39	.628	- 5.11	57.68
	6-10	.25	14.0	2.44	.17	.222	-	-
Total		9.01	124.0			3.172	-29.22	2040.99
Average		.901	12.40			.288	- 3.65	255.12

TABLE 2
SEEPAGE METER TESTS
ARS SEEPAGE METER

Northside Pumping Canal
Minidoka Project-Idaho

October, 1966

Lower Test Section

Station	Test No.	Penetration Inches d	Water Depth Inches h_w	Balanced Head, In. H_b	H_b/h_w	Seepage ft/day I_s	*** P In.	** R_a Hours
153+25	7-1	.75	12.0	19.49	1.62	2.491	- 7.49	15.64
	7-2	1.00	7.0	10.55	1.51	.056	- 3.55	376.83
	7-3	.50	20.0	21.65	1.08	.133	- 1.65	325.62
	7-4	1.00	14.0	11.38	.81	.059	+ 2.62	385.69
	7-5	1.00	21.0	28.15	1.34	.636	+ 7.15	88.52
	7-6	.25	15.0	1.30	.09	.069	-	-
	7-7	.25	19.0	28.35	1.49	.338	- 9.35	167.73
Total		4.75	108.0			3.782	-26.54	1360.00
Average		.68	15.43			.540	- 4.42	226.67
161+00	8-1	.75	9.0	4.13	.45	.100	-	349.08
	8-2	.75	7.5	10.47	1.39	.060	- 2.97	158.08
	8-3	.75	16.5	20.87	1.26	.264	- 4.37	188.30
	8-4	.50	16.5	24.29	1.47	.258	- 7.79	424.78
	8-5	.38	20.5	16.14	.79	.076	+ 4.36	-
	8-6	.25	22.0	5.31	.24	.045	-	964.17
	8-7	.50	22.5	23.62	1.04	.049	- 1.12	-
Total		3.88	114.5			.852	-11.89	2084.4
Average		.554	16.36			.122	- 2.38	416.88
169+00	9-1	1.25	20.0	22.24	1.11	.219	- 2.24	203.14
	9-2	.75	5.5	9.45	1.72	.095	- 3.95	198.92
	9-3	.25	22.0	15.79	.72	.062	-	-
	9-4	.75	22.5	18.11	.80	.143	+ 4.39	253.29
	9-5	1.25	15.0	11.97	.80	.148	+ 3.03	161.74
	9-6	1.75	8.5	16.54	1.95	.705	+ 8.04	46.90
	9-7	.88	21.0	22.83	1.09	.375	- 1.83	121.78
Total		6.88	114.5			1.747	- 8.56	985.77
Average		.983	16.36			.250	- 1.43	164.30
177+00	10-1	.50	25.0	12.52	.50	.063	-	-
	10-2	1.25	7.0	8.66	1.24	1.228	- 1.66	14.10
	10-3	2.13	17.0	25.98	1.53	.408	- 8.98	127.37
	10-5	.75	24.5	-	-	.038	-	-
	10-7	1.00	19.0	32.68	1.72	.166	-13.68	393.70
Total		5.63	92.5			1.903	-24.32	505.17
Average		1.126	18.50			.381	- 8.11	168.39

** R_a = Hydraulic impedance of slowly permeable layer.
*** P Soil water pressure beneath slowly permeable layer - inches of water.

is a compacted fill section in which the fill material is a dark brown to black silt-loam with organic silt layers. The hydraulic conductivity of the fill material is probably less than that of the natural silt-loam soil. Secondly, in the lower section, only one side of the canal had been 'sloped' as indicated in the discussion on ponding tests whereas, both sides had been 'sloped' in the upper test section. The effect of any impeding silt layer on the canal side slopes was eliminated entirely in the upper test section but only on one side slope in the lower test section. The canal bottom was not affected by the sloping operation in either test section.

Table 3 shows the location of seepage meter tests in the canal cross-section and Figure 4 shows the spatial variation of average seepage meter rates in the canal cross-section for the two 1966 test sections and the 1965 test section. The distribution in the 1966 upper test section is contrary to what would normally be expected in that the indicated seepage rates at locations farthest from the canal centerline where the water depth is smallest are considerably higher than rates in the center portion of the cross-section. This variation shows the effect of the sloping process which removed the impeding silt layer from the sides of the canal. The majority of the tests taken at points 6 to 8 from the canal centerline were in areas where the layer had been removed.

In the 1966 lower test section the indicated seepage rate was higher on one side of the canal cross-section and the larger seepage rate is probably a result of the 'sloping' which removed the top layer on one side of the canal only. The variation of seepage rate with location in the 1965 test section is small and is probably close

TABLE 3
DISTRIBUTION OF SEEPAGE METER TESTS IN CANAL CROSS SECTION*
NORTHSIDE PUMPING CANAL
UPPER TEST SECTION, 1966
LOCATION IN CROSS SECTION

	Ft. Left 6.1-8	Ft. Left 4.1-6.0	Ft. Left 2.1-4.0	Ft. Left .1-2.0	Center Line	Ft. Right .1-2.0	Ft. Right 2.1-4.0	Ft. Right 4.1-6.0	Ft. Right 6.1-8.0
	2.010	.437	.439	.176	2.372	.170	.078	1.535	2.330
	.733	.062	.443	.074	2.349	.092	.376	.422	2.790
	.453	1.804	.659	.078	1.653	.081	.120	.141	2.870
	.869	.417	.504	.222	.042	.064	.105	.149	2.541
	.387	.216	.071		.066		.262	.024	
	2.305	.307	.204		.063		.332	.106	
	1.829	.083	.287		.101		.082	.243	
	2.613		.072		.187			.091	
			.628		1.310				
Avg.	1.400	.475	.367	.138	.905	.102	.194	.339	2.633

LOWER TEST SECTION, 1966
LOCATION IN CROSS SECTION

	Ft. Left 6.1-8	Ft. Left 4.1-6.0	Ft. Left 2.1-4.0	Ft. Left .1-2.0	Center Line	Ft. Right .1-2.0	Ft. Right 2.1-4.0	Ft. Right 4.1-6.0	Ft. Right 6.1-8.0
	.060	.056	.059	.375	.636	.076	.133	2.491	.100
	.258	.219	.069		.049	.143	.338	.264	.095
	.148		.045		.063		.038	.408	.705
			.062					.166	1.228
Avg.	.155	.138	.059	.375	.249	.110	.170	.832	.532

TEST SECTION, 1965
LOCATION IN CROSS SECTION

	Ft. Left 6.1-8	Ft. Left 4.1-6.0	Ft. Left 2.1-4.0	Ft. Left .1-2.0	Center Line	Ft. Right .1-2.0	Ft. Right 2.1-4.0	Ft. Right 4.1-6.0	Ft. Right 6.1-8.0
	.59	.66	.57	.55	.28	.57	.44	1.97	.85
	.94	.28	.69	.21	.33	.40	.56	.39	.16
	.48	1.20	1.04	1.12	.76	.72	.90	.34	.14
	.23	.52	.70		1.60	.20	.58	.54	
	1.06	1.04	.64		.46	.96	.54	1.00	
	.77	.95	.25		.24		.94	.27	
		.47	.60		.68			1.82	
		.86	.66		1.01			.43	
		.17	.72		2.64			.38	
		.37	.60		.45			.70	
		.48	.70		.52			.26	
			.90					.49	
			.90						
			.32						
Avg.	.678	.636	.664	.627	.816	.570	.660	.716	.383

*Seepage rates are cubic feet per square foot per day (cfd).

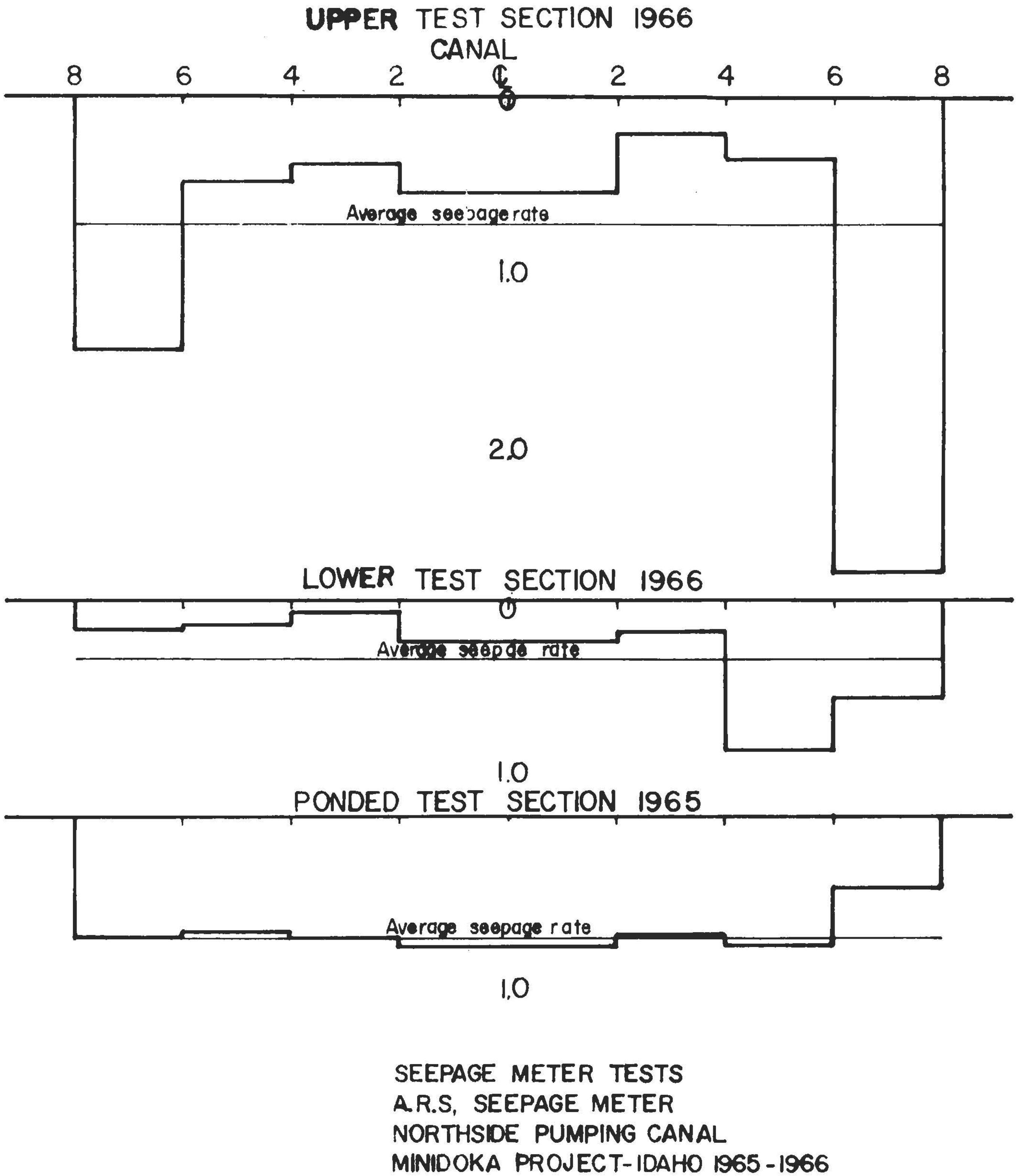
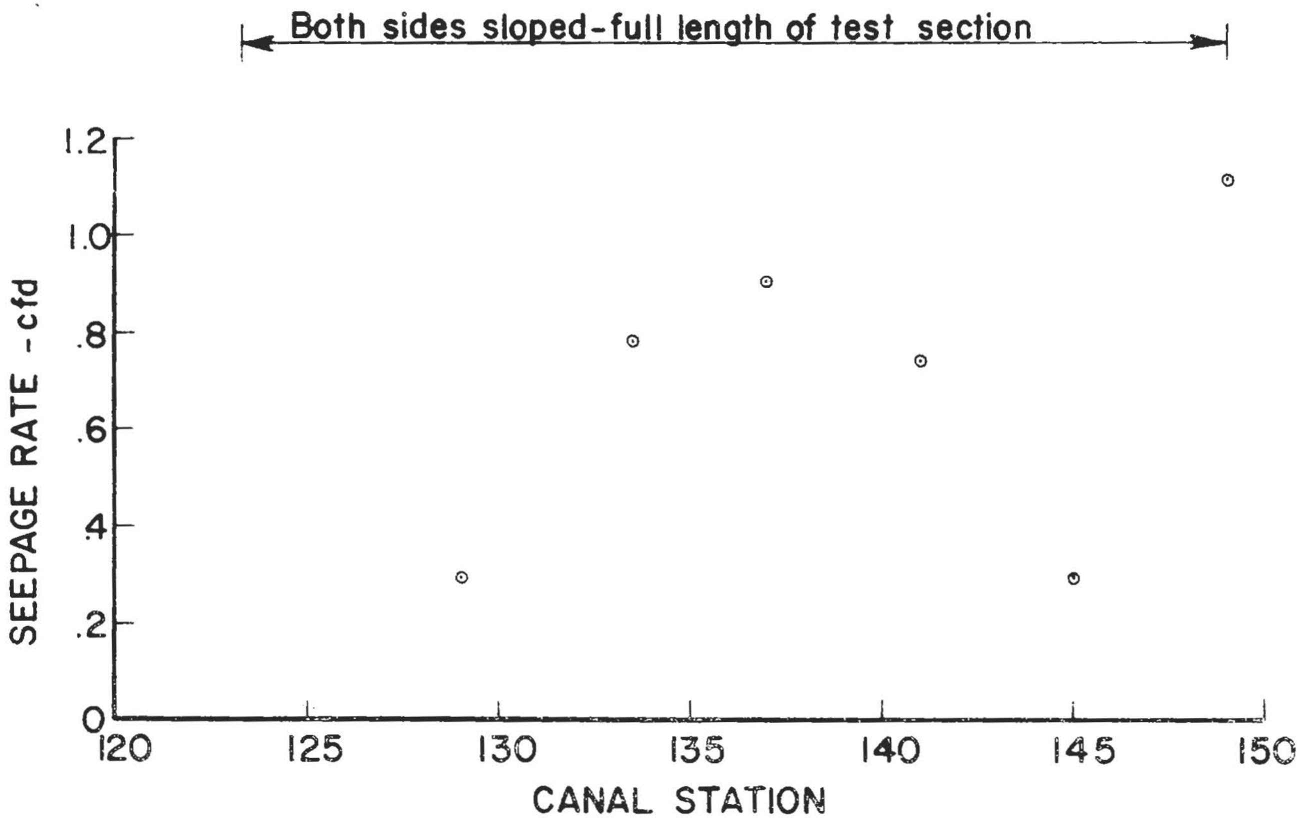


FIGURE 4 VARIATION OF SEEPAGE RATE IN CANAL CROSS SECTION

to the variation to be expected in the canal for undisturbed conditions. It is clear from Table 3 and Figure 4 that the average rates indicated by seepage meter tests on the canal centerline or at short distances right or left may not be the true average rate over the entire wetted perimeter. These variations in average seepage across the canal cross-section clearly show the ability of the seepage meter to delineate areas of relatively high seepage rates laterally within the cross-section.

Figures 5 and 6 show the variation of seepage rate along the length of each test section as indicated by groups of seepage meter tests. Figure 5, for the 1966 upper test section, shows considerable variation in seepage rate throughout the length of section. No reason for this variation is apparent either in visible soil variation or sloping procedure. Both sides of the upper test section were sloped in the same manner and the material in which the canal is built is quite similar throughout the section. In Figure 6 the lower seepage rates downstream of station 160 seem to indicate either the effect of the reduced seepage rate in the fill section or the fact that less sloping work was performed below station 160+75.

