

AD-A182 106

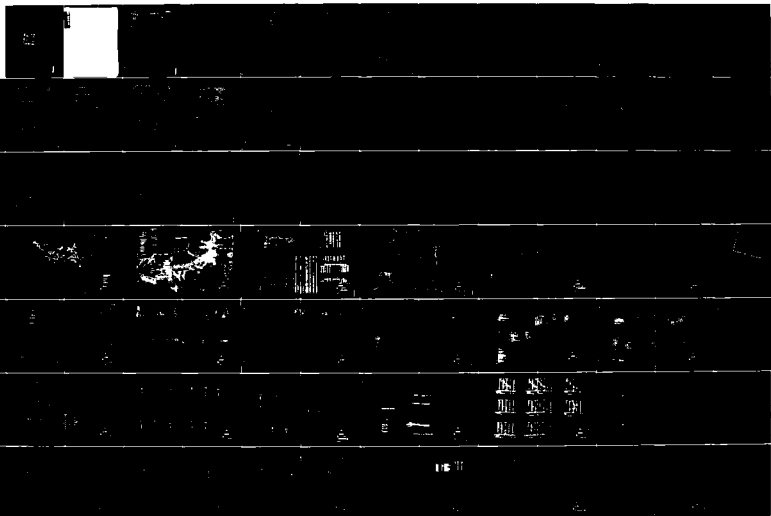
MULTIPLE-PURPOSE PROJECT: PLATTE RIVER BASIN LITTLE
PLATTE RIVER MISSOURI (U) CORPS OF ENGINEERS KANSAS
CITY MO KANSAS CITY DISTRICT F C WALBERG ET AL. MAR 87

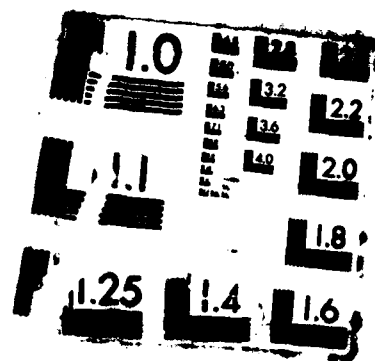
1/3

UNCLASSIFIED

F/G 13/2

NL





UNCLASSIFIED

(SUPPLEMENT NO. 1)

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Appendix V to the Smithville Lake O&M Manual	2. GOVT ACCESSION NO. ADA182106	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) MULTIPLE-PURPOSE PROJECT PLATTE RIVER BASIN, LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE OPERATION AND MAINTENANCE MANUAL, APPENDIX V EMBANKMENT CRITERIA & PERFORMANCE REPORT, SUPPLEMENT No. 1		5. TYPE OF REPORT & PERIOD COVERED Embankment criteria and Performance Report from April 1983 to Early 1985
7. AUTHOR(s) Mr. Francke C. Walberg-Chief, Dams & Found. Sec. Mr. John D. Holm-Project Engineer Mr. Mark S. George-Project Engineer		6. PERFORMING ORG. REPORT NUMBER
9. PERFORMING ORGANIZATION NAME AND ADDRESS Dams and Foundations Section (MRKED-FD) Foundations and Materials Branch (MRKED-F) Kansas City District, U.S. Army Corps of Engineers 601 E. 12th Street, Kansas City, Missouri 64106		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS PB-2b and ER 1110-2-1901 (31 DEC 81)
11. CONTROLLING OFFICE NAME AND ADDRESS Estimates and Specifications Section (MRKED-DE) Design Branch (MRKED-D) Kansas City District, U.S. Army Corps of Engineers 601 E. 12th Street, Kansas City, Missouri 64106		12. REPORT DATE March 1987
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)		13. NUMBER OF PAGES 178
		15. SECURITY CLASS. (of this report) UNCLASSIFIED
16. DISTRIBUTION STATEMENT (of this Report) Distribution in Accordance with ER 1110-2-1901, Paragraph 8, dated 31 December 1981 Distribution Statement "A"		
17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)		
18. SUPPLEMENTARY NOTES Control Determination: This supplement No. 1 to the titled report conforms to the intent of exempt report categories as set forth in AR 335-15 and under the paragraph 7-2y of the AR.		
19. KEY WORDS (Continue on reverse side if necessary and identify by block number)		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The purpose of this Supplement No. 1 to the Smithville Lake, Embankment Criteria and performance report is to present the general plan used to improve the stability of the dam at high pool levels, which includes (1) investigations performed, (2) results tabulated, (3) remedial measures installed and (4) performance to date. to D. V. I.		

DD FORM 1 JAN 73 1473

EDITION OF 1 NOV 65 IS OBSOLETE

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

OPERATION AND MAINTENANCE MANUAL

SMITHVILLE, LAKE
LITTLE PLATTE RIVER, MISSOURI

APPENDIX V

EMBANKMENT CRITERIA AND
PERFORMANCE REPORT

SUPPLEMENT NO. 1

MARCH 1987

DEPARTMENT OF THE ARMY
KANSAS CITY DISTRICT, CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Accession For	
NTIS CRA&I	☒
DTIC TAB	☐
Unannounced	☐
Justification	
By	
Distribution/	
Availability Codes	
Dist	Avail and/or Special
A-1	



OPERATION AND MAINTENANCE MANUAL
SMITHVILLE LAKE
LITTLE PLATTE RIVER, MISSOURI

APPENDIX V

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

SUPPLEMENT NO. 1

LEFT ABUTMENT REMEDIAL MEASURES

Form 1473 — CONTENTS :

<u>Paragraph</u>	<u>Title</u>	<u>Page</u>
<u>INTRODUCTION</u>		
1	Purpose of Supplement	V-1-1
2	Impoundment History	V-1-1
3	Piezometric Levels and Seepage	V-1-1
4	Investigations and Actions	V-1-1
	a. Initial	V-1-1
	b. Subsequent	V-1-2
	c. Final	V-1-2
<u>GEOLOGY</u> ;		
5	General	V-1-2
6	Glacial History	V-1-2
7	Overburden	V-1-3
8	Bedrock	V-1-3
9	Characterization of Left Abutment	
	Foundation	V-1-