In using the ARS seepage meter it is possible to determine the impedance of a slowly permeable layer on the canal bottom.(3) The hydraulic impedance of a slowly permeable layer is defined as the ratio of the thickness of the layer, L_s, to the hydraulic conductivity of the layer, K_s, and has the dimensions of time. In 49 out of 60 tests in the upper test section, the water depth h_w was exceeded by the balanced head H_b.(1) This indicates that the head of water h_w is dissipated in the length of soil equal to the depth of penetration of the meter. Bouwer(3) indicates that if the ratio H_b/h_w is greater than 0.8 then the hydraulic impedance of the layer or of the material to



SEEPAGE METER TEST, A.R.S. SEEPAGE
METER. NORTH SIDE PUMPING CANAL,
MINIDOKA PROJECT - IDAHO
OCTOBER 1966

FIGURE 5 VARIATION OF SEEPAGE RATE IN UPPER TEST SECTION

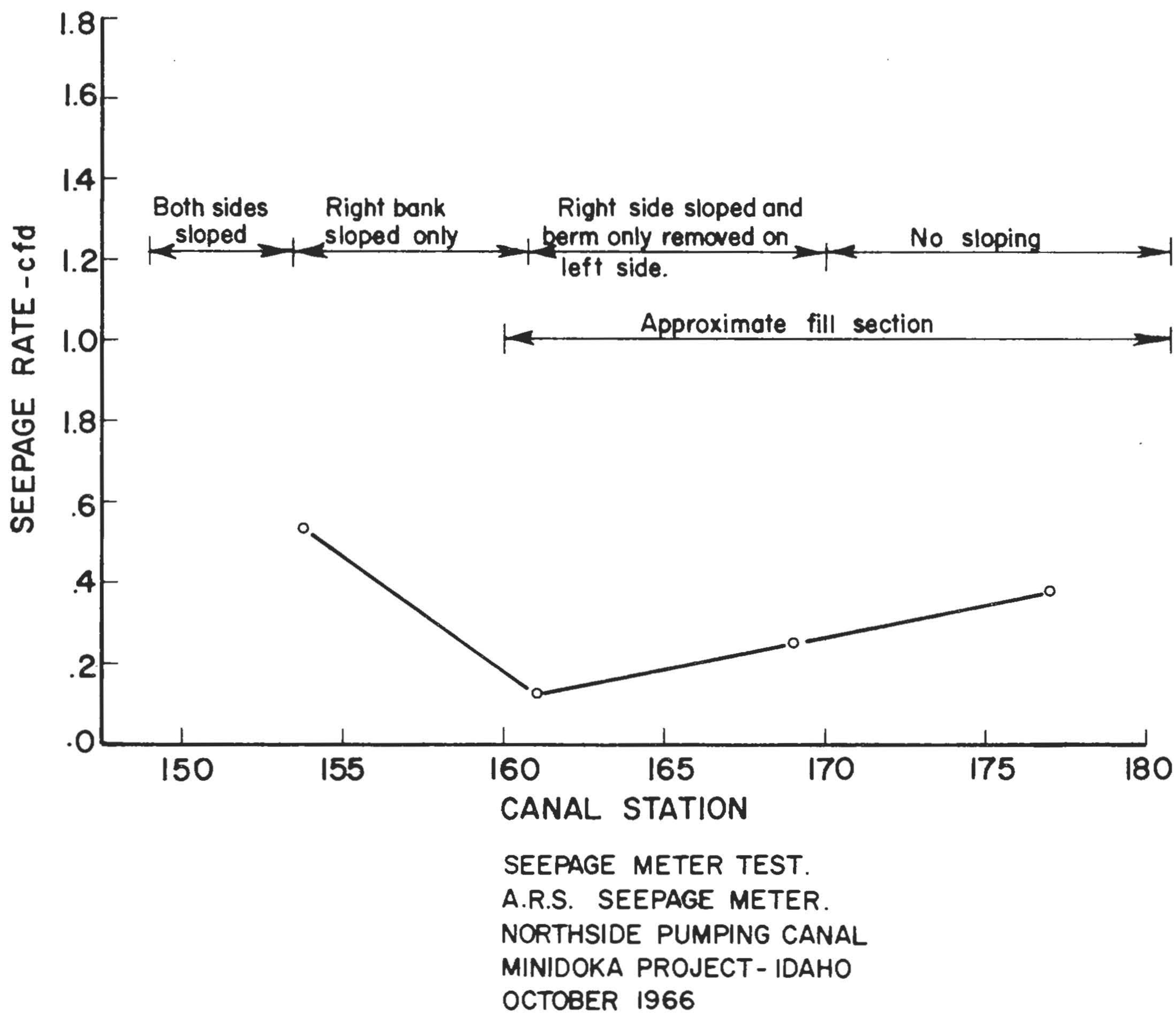


FIGURE 6 VARIATION OF SEEPAGE RATE IN LOWER TEST SECTION

a depth equal to the penetration of the meter bell is closely approximated by

$$R_a = \frac{H_b}{I_s}$$

where R_a = impedance of slowly permeable layer = $\frac{L_s}{K_s}$

I_s = seepage rate indicated by the meter.

H_b = measured balanced head.

Thirty-eight of the 60 tests in the upper test section were taken in areas where the canal bottom had not been disturbed and for each test the value of H_b/h_w exceeded 0.8 so that the average impedance of these 38 tests should be a reasonable approximation of the impedance of the restricting layer. Also, the value of the hydraulic conductivity of the restricting layer should be approximated by the average of the values as computed for each of the 38 tests. When the depth of penetration of the meter is less than the thickness of the restricting layer, the value of L_s used in the conductivity calculation should be the depth of penetration. The average value of the hydraulic conductivity of the impeding layer, K_s, for the 38 tests in the undisturbed bottom of the upper test section is .018 feet per day (5.49 mm/day). Similarly, for the 1966 lower test section the average value for the hydraulic conductivity of the impeding layer is .019 feet per day (5.50 mm/day) and for the 1965 test section the average value of K_s was .037 feet per day (11.3 mm/day).

Extrapolation of average seepage meter rates to an operating seepage rate in the 1966 test sections of the Northside Pumping Canal was not attempted. The sloping process performed on the canal prior to the tests eliminates the impeding effect of any silt layer on the seepage rate through the canal side slopes. This prevents any extrapolation of the seepage meter rates at a low water depth to operating depth by assuming a uniform impeding layer on the total canal perimeter.[1,2] Also, the presence of the consolidated layer which intersects the canal cross-section, Figure 8, complicates the procedure. The inability to determine operating seepage loss rates using the ARS meter negates the advantages of ease of operation and apparent accuracy. In order to make intelligent estimates of losses at operating depth, considerable time and effort must be spent in determining the location and properties of various strata underlying and intersecting the canal.

METHOD FOR ESTIMATING REQUIRED NUMBER OF SEEPAGE METER TESTS

STATISTICAL ANALYSIS

In making efficient use of seepage meters for estimating losses in existing canals and laterals, the question always arises as to how many measurements should be taken in a given reach of canal. There are many variables which must be considered, but with a few assumptions, the problem can be approached from a statistical standpoint.

First of all, it must be assumed that individual measurements are performed in the same manner by competent personnel using the proper technique for the particular type of meter. However, even if the same person performs all tests in the same manner using the proper technique, there will still be variability in the results which can be attributed to the measurement procedure. For instance, when inserting the seepage meter bell, it is not possible for each test to exactly duplicate the depth of insertion or the disturbance caused to the adjacent soil. These differences in soil disturbance as well as other small differences in technique will cause variability in the results. Soil variability, however, is probably the primary cause of variability in the results of any group of seepage meter tests. The following analysis is aimed at defining a level of confidence to be used in seepage meter tests based solely on the variability of individual measurements as affected by random variation of soils and human techniques.

In determining the variation in the results of a group of measurements, called a sample, the most common parameter is the square root of the sample variance or the standard deviation.

$$s = \sqrt{\frac{\sum_{i=1}^{N} (x_i - \bar{x})^2}{N-1}} \qquad 1$$

where s = standard deviation of the N measurements in the sample.

x_i = individual measurement.

$\bar{x}$ = mean of the N measurements or $\frac{\sum_{i=1}^{N} x_i}{N}$

Since the small group of measurements for which the standard deviation is calculated is essentially a sample from a much larger number of measurements or the total population, an estimate of the standard deviation of the means of a larger number of sample groups, $s_{\bar{x}}$ can be calculated from:

$$s_{\bar{x}} = s/\sqrt{N}$$

The coefficient of variation is another measure of the variation of the group of measurements and is usually expressed in percent as a ratio of the sample standard deviation to the mean of the sample group:

$$\%CV = \frac{100s}{\bar{x}} \qquad 2$$

In performing a series of measurements it is desirable to know how much confidence should be placed in the results of the measurements. For example, if 10 tests were performed in a one-mile reach of canal, it would be presumptuous to assume that the mean of the 10 tests, $\bar{x}_1$ was the same as the true mean, μ. It would, however, be logical to put less confidence in the mean of 10 tests, $\bar{x}_1$, than in the mean computed from 100 tests, $\bar{x}_2$. An inference as to how close the mean of the sample $\bar{x}$, might be to, μ, the true mean, can be obtained from the formula:

$$t = \frac{\bar{x} - u}{s_{\bar{x}}} \tag{3}$$

where t = a probability function called Student's t[15] dependent on the desired confidence level and the number of tests.

$\bar{x}$ = observed sample mean.

μ = mean of a large number of measurements or total population.

$\bar{x}-\mu$ = confidence interval.

$s_{\bar{x}}$ = estimate of the standard deviation of the mean of a large number of samples.

Since $s_{\bar{x}}$ can be approximated by $s/\sqrt{N}$, an interval in which we may reasonably expect the computed mean, $\bar{x}$, to fall is given by $\frac{t\,s}{\sqrt{N}}$, or in other words, the computed mean, $\bar{x}$, is probably within $\frac{t\,s}{\sqrt{N}}$ of the, μ, true mean. The value of t determines the degree of confidence to be placed in the computed mean, $\bar{x}$.

Now, if the confidence interval, $\bar{x}-\mu$, is expressed as a percent of the computed mean, $\bar{x}$, then:

$$\frac{D\bar{x}}{100} = \bar{x}-\mu = \frac{ts}{\sqrt{N}} \tag{4}$$

where D = the maximum percent by which the computed mean might vary from the true mean and the required number of tests can be estimated from:

$$N_c = \left(\frac{100ts}{D\bar{x}}\right)^2 \qquad 5$$

where N_c = computed number of tests required for a given confidence level.

It should be noted here the strong dependence of N on the computed mean value of the seepage $\bar{x}$. If $\bar{x}$ is reduced by one-half, say from 1.0 to .5 cfd, four times as many tests will be required to achieve the same percent error for a fixed confidence level and standard deviation. To use this equation for estimating N requires a decision as to the desired confidence level and allowable percentage difference, D, and an estimate of the standard deviation and average seepage rate for the group of tests to be performed. For estimates of seepage rates for feasibility studies on the economics of canal lining, if one could be reasonably sure that 9 times out of 10 the average of a group of seepage meter measurements was within 20 percent of the true mean, this should be sufficient. These limits would be defined by a confidence level of 90 percent and an allowable percentage difference, D, of 20 percent. After the first calculation of the required number of tests based on the estimated values, a few tests can be run and the estimate of N can be refined based on the new measured mean and a new computed standard deviation, s.

It should be pointed out that in this analysis, no indication of the absolute accuracy of the computed mean, $\bar{x}$, is possible. Only an estimate of the closeness of the computed mean of a small number of tests to the mean of a very large number of tests can be made. The determination of the accuracy of the seepage meter tests is possible only by comparing the computed mean, $\bar{x}$, with actual seepage rates determined by ponding.

Equation 5 is difficult to solve in its present form because of the dependence of N on t.

However, it can be expressed as:

$$D = \frac{100s}{\bar{x}} \frac{t}{\sqrt{N_c}}$$

and since $100s/\bar{x}$ = CV, the coefficient of variation,

$$D = \%CV \frac{t}{\sqrt{N_c}} \qquad 5$$

Since t for any confidence level is difficult to express mathematically as a function of N, solution of equation 5 is best solved graphically as shown in Figure 7. Using Figure 7, which is computed for a confidence level of 90 percent, and with initial estimates of s and $\bar{x}$ or CV, an estimate of the required number of tests to achieve a desired percent error, D, can be made. Also, after a number of tests are run, Figure 7 can be used to obtain a new estimate of N.

PROCEDURE

The following example illustrates a method to be used in estimating the required number of seepage meter tests, N. Assuming that a reasonable confidence level is about 90 percent and a reasonable percentage difference is probably about 20 percent, the requirement then is to determine the number of seepage meter tests, N, needed to be sure that 9 times out of 10, the true mean will be within 20 percent of the mean of the N tests. In the seepage meter tests with the ARS meter run in 1965 and 1966, the average standard deviation of 17 groups of tests for a total of 156 tests was .538 cfd. A similar analysis of 54 sample groups for a total 762 tests run with the U.S. Bureau of Reclamation meter on various types of soils showed an average standard deviation of .508 cfd.(4) A reasonable initial estimate of the standard deviation is probably about 0.5 cfd. For this example,

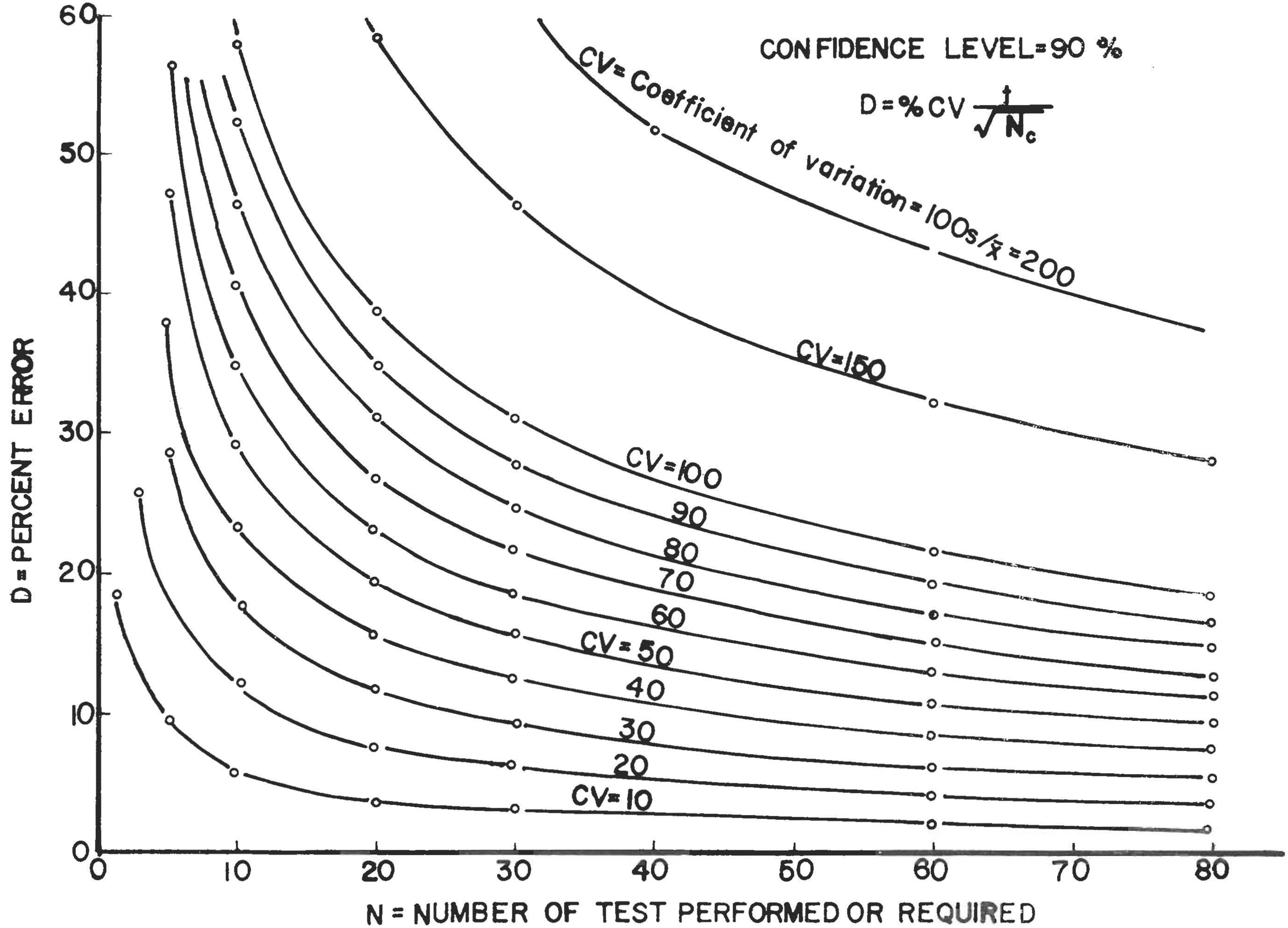


FIGURE 7 SEEPAGE METER STUDY-DETERMINATION OF REQUIRED NUMBER OF TESTS

suppose it is desired to determine the number of tests for a one-half mile reach of the Northside Pumping Canal in which the soil is Portneuf silt-loam. Estimates of the average seepage rate for the soil are from about .5 to 1.0 cfd. Using an initial estimate of .75 cfd for the average seepage rate and 0.5 cfd for the standard deviation, the estimated coefficient of variation is $100(.5)_{.75} = 66.7$. For a percent error of 20 and a confidence level of 90 percent Figure 7 shows that a minimum of 32 tests would be required.

Table 4 outlines the procedure for obtaining a new estimate of N. The results shown in the first part of Table 4 are for the series of seepage tests performed in 1965 on the test section of the Northside Pumping Canal. After 10 tests were taken the calculated mean seepage rate was .727 cfd and the standard deviation,s, as computed from equation 1 was .481. The new CV is therefore 66. Again using Figure 7, the new estimate of the required number of tests is 31. After 20 measurements the revised estimate was 28 tests required and after 30 tests the required number of tests was 25. The required percentage difference was achieved after 30 tests and the testing could have been terminated.

For the 1965 tests after 71 tests had been taken, the computation of N showed that only 29 tests would have been necessary to achieve the 90 percent confidence level and 20 percent error. The actual D determined from Figure 7 for the 71 tests is 13 percent so that for this group of tests one can say that 9 chances out of 10 the true mean of the 71 tests is within 13% of the measured mean or within (.69) (.13) = .09 cfd. Table 4 also shows the same analysis for seepage meter tests in the upper and lower test sections of the Northside Pumping Canal in 1966. All of the computations of the required N shown in Table 4 are based on Figure 7 which is drawn for a confidence level of 90 percent and a percent error Of 20. The same analysis

TABLE 4

STATISTICAL ANALYSIS OF SEEPAGE METER TESTS
NORTHSIDE PUMPING CANAL

TEST SECTION, 1965

	Initial Estimate	Number of Tests Completed 10	20	30	40	51	61	71
Mean Seepage $\bar{x}$	.75	.727	.658	.614	.661	.643	.693	.673
Std. Deviation s	.5	.481	.410	.357	.364	.387	.448	.432
Coef. of Variation CV	66.7	66.2	62.4	58.2	55.1	60.1	64.6	64.2
Tests Required N	32	31	28	25	22	26	30	29
Actual D		39	24	18	15	14	14	13

UPPER TEST SECTION, 1966

	Initial Estimate	Number of Tests Completed 10	18	28	38	49	60
Mean Seepage $\bar{x}$	.75	1.113	.747	.744	.785	.783	.692
Std. Deviation s.	.5	.864	.778	.740	.908	.923	.869
Coef. of Variation CV	66.7	77.5	104	106	116	118	125
Tests Required N	30	41	75	78	95	102	105
Actual D		41	43	34	32	28	27

LOWER TEST SECTION, 1966

	Initial Estimate	Number of Tests Completed 7	14	21	26
Mean Seepage $\bar{x}$	.5	.540	.331	.304	.319
Std. Deviation s.	.5	.886	.643	.535	.518
Coef. of Variation CV	100	164	194	176	162
Tests Required N	70	254	295	230	196
Actual D		120	92	66	54

could be carried out for a confidence level other than 90 percent by using equation 5 or curves similar to Figure 7.

DISCUSSION

In Table 4 the results of the analysis for tests in the upper and lower test sections of the Northside Pumping Canal in 1966 show that the variability of the measurements was considerably greater than for the 1965 tests. For the upper test section in 1966 the initial estimate of N was quite low as evidenced by the revisions of N as increasing numbers of tests were taken. The initial estimate is low primarily because of the poor estimate of the standard deviation. It is interesting to note in this series of tests that the variability of the measurements as indicated by the tabulated coefficient of variation, CV, increased with increasing numbers of tests whereas for the 1965 tests CV did not change appreciably. After 60 tests were taken, the determination of N showed that 105 tests would have to be taken to achieve the desired 20 percent error. This number of tests is obviously too large to be practical.

In the 1966 lower test section, the initial estimate of N was low, however, after 7 tests were taken the revised N even though very large (254) was at least close to the final required N (196) determined after 26 tests. These results point out a misconception which often prevails concerning the use of seepage meters or in the use of point measurements to obtain average values of a quantity over a large area. The common belief is that any percent error can be achieved if enough tests are taken. Theoretically this is true but from a practical standpoint it is not. For instance, in Table 4, the lowest percent error achieved after 60 tests on the 1966 upper test section was 27 for a confidence level of 90 percent. It is highly unlikely that any practical use of a seepage meter would involve as many as 60 tests per half mile of canal. More likely, for investigative purpose on feasibility studies 10 or 20 tests per half-mile might be the practical limit in which case it would be necessary to settle for a higher percent error for a given confidence level. Actual percent errors for a 90 percent confidence level attained on the three test sections of the Northside Pumping Canal are also shown in Table 4.

COMPARISONS OF SEEPAGE METER AND PONDING TEST MEASUREMENTS

A comparison of seepage rates obtained in 1966 from ponding tests and seepage meters can be made for the upper test section of the Northside Pumping Canal. The 1965 ponding tests showed that for low water depths the difference between seepage rates computed for the first and second pond fillings was negligible. The ponded rate at low water depths should therefore be indicative of the actual seepage even though only one pond filling was possible. Table 5 shows the comparison of the 1965 and 1966 seepage meter tests with ponding rates for two one-half mile ponds.

TABLE 5
PONDING RATE AND SEEPAGE METER COMPARISON

	1965	1966	Avg. 1965-1966
Average ℄ water depth	22 inches	20 inches	21 inches
Number of tests	71	60	65.5
Percent of wetted area tested	.092%	.086%	.089%
Ponded seepage rate	.50 cfd	.56 cfd	.53 cfd
Seepage meter rate	.68 cfd	.69 cfd	.685 cfd
Difference	.18 cfd	.13 cfd	.155 cfd
Percentage difference	36%	23%	29.5%

The results of the two series of seepage meter tests are encouraging. First of all the apparent repeatability of the meter under similar conditions is evident. The meter rates for both series of tests are higher than corresponding ponded rates by an average of about 30%. However, in this case, the absolute difference in seepage rates is more significant than percentage difference because of the low seepage rates. An error of 0.15 cfd in estimating the seepage rate is certainly tolerable whereas an error of 30 percent may not be tolerable for higher rates.

One reason for the 30 percent difference between the ponded rate and the seepage meter rate may be the difference in location of the water surfaces during the tests. During the seepage meter tests a low flow was maintained in the canal and the water surface slope was essentially equal to the friction gradient or about .9 feet per half mile. During ponding tests, the water surface was level so that even though the ponded rate is computed at a water surface elevation corresponding to the average elevation of the sloping water surface during seepage meter tests the actual area sampled is not the same as in the seepage meter tests. An estimate of the magnitude or even the direction of the difference attributable to this effect is difficult. Of course, the possibility that the meter itself does not measure the true seepage rate of the soil into which it is inserted should not be overlooked. The continual problem of disturbance of the soil during insertion of the meter bell could cause indicated seepage rates to be higher than actual as could an insufficient seal between the bell and the soil. However, with the ARS meter, a test is always performed to check the seal prior to each measurement.[(3)] A previous investigation has shown that when the meter bell is pushed in by hand or stepped on to insert it into the canal bottom the indicated average seepage meter rate can be as much as 23% greater than the ponded rate.[(5)] In both the 1965 and 1966 tests, less than .1 percent of the wetted perimeter of each pond was actually sampled with the seepage meter and yet the error in the seepage rate estimate averaged only .15 cfd.

INFLOW - OUTFLOW LOSS MEASUREMENTS

Inflow - outflow loss measurements were performed by the Bureau of Reclamation on the 4.56 miles of the Northside Pumping Canal. The measurements were made on a volumetric basis for 12 periods approximately 2 weeks long during the irrigation season. The losses were computed from continuous flow records on all turnouts from the canal and were adjusted for measured evaporation losses. Wetted surface area was determined from survey data on 37 cross-sections throughout the length of the canal.

Measured loss rates for the two week periods varied from 1.56 cfd at the start of the irrigation season to 0.86 cfd near the end of the season. Table 5[6] shows the variation in loss rate throughout the irrigation season. The loss rates measured by the inflow-outflow method cannot be attributed entirely to seepage losses. Small leaks through turnout gates which are below measurable flows and other operating losses are also included in the measured loss rates. The average inflow-outflow loss rate of .86 cfd near the end of the irrigation season is higher than the average rate from ponding tests of .65 cfd. The ponded rate is an average rate as measured on 1.5 miles of canal during the fall of 1965 and 1966. The repeatability of the measurements throughout the season, and from season to season, is much greater than is normally expected using inflow-outflow methods.

The decrease in measured loss rates throughout the season reflects the gradual sealing of the canal as indicated by tensiometer measurements.

TABLE 6

INFLOW - OUTFLOW LOSS MEASUREMENTS

NORTHSIDE PUMPING CANAL

1966

Period by dates	Days in Period	Inflow AF	Outflow AF	Total Loss AF	Evaporation AF	Other Loss AF	Average Daily Loss AF	Average Daily Loss Cu.ft./day	Average Daily Loss Cu.ft/sq.ft./day
April 13 - May 2	19	2217.93	1787.82	430.11	4.74	425.37	22.40	975,740	1.56
May 2 - May 16	14	4501.37	4263.29	238.08	4.44	233.64	16.69	727,020	1.12
May 16 - June 1	16	5925.25	5673.93	251.32	5.41	245.91	15.37	669,520	1.04
June 1 - June 16	15	5884.45	5647.44	237.01	4.12	232.89	15.53	676,490	1.03
June 16 - July 1	15	6128.51	5899.92	228.59	3.98	224.61	14.97	652,090	0.99
July 1 - July 15	14	6725.80	6496.63	229.17	4.31	224.86	16.06	699,570	1.05
July 15 - Aug. 1	17	7956.14	7665.65	290.49	5.71	284.78	16.74	729,630	1.10
Aug. 1 - Aug. 15	14	5040.76	4825.73	215.03	3.28	211.75	15.12	658,630	1.00
Aug. 15 - Sept. 1	17	5937.24	5672.56	264.68	5.43	259.25	15.25	664,290	1.01
Sept. 1 - Sept.15	14	2800.93	2608.12	192.81	2.70	190.11	13.58	591,540	0.91
Sept.15 - Sept.30	15	1924.28	1729.60	194.68	3.56	191.12	12.74	554,950	0.86
Sept.30 - Oct. 14	14	1824.24	1646.94	177.30	2.32	174.98	12.50	544,500	0.88
Total & Average	184	56866.90	53917.63	2949.27	50.00	2899.27	15.76	686,510	1.06

FIELD TENSIOMETER STUDIES NORTHSIDE PUMPING CANAL

CANAL STATION 104+00

PROCEDURE

To investigate the magnitude and seasonal variation of soil water pressure beneath an operating canal prism, 10 observation piezometers were installed at station 104+00 of the Northside Pumping Canal, Figure 8. Seven of the piezometers were 1 inch diameter electrical conduit and were installed in the operating canal. Three piezometers were of .75 inch diameter steel boiler tubing driven into the soil of the canal bank. The piezometers in the canal were driven to depths of about 1.0, 2.0, or 3.0 feet below the bottom using a driving point inserted through the tube and then removed after installation. Water immediately went out of all eight of the piezometers in the canal indicating unsaturated flow beneath the entire canal prism. Piezometer M located 42 feet left of the canal centerline indicated no local water table at a depth of 42 feet below the ground surface. However, piezometers E and L located 15 feet and 17.7 feet left of the canal centerline respectively indicated a perched water table about 12 feet below the bottom of the canal.

In order to measure the soil moisture tension in the unsaturated soil beneath the canal, tensiometers were installed in four of the eight piezometers in the canal. The tensiometer tips consisted of 3/8 inch diameter porous porcelain cups about 2 inches long, Figure 9A. Two .096 inch diameter nylon tubes, attached to the cup with epoxy resin cement, were used as an indicator tube and bleed tube so that accumulated air bubbles could be removed from the cup. The cups were pushed into the soil at the end of each piezometer tube using a special inserting device and the

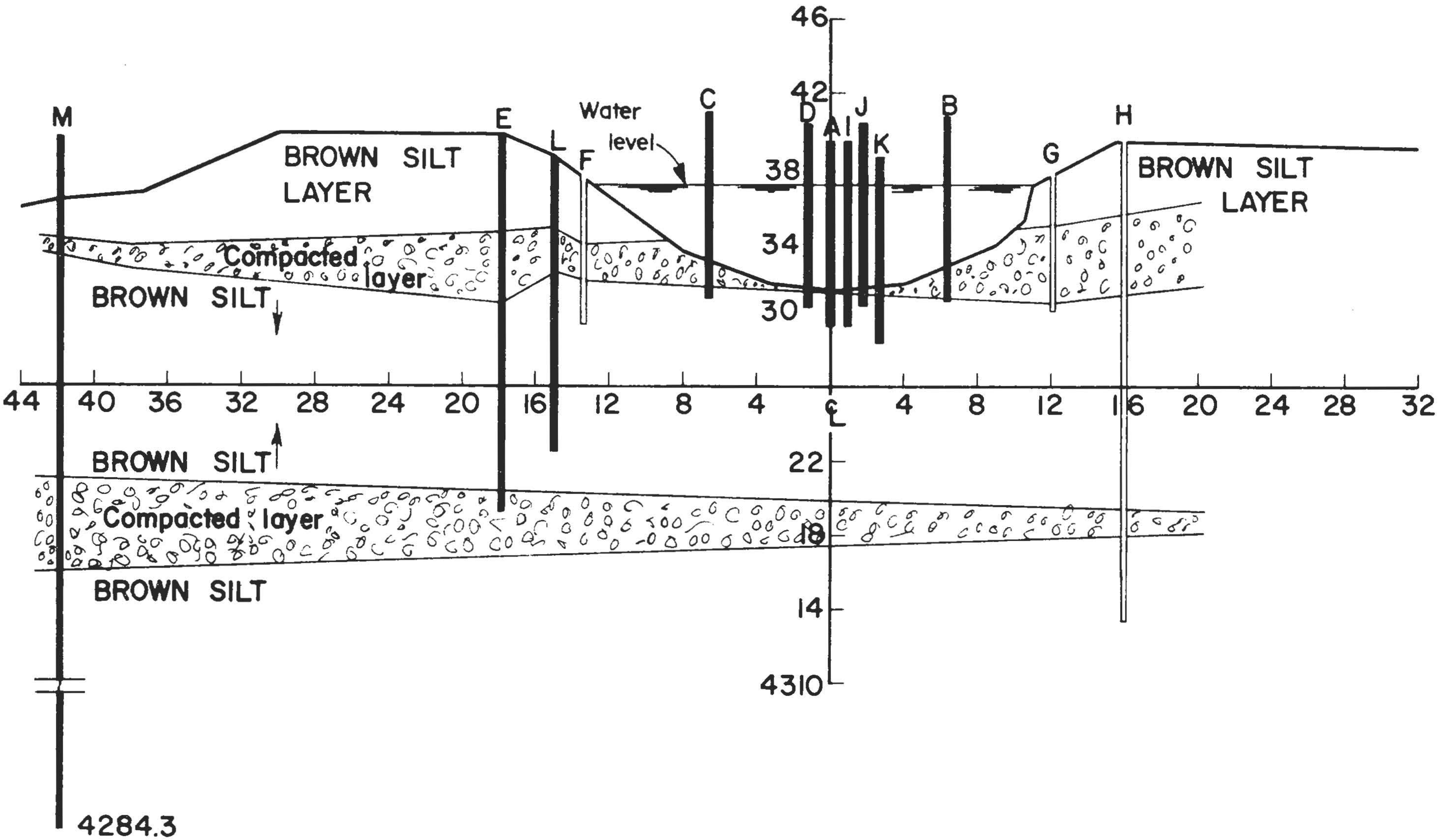
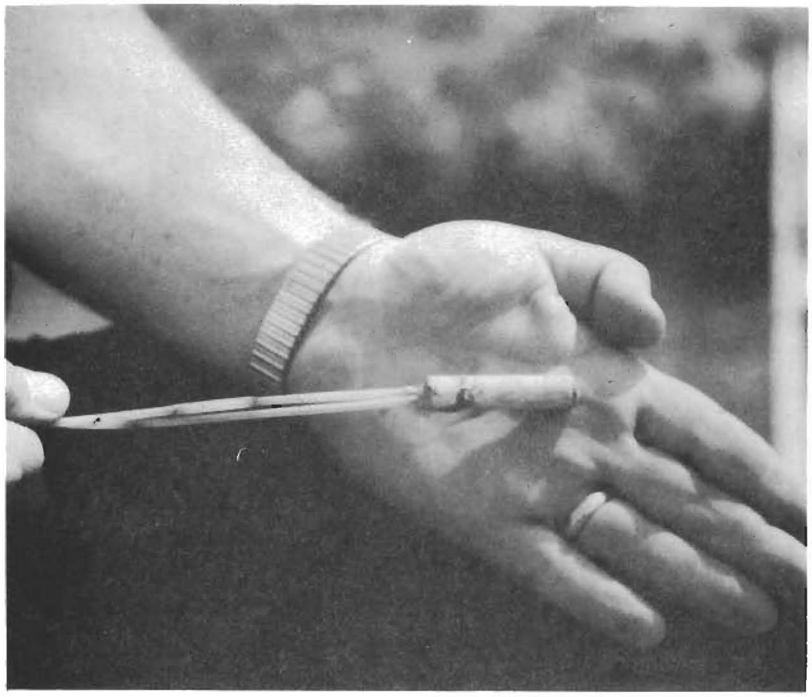
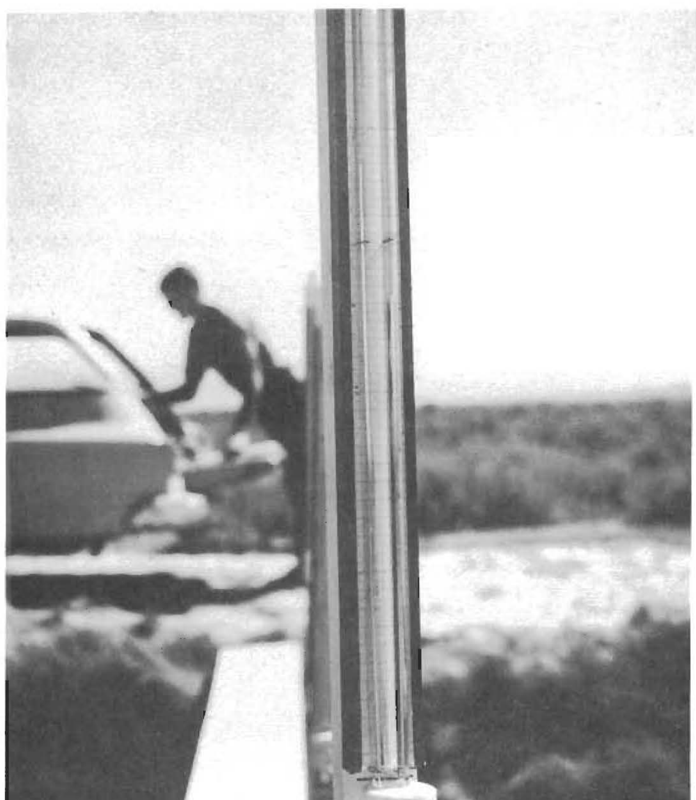


FIGURE 8 LOCATION OF PIEZOMETERS AND TENSIOMETERS
STATION 104 NORTHSIDE PUMPING CANAL , 1966



A. Porous Porcelain tensiometer tip.



B. Double tube mercury manometer for tensiometer readout.

FIGURE 9 TENSIOMETER TIP AND MANOMETER
FIELD TENSIOMETER STUDIES
NORTHSIDE PUMPING CANAL

piezometer tube was left open to the atmosphere. A mercury pot-type manometer constructed from 1.5 mm I.D. glass capillary tubing and a small plastic vial was used as a readout for the tensiometers, Figure 9B. With this tensiometer system water pressures could be read to the nearest millimeter of mercury and the system proved quite efficient and reliable for field use.

RESULTS

Figure 10 shows a plot of the elevation potential, water pressure plus elevation, measured by the piezometers and tensiometers at station 104 over the irrigation season. Definite fluctuations in potential are evident in all the tensiometers and the fluctuations are mirrored by the changes in the perched water table as measured by piezometers E & L.

DISCUSSION

No definite reasons can be given for the fluctuations observed in the tensiometers throughout the season. However, the long term general decrease in potential or increase in soil moisture tension beneath the canal can be attributed to the gradual sealing of the canal perimeter with the growth of the impedance of the silt layer or surface layer in the canal bottom. Figure 11 shows a comparison of the fluctuation in barometric pressure and maximum daily temperature with the elevation potential over the season. The maximum daily temperature was used for comparison in this case since tensiometer readings were normally taken between noon and 3:00 P.M. No definite correlation exists between the potential as measured by tensiometer 104 D beneath the canal and either barometric pressure or maximum daily temperature. It was thought that perhaps the perched water table and therefore the elevation potentials were affected by the rates and frequency of irrigation of fields adjacent to the canal. Figure 11 also shows the times and rates of irrigations on a pea field and

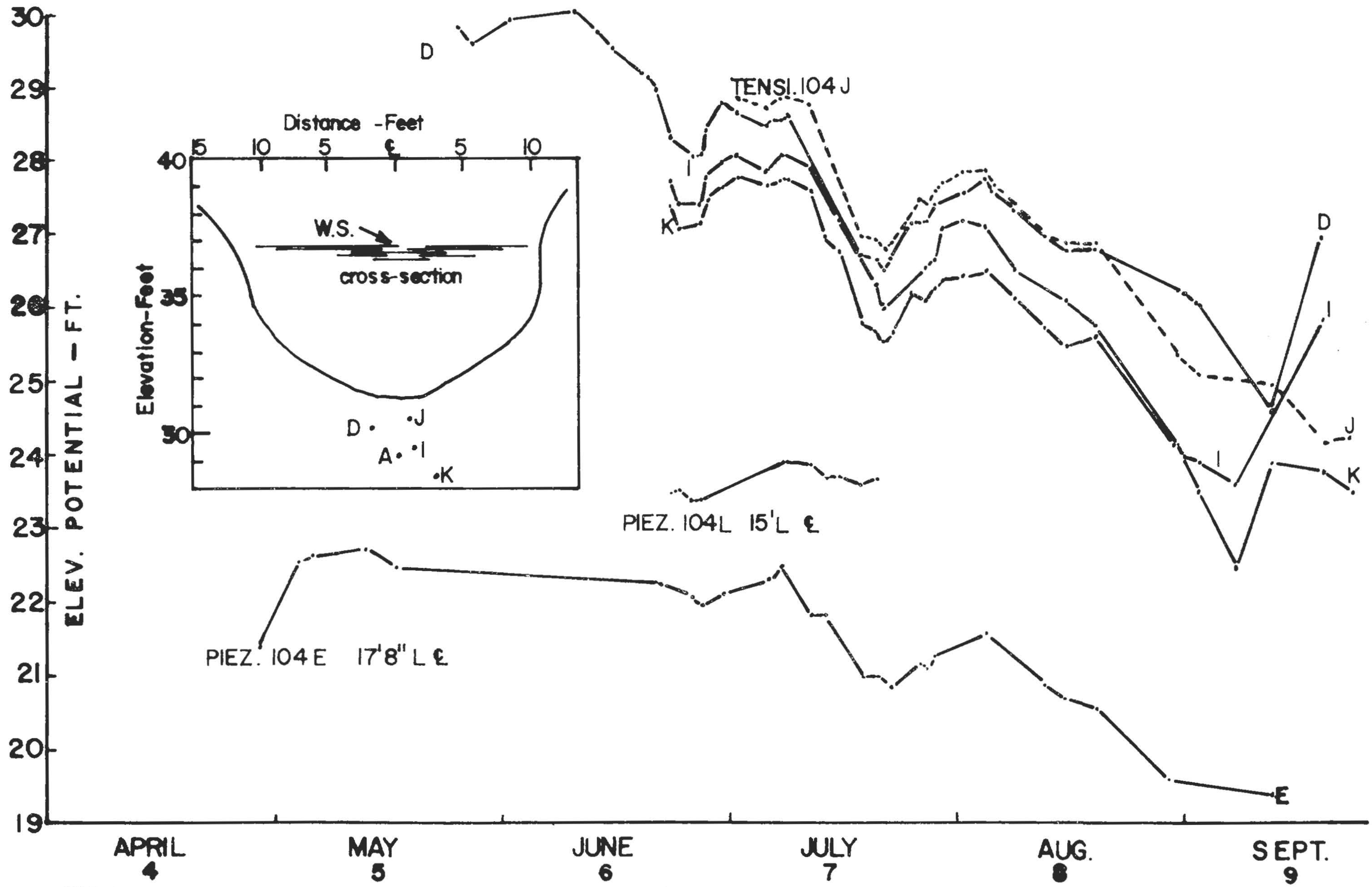


FIGURE 10 TENSIOMETER AND PIEZOMETER POTENTIALS RECORDED AT STATION 104 DURING 1966 IRRIGATION SEASON

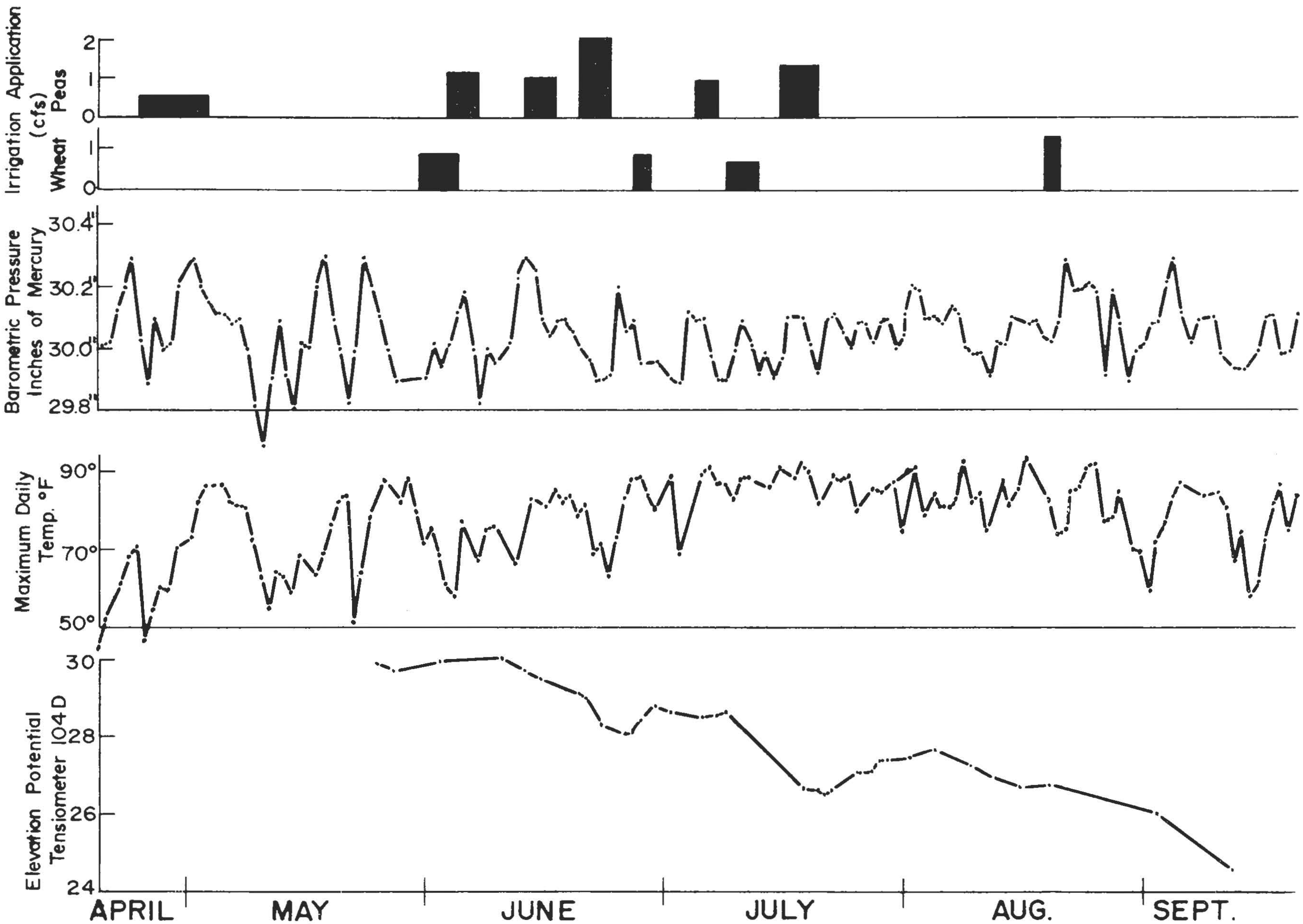


FIGURE 11 EFFECT OF TEMP., BAROMETRIC PRESSURE, AND IRRIGATION APPLICATION ON ELEVATION POTENTIAL NORTHSIDE PUMPING CANAL, 1966

wheat field located adjacent to and on the same side of the canal.* The land on the side of the canal opposite the irrigated fields is brush covered and not being farmed. In this instance at least, short term fluctuations in the potentials beneath the canal apparently cannot be attributed to the application of irrigation water to adjacent fields.

Tensiometer 104 D located 2' below the canal bottom, Figure 10, was monitored effectively for 110 days during the irrigation season and showed a drop in potential of 5.5 feet which corresponds to an increase in soil water tension from 0.0 to 5.5 feet of water. Similar changes in soil moisture tension were measured for the other three tensiometers at station 104. These observed changes in moisture tension over the season indicate significant changes in the unsaturated hydraulic conductivity of the soil beneath the canal with corresponding changes in seepage rate.

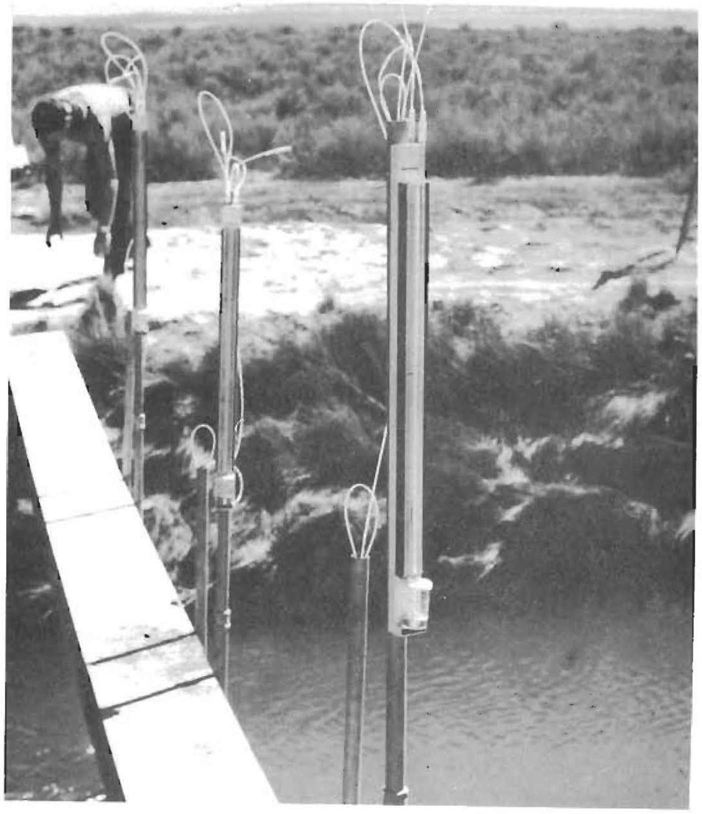
CANAL STATIONS 132+75, 133+00 and 133+14

PROCEDURE

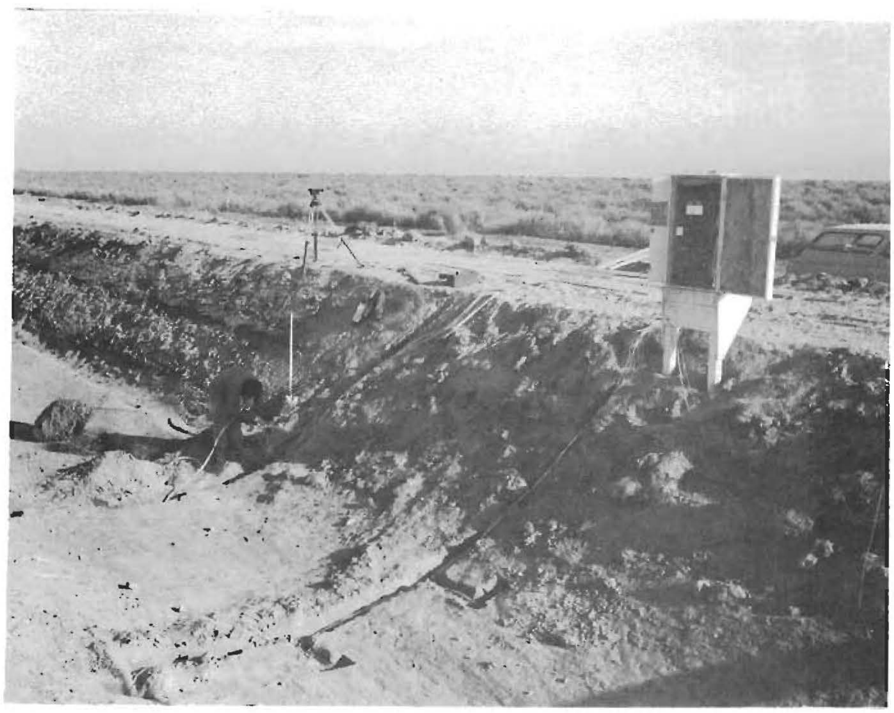
Tensiometers installed in the bottoms of piezometer tubes may not indicate the correct soil moisture tension because of disturbances in the flow field caused by the presence of the piezometer tube. To compare methods of tensiometer installation, a section of the ponded reach of the Nortside Pumping Canal was instrumented. At Station 132+75, six 3/8" diameter porous cups were installed in the ends of piezometer tubes using the same procedure as followed at Station 104+00. Figure 12A shows the piezometer tubes and manometers for reading the tensiometers at Station 132+75.

At Station 133+14, nine 3/8" diameter porous ceramic cups were installed beneath the canal perimeter by pushing them into the vertical

* Irrigation application data, courtesy U. S. Bureau of Reclamation.



A. Piezometer tubes and mercury manometers for tensiometers at Station 132+75.



B. Installations in pits at Station 133+00 and 133+14.

FIGURE 12 METHODS OF TENSIOMETER INSTALLATION
FIELD TENSIOMETER STUDIES
NORTHSIDE PUMPING CANAL.

sides of pits dug into the canal bottom, Figure 12B. Three tensiometers were placed in each of three pits at depths of 6, 12, 24 inches. The nylon tubes were lead from the tensiometer cups to the canal bank through shallow trenches. Approximately 1-inch of bentonite was placed against the vertical side of the pits during backfilling to provide an impervious layer between the tensiometers and the disturbed soil in the pit. A thin layer of bentonite was also placed over the top of the pits and trenches during backfilling. All of the tensiometers at Station 133+14 were connected to mercury manometers installed on the canal bank.

A cross-section of the canal at Station 133+00 was also instrumented with nine tensiometers placed similarly to those at Station 133+14. The tensiometer tips used were of 3/16 inch glass tubing with 1/4 inch long hollow porous ceramic tips fused to the glass tube. Two nylon tubes were again used for indicator and bleed tubes.

The readout unit for the tensiometers at Station 133+00 consisted of a 10 tube mercury manometer. The manometer cabinet included a water supply reservoir for bleeding the tensiometers, a common mercury reservoir and proper valving to bleed and drain each tensiometer tip. A schematic diagram of one tube of the system is shown in Figure 13, and the cabinet can be seen on the canal bank in Figure 12B. To eliminate the response time required for the manometer system, a pressure transducer, and stepping multipoint valves were installed. The transducer, valves, and transducer indicator are all battery powered with the power source located in the cabinet. The differential between any two tensiometers or between any tensiometer and atmospheric pressure could be read by positioning each of the two solonoid operated stepping valves. The complete schematic diagram for the operation of the valving system is not shown in Figure 13. Unfortunately, the tensiometers at Station 133+00 did not function during the ponding test conducted in 1966.

TENSIOMETER READ-OUT UNIT

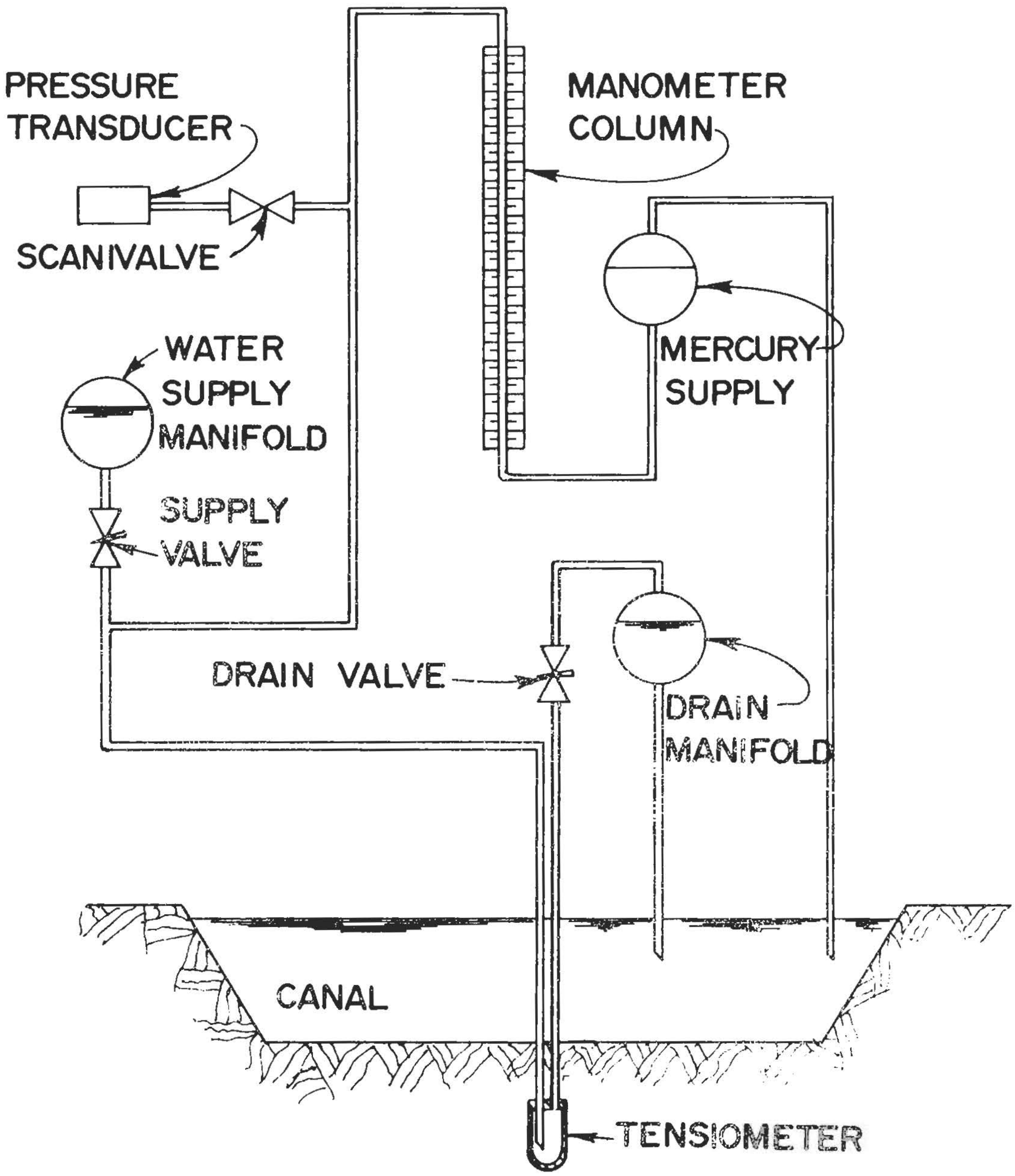


FIGURE 13 TENSIOMETER READ-OUT SYSTEM
NORTHSIDE PUMPING CANAL 1966

It was determined that the nylon tubing that had been inserted into the tips for indicator and bleed tubes had expanded as it became wet and pressed the open end of the bleed tube against the end of the tensiometer tip thereby preventing bleeding of the tensiometers. Later tests showed that the .056 diameter nylon tubing elongated about 1 percent after soaking in tap water for 48 hours. The tensiometer readout system including multipoint valves and pressure transducer worked effectively in the laboratory but could not be evaluated in the field.

RESULTS

Readings were taken on all tensiometers for approximately 170 hours during the time the ponding tests were being performed. Figure 14 shows the locations of the six tensiometers at Station 132+75 and the elevation potentials measured during the ponding test. Since the water surface elevation in the pond was always dropping as shown in Figure 14 during the entire seepage test, the flow system was in a transient state and no steady state conditions were achieved.

The elevation potentials measured by the tensiometers at Station 133+14 for the 170 hour ponding test are shown in Figures 15, 16 and 17.

DISCUSSION

The primary reason for instrumenting canal Stations 132+75, 133+00, and 133+14 was to examine various methods of tensiometer installation and readout for subsequent use in field investigations. The performance of tensiometers installed in piezometers, Figure 14, is encouraging and except for some discrepancies caused by leaks in the bleed system, the gradients measured during the 7 day ponding test are reasonable. Some difficulty was encountered in maintaining tensiometer F properly bled and as a result the gradients measured between tensiometer E and F are

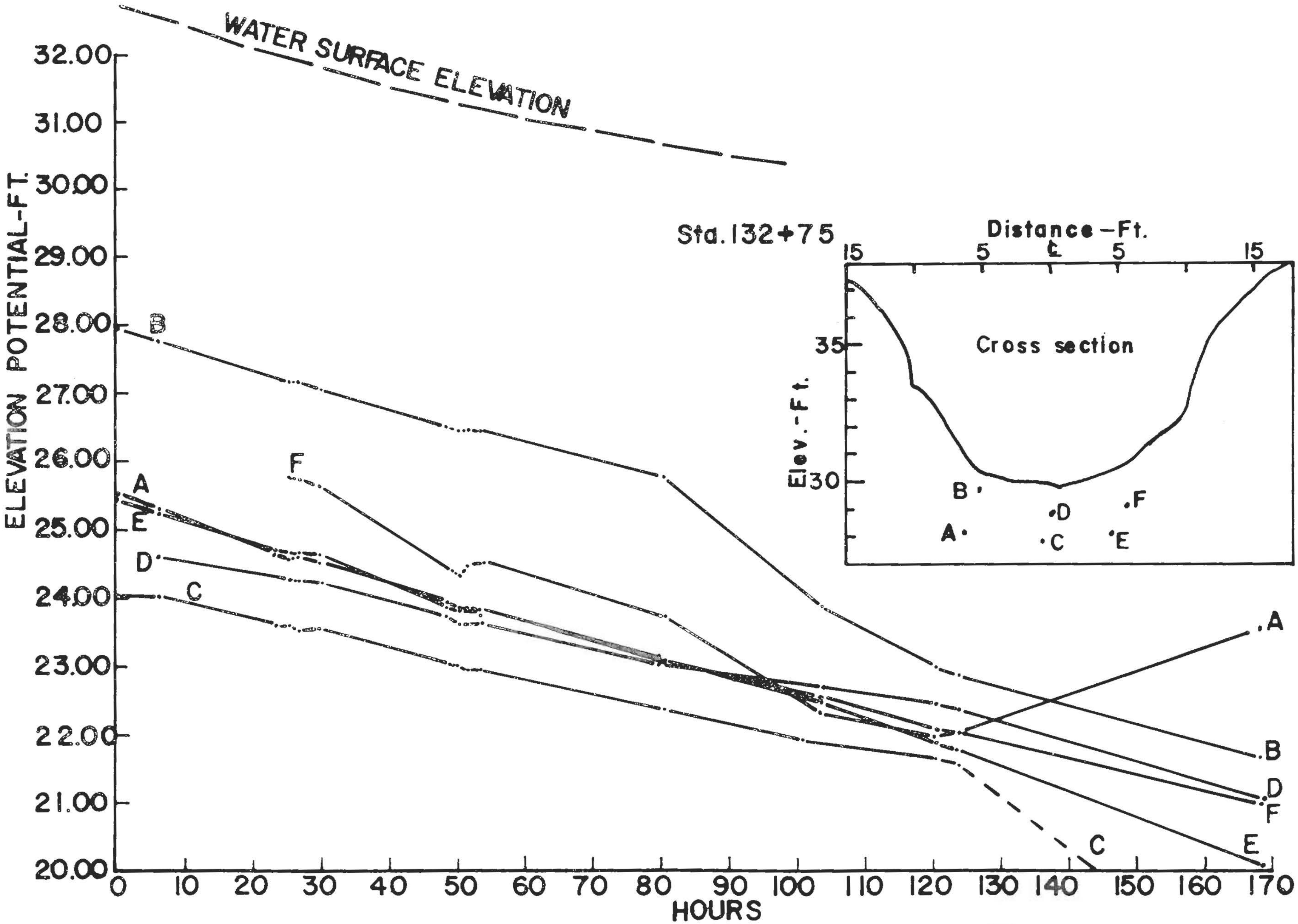


FIGURE 14 CHANGING POTENTIALS AS RECORDED AT STATION 135+75

DURING A SEVEN-DAY PONDED SEEPAGE TEST (FALL 1966).

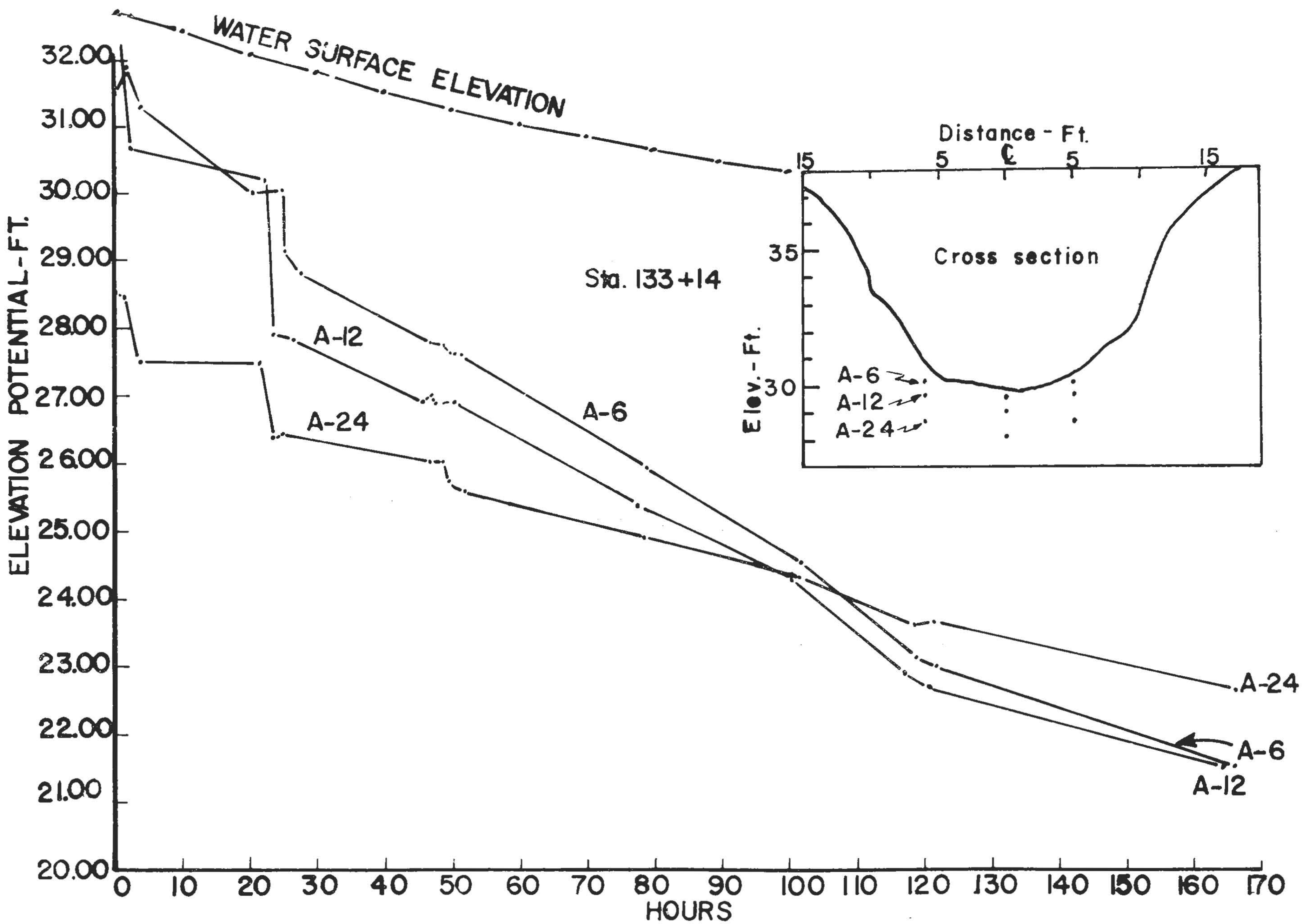


FIGURE 15 CHANGING POTENTIALS RECORDED AT STATION 133+14 DURING PONDED SEEPAGE TEST (FALL 1966). (LEFT)

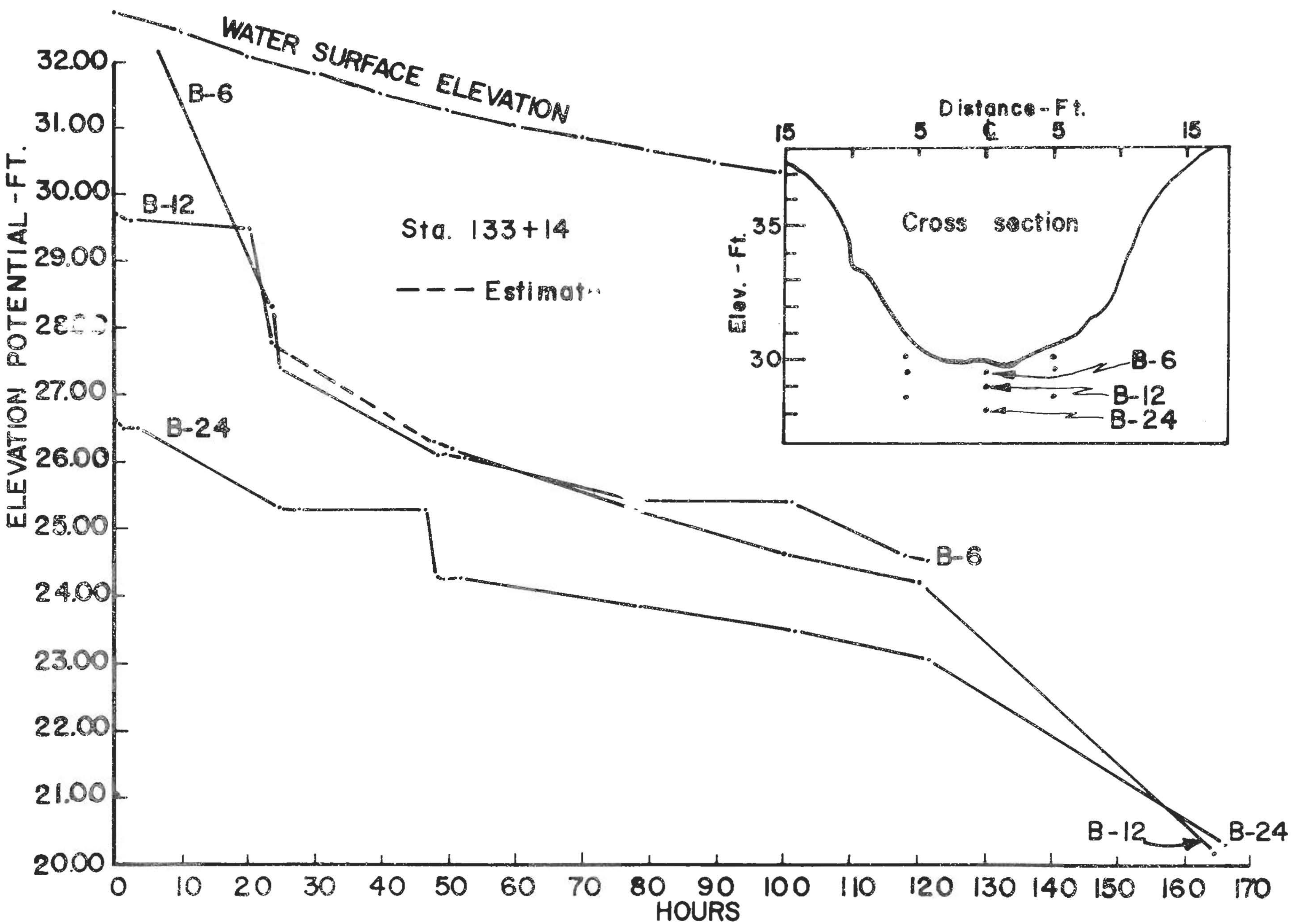


FIGURE 16 CHANGING POTENTIALS RECORDED AT STATION 133+14 DURING PONDED SEEPAGE TEST (FALL 1966). (CENTER)

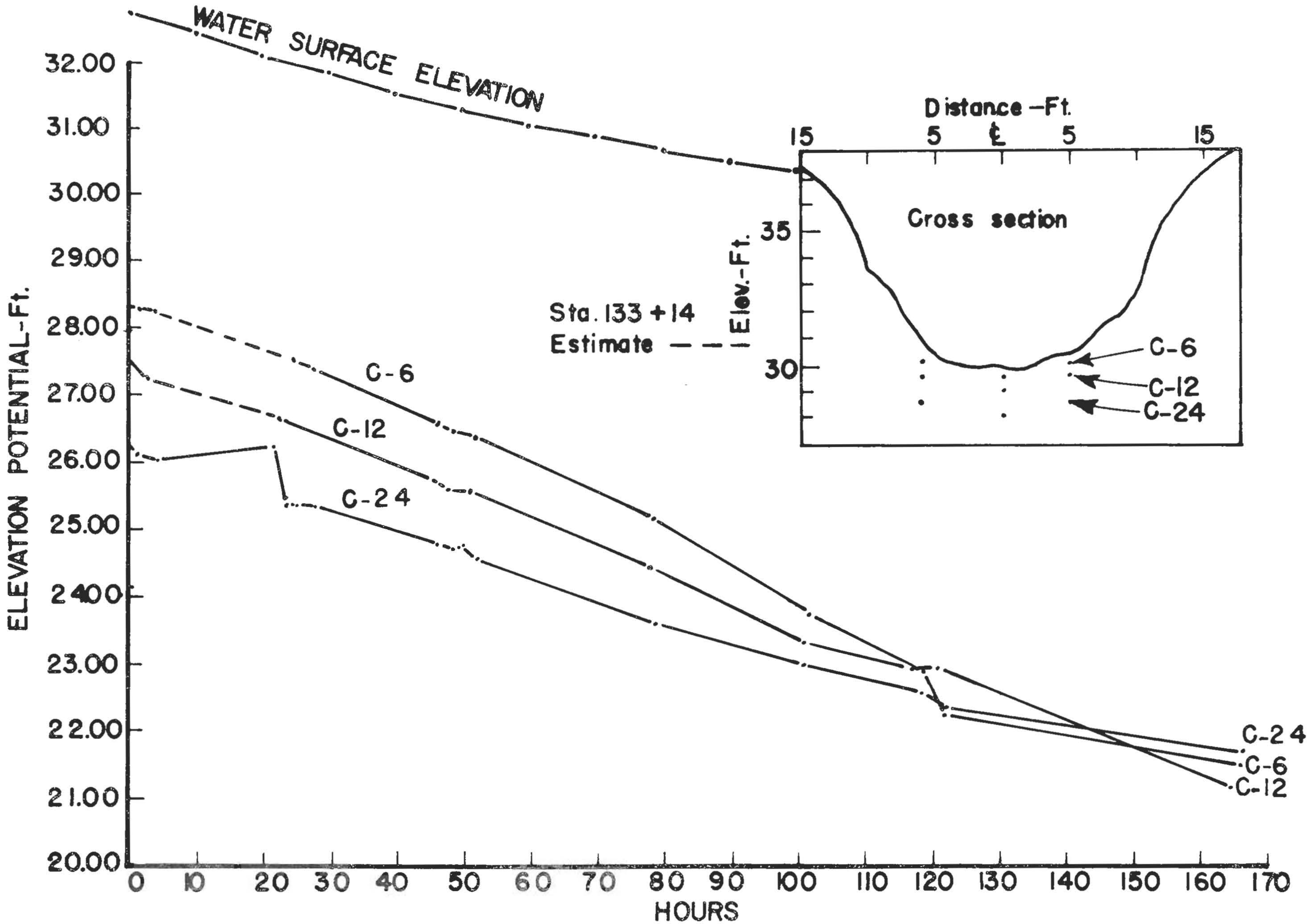
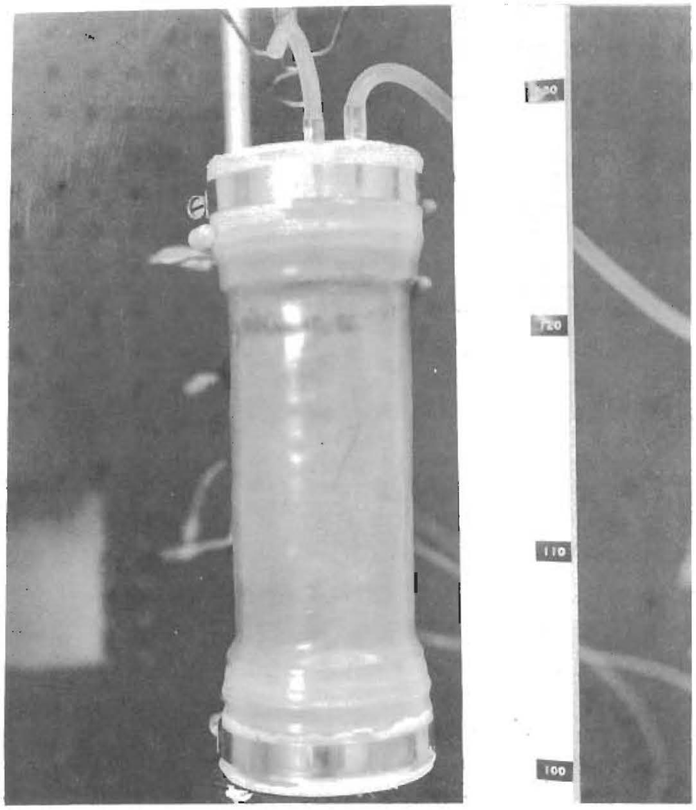
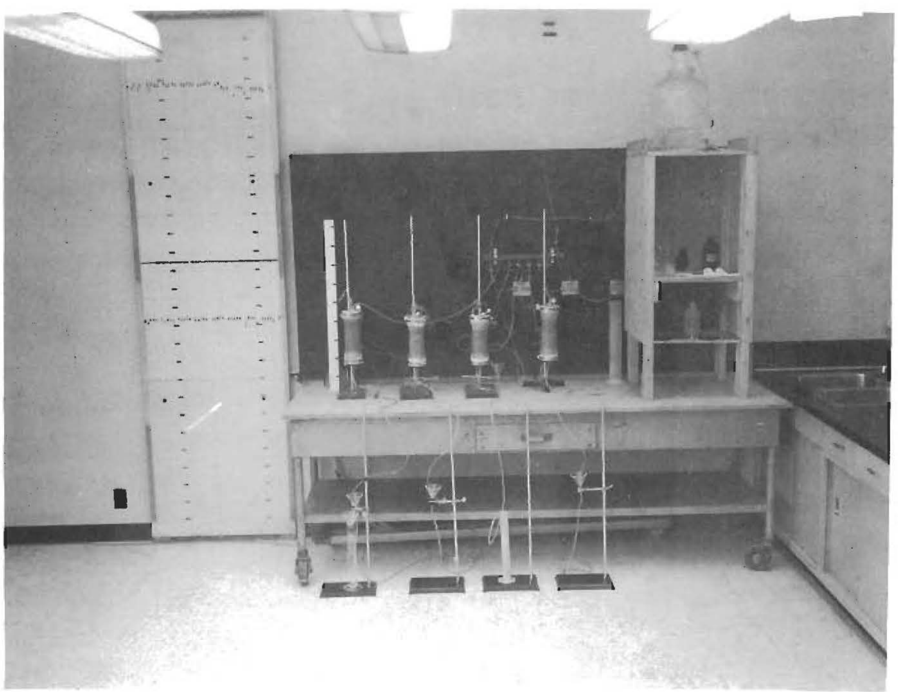


FIGURE 17 CHANGING POTENTIALS RECORDED AT STATION 133+14 DURING THE PONDED SEEPAGE TEST (FALL 1966). (RIGHT)



A. Undisturbed soil core in shrinkable plastic tubing.



B. Laboratory equipment with tests in operation.
FIGURE 20 LABORATORY APPARATUS FOR CONDUCTIVITY STUDIES
INVESTIGATION OF IMPEDING LAYER
NORTHSIDE PUMPING CANAL

erratic. The response of all tensiometers at Station 132+75 to the falling water table in the canal was good. The greatest advantage which the piezometer type installation has over the pit installation is the ability to install the tensiometer while the canal is in operation. However, it is difficult to install tensiometers in piezometers at shallow depths in the soil. The piezometer tube must be pushed deep enough into the canal bottom and be rigid enough to prevent leakage around the tube when subjected to forces and vibrations of flowing water.

Performance of the nine tensiometers installed in pits at Station 133+14 is considerably more erratic than the tensiometers at Station 132+75. There is some question whether the use of bentonite in the pits is advisable. During the wetting process the bentonite swells considerably and since it is in contact with the side of the pit into which the tensiometer tips were inserted, it could conceivably have affected the measured soil moisture tension. The continued swelling of the bentonite may have caused increased pressure around the tensiometer tips thereby lowering indicated tensions. The average soil moisture tension measured by the three, two-foot deep tensiometers at Station 133+14 was 1.1 feet of water lower than the average of the three similar tensiometers at Station 133+75.

Based on the results of these studies it appears that the installation of tensiometer tips in the soil at the ends of piezometer tubes is both more convenient and produces more reliable results than the installation in the sides of pits excavated in the canal bottom. Additional studies during the 1967 irrigation season will explore the use of additional installation methods such as pushing the tensiometers vertically into the soil beneath the canal without excavating pits.

INVESTIGATION OF IMPEDING SILT LAYER
NORTHSIDE PUMPING CANAL

DESCRIPTION OF LAYER

The accumulation of natural silt layers on the bottoms of operating canals and their beneficial effect in reducing seepage losses is well known.[7,8] However, the magnitude of the seepage loss reduction and physical conditions conducive to the development and preservation of silt layers are not fully understood. The Northside Pumping Canal offers an excellent opportunity to study this phenomenon in a silt soil since a well developed layer successfully impedes the seepage flow over the full length of the canal.[1] The total layer varies from about 1/2 to 1 inch thick and appears to contain layers of organic material as well as numerous visible organisms. The top 1 - 1-1/2 inches of the soil in the canal bottom could be divided into 4 fairly distinct layers. The top layer generally varied between 1/4 and 1/2 inches in thickness and could be separated readily from the soil immediately below it. The second layer varied in thickness from 1/4 inch to 3/4 inch and contained many small redish colored worms about 5-6 mm. long. This layer was perforated with worm holes, dead worms, and what appeared to be organic waste products. A third distinct layer was generally less than 1/4 inch thick and considerably darker in color. There were some worm holes but they rarely penetrated through the layer. The fourth layer was very faint and blended into the natural silt below the layer. The fourth layer contained some pockets of darker soil but no worm holes or organic material.

PARTICLE SIZE ANALYSIS

To investigate the particle size variation within the silt layer, a sample was taken at Station 133+50 in the Northside Canal and separated into 4 sublayers as previously explained. A mechanical analysis was made of each sublayer using the Bouyoucos Hydrometer Method[9] to determine the percentage of sand, silt and clay. The variation in the precentage of sand, silt, and clay within the layer is shown in Figure 18. The average distance from the top of the layer to the center of each sub-layer was plotted as the ordinate in each case. There is very little difference in the gradation of the top two sublayers; however, the third relatively thin layer shows a marked increase in the clay fraction with a resulting decrease in silt percentage. The bottom sublayer has a larger sand fraction than the top three sublayers.

Any attempt to explain the mechanics of the deposition of the total layer and the reasons for the dissimilarities among the sublayers would involve a considerable study of the past history of the flow conditions, suspended solids in the flowing water, maintenance performed on the canal and a thorough knowledge of the ecology and biology of micro-organisms in the prevailing environment.

HYDRAULIC CONDUCTIVITY MEASUREMENTS

Regardless of the mechanics of deposition or organic activity, it is possible to measure the hydraulic properties of the layer. In the fall of 1966, 40 undisturbed soil cores were obtained from the perimeter of the Northside Pumping Canal for future laboratory studies of the hydraulic properties of the soil. Soil cores, 3.25 inches in diameter and ranging in length from 6 to 11 inches were taken with a new sampler built and designed by personnel of the Snake River Conservation Research Center.[10] The sampler is very easy to use and secures a core sample in the silt loam soil with very little compaction or disturbance to the sides of the core, Figure 19. Each core was wrapped in Saran plastic immediately after it was taken to preserve the moisture and then transported from the field

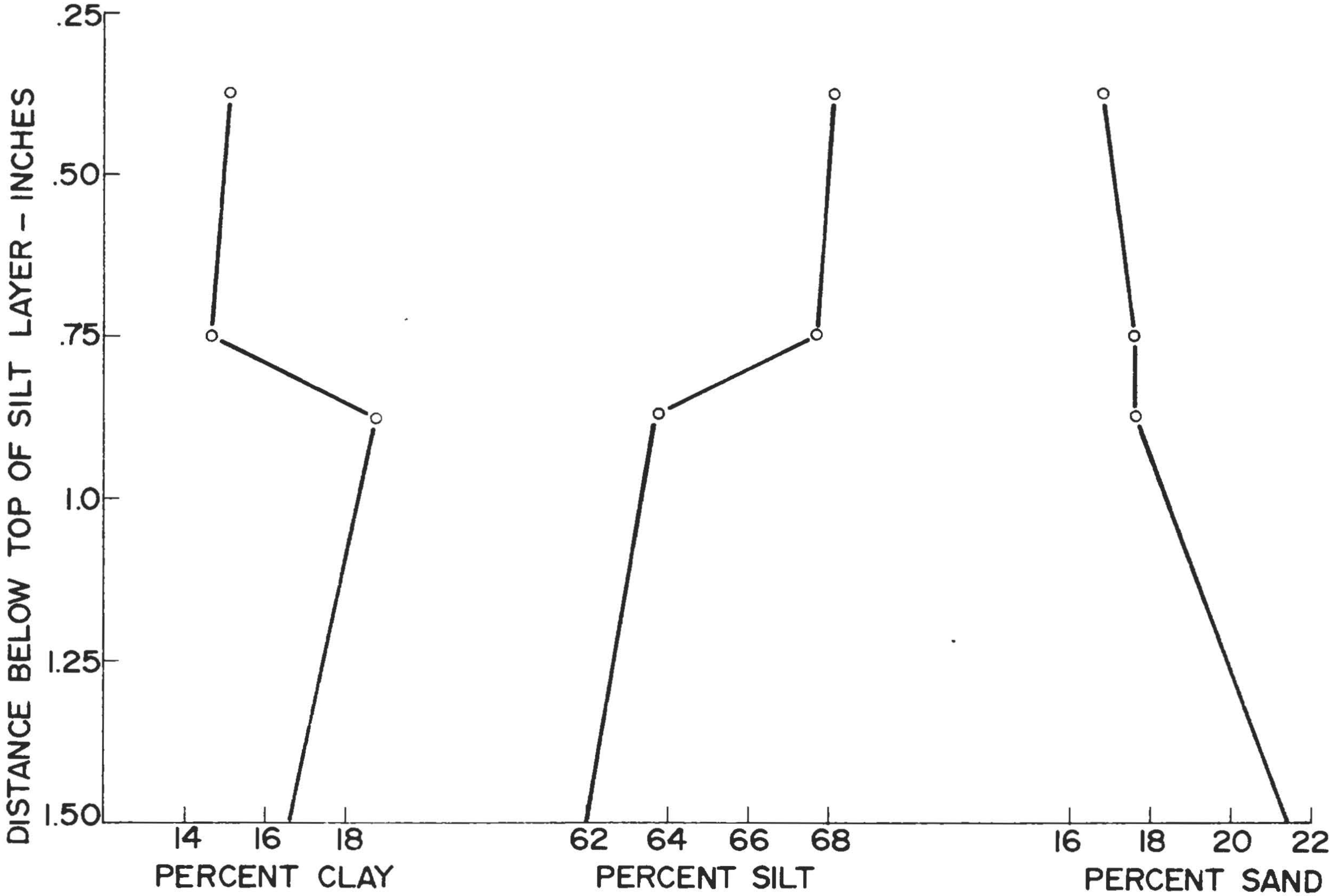
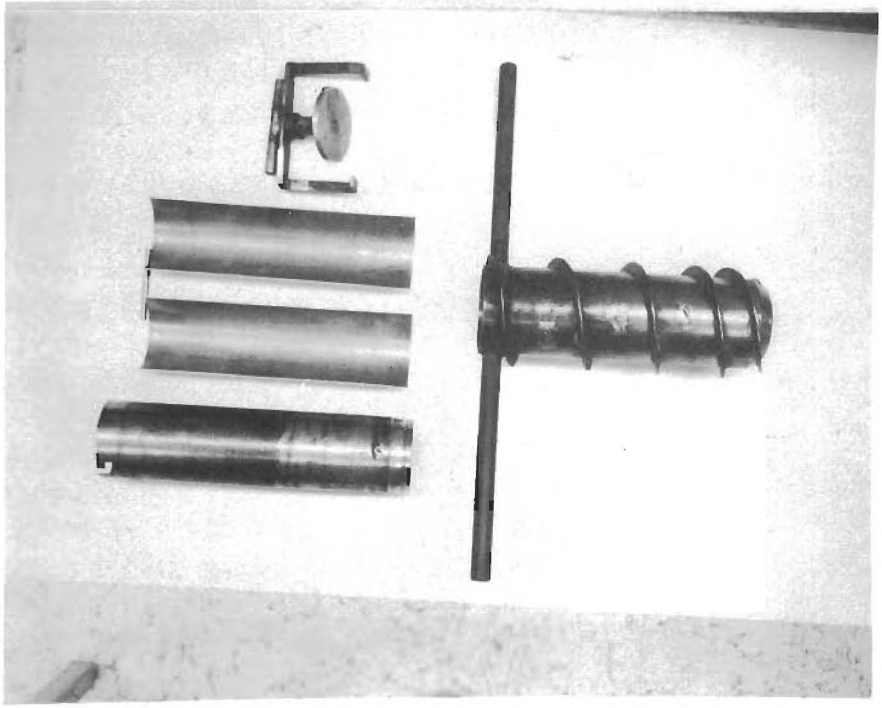


FIGURE 18 VARIATION OF PARTICLE SIZE WITH DEPTH - SILT LAYER
NORTHSIDE PUMPING CANAL 1966

in 1 quart ice cream containers. The Saran wrap seems to be effective in eliminating moisture loss from the sample. Cores which have been stored for seven months at room temperature appear to be in good condition.

A new technique was developed for casing the core samples for hydraulic conductivity tests.[11] The 3.25 inch diameter cores were encased in 55-mil, five-inch diameter irradiated polyolefin clear shrinkable tubing. End caps machined from acrylic plastic were placed on the ends of the sample and the tubing was then shrunk in place with an electric heat gun forming a protective conforming case around the soil and adhering to the end caps by pressure. It was necessary to also secure the tubing to the end caps with adjustable metal clamps to prevent leaks. The tubing is reasonably translucent and the core is quite visible through it, Figure 20A.

The cores which were encased were of a silt loam soil at approximate field capacity and no measurable deformation of the sample occurred during the process of shrinking the tubing around the sample. The soil cores must stand without support while the tubing is being shrunk. This would prohibit the use of shrinkable tubing on cohesionless soils or on cohesive soils with high moisture contents. The core samples were prepared in the laboratory, however, the procedure can be carried out in the field with the power for the heat gun being supplied by a portable generator. A heat gun of at least 1500 watts capacity is necessary to properly shrink the 4 inch dia.55 mil. tubing. The tubing is strong enough to provide support to the sample for handling and transporting. Pencil type 3/16 inch diameter tensiometers were installed in the soil cores by simply punching or drilling a hole through the tubing and sealing around the



A. Sampler parts: auger, split tube, and cutting cylinder.

B. Sampler in use on silt loam soil.
FIGURE 19 UNDISTURBED CORE SAMPLER
INVESTIGATION OF IMPEDING SILT LAYER
NORTHSIDE PUMPING CANAL

exposed opening with liquid rubber cement.

The laboratory facility for determining hydraulic conductivity included a 40 tube, 10 foot high water manometer, a Mariotte siphon apparatus for a constant head water supply, and a supply manifold to accommodate 8 soil columns. Figure 20B shows the laboratory set up with 4 columns operating.

To examine the variation in hydraulic conductivity within the soil profile beneath the Northside Pumping Canal an 18.5 cm long core obtained from the canal centerline at Station 148+57 was instrumented with 3 tensiometers and saturated hydraulic conductivity was determined. Facilities were not available at the time the column was set up for determining the unsaturated conductivity under pressure conditions similar to those found beneath the canal. The inflow was maintained at a constant head with a Mariotte siphon device and the flow through the column determined from time and volume measurements. Saturated hydraulic conductivity was determined from measured gradients and flow rates for 3 sections of the soil column. The top section from 0 to 4.3 cm included the silt layer which was about 2.7 mm thick. The second section, 4.3 to 8.8 cm, and the third section 8.8 to 13.6 cm appeared to be very similar and contained no visible evidence of organic activity.

The change in saturated hydraulic conductivity with depth in the profile was quite pronounced. In the top section. o to 4.3 cm, the initial conductivity measured after the column had been wet and flowing for 12 days was 20.2 mm/day; the second section 4.3 to 8.8 cm, had an initial conductivity of 74 mm/day and the third section, 8.8 to 13.6 cm. 177 mm/day. To determine the relative change in hydraulic conductivity with time, the column was kept flowing continuously and conductivity was measured periodically over a two month period. The variation of conductivity with time is shown in Figure 21. The conductivity is plotted as a percentage of the initial conductivity, K, measured on March 13, 1967. Hydraulic conductivity of the top section containing the impeding layer decreased quite rapidly to about 20 percent of its initial value whereas the conductivity of the lower soil although fluctuating over the two month period did not change more than 10 percent from the initial value.

COMPARISON OF FIELD AND LABORATORY CONDUCTIVITY MEASUREMENTS

Some interesting comparisons can be made between field and laboratory measurements of the hydraulic conductivity of the impeding layer. At Station 104 of the Northside Pumping Canal an estimate of the conductivity of the layer can be made by projecting the hydraulic gradient measured with tensiometers I & J at 1 and 2 feet below the canal to the bottom of the impeding layer. Using a seepage rate of 0.65 cfd as measured in the 1965 ponding tests [(1)] in the canal reach containing Station 104 and a layer thickness of 4 cm., the estimated conductivity of the impeding layer during the month of August 1967 ranged from 2.99 mm/day to 3.05 mm/day and averaged 3.02 mm/day. Similarly, at Station 132+75 during the ponding tests in October 1966 the estimated conductivity of the layer as measured by tensiometers A through F was 9.11, 3.44, and 3.78 mm/day or an average of 5.44 mm/day. Laboratory measurements of the saturated conductivity of the 4 cm. layer indicated a value of 3.74 mm/day after the rate of decrease in conductivity had apparently subsided, Figure 21. The results of conductivity determinations of the impeding layer from seepage meter tests show a value of 5.5 mm/day for the 1966 upper test section, 5.5 mm/day for the 1966 lower test section and 11.3 mm/day for the 1965 test section. Table 7 is a summary of hydraulic conductivity determinations by different methods.

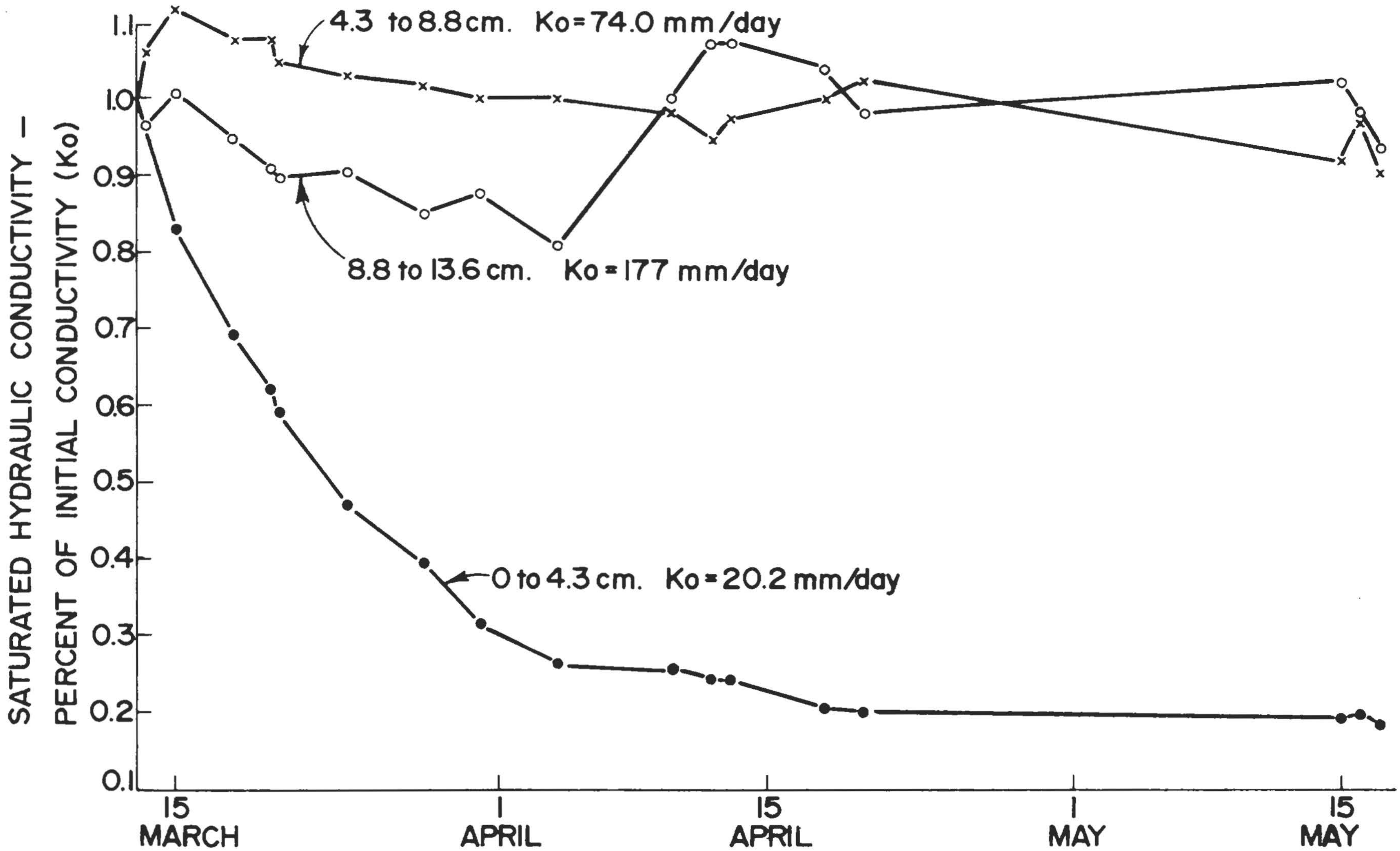


FIGURE 21 VARIATION OF SATURATED HYDRAULIC CONDUCTIVITY WITH TIME UNDISTURBED SOIL CORE — NORTHSIDE PUMPING CANAL 1966

TABLE 7

HYDRAULIC CONDUCTIVITY OF IMPEDING LAYER

	1965 Test Section mm/day	1966 Lower Section mm/day	1966 Upper Section mm/day
Estimate from tensiometer data.	3.0		5.4
Seepage meter	11.3	5.5	5.5
Laboratory			3.7

The comparisons between the saturated conductivity measured in the laboratory and field estimates in which the moisture flow beneath the layer is unsaturated are valid since the moisture content of the layer in the field is probably very near saturation and the conductivity of the layer does not vary appreciably from saturated conductivity. These comparisons are encouraging and suggest that estimates of the conductivity of the layer by use of seepage meter data may be reasonable. By estimating the conductivity from seepage meter tests, the changes throughout the season might be studied without resorting to ponding tests.

DISCUSSION

The change in conductivity of the top layer of the soil profile accounts for the gradual increase in soil moisture tension below the canal over the irrigation season. The water used for the column tests was obtained from a well in the vicinity of the laboratory and serves as the domestic water supply. No additional silt or suspended material was added to the water flowing through the columns so that the suspended solids content was much lower than the water in the Northside Pumping Canal. The decrease in conductivity cannot be attributed primarily to the deposition of suspended solids on the surface as is often assumed in an operating canal. This water supply is considerably higher in sodium than the water in the Northside Pumping Canal and it is possible that the decrease in hydraulic conductivity is related to the swelling of soil colloids in soil pores. McNeal[12] has shown that clay and fine silt fractions from certain soils exhibit swelling characteristics which significantly reduce hydraulic conductivity upon percolation of high Na, low salt solutions. Considerable study of the effects of solution composition on the hydraulic conductivity of the silt layer in the Northside Pumping Canal are necessary to determine the contribution of this effect to the seasonal decrease in conductivity observed both in the laboratory and in the field. Visual observation of the top layers in the profile indicate the presence of organisms and it is highly probable that the soil contains numerous types of microorganisms. It has been shown that the size distribution in a loessal soil can be altered by innoculation of the soil with various microorganisms.[13] The effect of microorganism activity on the hydraulic conductivity of soils has been documented [14] and suggests that the reduction of conductivity may be attributed partly to the dispersion of aggregates due to microbial attack and to the clogging of pores by products produced by microbial activity.

CONCLUSIONS

Ponding tests in 1966 on the one-half mile section of the Northside Pumping Canal indicated an operating seepage rate of 0.75 cfd or about .15 cfd higher than the average rate measured on an adjacent one-mile section in 1965. The higher rate is probably caused by sloping operations on the canal banks which removed an impeding silt layer.

The seepage meter developed by the Agricultural Research Service consistently indicates seepage rates about 30 percent higher than measured ponded rates. It is not known whether these differences in seepage rate are caused by the differences in actual seepage patterns from the canal during seepage meter testing and ponding tests or whether individual seepage meter tests may consistently measure too high. The difficulty and sometimes impossibility of predicting operating losses from seepage meter tests performed with the ARS meter at low depth negates the advantages of ease of operation and apparent accuracy. It is possible to estimate statistically the number of seepage meter tests required for a selected confidence level; however, results of over 900 seepage meter tests indicates that the numbers of tests required for reasonable confidence levels may be too large for practical applications.

The installation of tensiometers in the bottom of piezometer tubes for measuring soil moisture tensions below an operating canal is both more convenient and produces more reliable results than the installation in the side of pits excavated in the canal bottom. The greatest advantage of the piezometer type installation over pit installation is that tensiometers can be installed without draining the canal; however, it is difficult to install tensiometers in piezometers at shallow depths in the soil.

The sealing of the canal bottom during the irrigation season is apparent from changes in the inflow-outflow loss rates and a gradual increase in soil moisture tension below the canal. The changes in soil moisture tension are caused by changes in the hydraulic conductivity of an impeding layer and indicate significant changes in the unsaturated conductivity of the soil with corresponding changes in seepage rates.

A new coring device for obtaining an undisturbed core was used successfully to obtain soil samples for laboratory studies of the hydraulic conductivity of the natural soil and of the impeding silt layer on the canal bottom. Shrinkable electric insulating tubing works very satisfactorily for encasing the soil cores for laboratory tests. The impeding silt layer which has developed on the Northside Pumping Canal has a saturated hydraulic conductivity of about 1/50 of the conductivity of underlying silt soil.

A decrease in the conductivity of the impeding layer to about 20 percent of the initial value was measured over a two-month period in the laboratory. The change in conductivity of the layer is most likely caused by a combination of the clogging of soil pores by finer sediments and the activity of microorganisms.

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APPENDIX 1

COMPUTER PROGRAM

ANALYSIS OF SEEPAGE METER TESTS

The method for graphical analysis of ARS falling-head seepage meter tests is given by Bouwer.[2,3] However, the procedure is time consuming and can be performed mathematically using a computer program. Input for the program, written for the IBM 1620 computer, consists of the timed readings of the manometer and a constant which depends on the diameter of the seepage meter bell and falling head reservoir. The computer determines the equations of the least squares polynomial fit on both curves of manometer vs. time, solves the equations simultaneously to determine the point of intersection and then determines $\overline{H}$ by summing the absolute values of the first derivates of the curves at the point of intersection. This program was used successively for seepage meter tests in 1965 and 1966 and the use of second degree polynomial curves is adequate.

Seepage rate is computed from the equation:

$$I_s = \frac{R_v^2}{R_c^2} \overline{H}$$

R_c = radius of the seepage meter bell.

R_v = radius of the falling level reservoir.

If manometer readings are in centimeters and time in seconds, then computer output for $\overline{H}$ is in cm/sec.

$$\text{Seepage } I_s = \frac{R_v^2}{R_c^2} \overline{H}$$

to get I_s in feet/day, $I_s = \dfrac{R_v^2}{R_c^2} \quad \dfrac{86400}{30.48} \overline{H}$

$$= SK \ \overline{H}$$

SK = seepage coefficient for computer input and depends on reservoir diameter and meter diameter. Diameter of seepage bell is 9.803". Radius of seepage bell = 4.901 inches = 12.45 cm.

DESCRIPTION OF RESERVOIR		SK
2.5' King manometer	Dia. = .2136"	1.346
5' King manometer	Dia. = .2166"	1.321
1.657" well		80.99
1.657" well with 1.513" insert (De = .6756")		13.46
1.657" well with 1.024" insert (De = 1.303")		50.06
1.921" well		108.9
1.921" well with 1.513" insert (De = 1.184")		41.33
1.921: well with 1.024" insert (De = 1.625")		77.92
Black manometer (Temple)	Dia. .246"	1.785
1.75 well with 1.25 insert	De = 1.24	44.20

Statement No.	Description
11	Variable TEST is alphameric Test Number Code. SK is a coefficient depending on reservoir diameter used and input units. QINC is the time increment between successive printouts in tabular curve output. PL1, PL2, PL3 - alphameric location code.
60+1	X(1,1) and Y(1,1) are successive tensiometer and manometer readings on the seepage meter leg of the manometer. The units are seconds and centimeters. However, if different units are used, SK can be changed to keep the computer seepage in cfd. X(2,1) and Y(2,1) are successive tensiometer and manometer readings on the free leg of the manometer.

TYPICAL INPUT AND OUTPUT DATA

SEEPAGE METER PROGRAM

INPUT

```
1-7        120          50.06 N SIDE CANAL 1966
   0          36.6          0     33.7
  30          36.3         30     33.9
  60          36.1         60     34.1
  90          35.9         90     34.2
 150          35.4        150     34.6
 360          34.1        360     36.0
 420          33.5        420     36.2
 480          33.2        480     36.6
```

OUTPUT

SEEPAGE MEASUREMENTS USING VARIABLE - HEAD SEEPAGE METERS

TEST NUMBER 1-7 N SIDE CANAL 1966

CURVE NUMBER 1 Y EQUALS 36.5560 PLUS -.0075T PLUS, 1.087E-06T SQUARED

CURVE NUMBER 2 Y EQUALS 33.6899 PLUS .0063T PLUS, -6.415E-07T SQUARED

CURVE NUMBER 1		CURVE NUMBER 2	
T VALUE	SM VALUE	T VALUE	FW VALUE
.0	36.55	0.0	33.68
120.0	35.66	120.0	34.44
240.0	34.81	240.0	35.18
360.0	33.98	360.0	35.90
480.0	33.19	480.0	36.60

HBAR EQUALS, .0131 SEEPAGE EQUALS .659FT PER DAY

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C       ANALYSIS OF ARS SEEPAGE METER TESTS
        FOLLOW DATA CARDS WITH A BLANK TRAILER CARD
        DIMENSION X (2,20), Y (2,20), A (4,4), C (2,4), SUMX (4), SUMXY (4), NPT (2),
       1CC (3), XE (2)
      1 FORMAT (A6,F6.0,F8.0,3A6)
      2 FORMAT (4F8.0)
      30FORMAT (/2X, 12HCURVE NUMBER, I2, 9H Y EQUALS,F8.4,6H PLUS ,F8.4,
        18HT PLUS ,E 10.3,9HT SQUARED)
      4 FORMAT(/3X, 11HTEST NUMBER,A6,2X,3H    ,3A6)
      5 FORMAT(6X, 14HCURVE NUMBER 1,  12X, 14HCURVE NUMBER 2 /)
      6 FORMAT(6X,7HT VALUE,6X,  8HSM VALUE,5X,7HT VALUE,5X,8HFW VALUE /)
      7 FORMAT(4X,F8.1,5X,F8.2,5X,F8.1,5X,F8.2)
      80FORMAT(/2X, 12HHBAR EQUALS,F6.4, 16H SEEPAGE EQUALS ,F6.3, 10HFT PER
        1DAY/)
      9 FORMAT(2X,78HSEEPAGE MEASUREMENTS USING VARIABLE-HEAD SEEPAGE METER
       1RS)
        K9=0
     10 PUNCH 9
     11 READ 1,  TEST,QINC,SK,PL1,PL2,PL3,
        PUNCH 4,  TEST,PL1,PL2,PL3
        K9=K9+1
        J=0
        K=0
        I=0
     60 I=I+1
        READ 2, X(1,1)Y(1,1), X(2,1),Y(2,1)
        IF(Y(1,1))71,70,71
     70 K=K+1
     71 IF(Y(2,1)) 73,72,73
     72 J=J+1
     73 IF(K-1)  75,74,75
     74 NPT(1)=I-1
     75 IF(J-1) 77,76,77
     76 NPT(2)=I-1
     77 IF(J-1)  79,78,78
     78 IF(K-1) 79,80,80
     80 GO TO 12
     79 GO TO 60
     12 DO 101 L= 1,2
        MPT = NPT(L)
        XE(L) = X(L,MPT)
        SUMY = 0.0
        DO 31 I =1,MPT
        SUMY = SUMY + Y(L,I)
        DO 31 J =1,4
        IF(I-1)  41,41,42
     41 SUMX(J)    =0.0
        SUMXY(J)   =0.0
```

```
 42 SUMX(J) = SUMX(J)   +X(L,I)**J
    IF(J-2)  105,105,31
105 SUMXY(J)   = SUMXY(J)   +X(L,I)**J  *Y(L,I)
 31 CONTINUE
    PTN = MPT
    DO 32  I =1,3
    IF(I-1)  106,106,43
106 A(I,4) = SUMXY(I-1)
    TO TO 44
 43 A(I,4) = SUMXY(I-1)
 44 DO 32 J = 1,3
    IF(I-1) 107, 107,45
107 IF(J-1) 108, 108,45
108 A(I,J) = PTN
    GO TO 32
 45 M =J + I -2
    A(I,J) = SUMX(M)
 32 CONTINUE
    DO 33 I = 1,2
    M = I + 1
    DO 33  N = M,3
    IF(ABSF(A(I,I))-ABSF(A(N,I)))  109,33,33
109 DO 34 J =1, 4
    R = A(I,J)
    A(I,J) = A(N,J)
    A(N,J) = R
 34 CONTINUE
 33 CONTINUE
    DO 35  IP = 1,3
    R = A(IP,IP)
    DO 36 J = IP, 4
 36 A(IP,J)  =A(IP,J) /R
    DO 35  I =1, 3
    IF(I-IP)  110,35,110
110 R = A(I,IP)
    DO 37  J = IP, 4
 37 A(I,J) = A(I,J) -R * A(IP,J)
 35 CONTINUE
    DO 38 I =1, 3
 38 C(L,I) = A(I,4)
    CO1 = C(L,1)
    CO2 = C(L,2)
    CO3 = C(L,3)
    PUNCH 3 ,1 ,CO1, CO2, CO3
101 CONTINUE
    PUNCH 5
    PUNCH 6
    XC=XE(1)
    XC =0.0
 51 SUMYC1 =0.0
    SUMYC2=0.0
    DO 40 I =1,3
    SUMYC1= SUMYC1+ C(1,I) * XC**(I-1)
```

```
    SUMYC2= SUMYC2 +C(2,I) * XC**(I-1)
 40 CONTINUE
    PUNCH 7, XC,SUMYC1, XC, SUMYC2
    XC =XC + QINC
    IF(XE(1)-XE(2)) 200,201,201
200 IF(XC-XE(2)) 51,51, 199
201 IF(XC-XE(1)) 51,51, 199
199 DO 61 I = 1,3
 61 CC(I) = C(2,I) - C(1,I)
    TEMP1 = 2.0 * CC(3)
    TEMP2=SQRTF(CC(2)**2.-4.0*CC(3)*CC91))
    TI = (-CC(2) - TEMP2)/TEMP L
    IF(TI-0.)  113,113,112
113 TI = (-CC(2) + TEMP2)/TEMP 1
112 DT=ABSF(2.0*CC(3)*TI+CC(2))
    SEEP = SK * DT
    PUNCH 8,  DT,  SEEP
    IF(K9-2)  11,10,10
300 PAUSE
    END
```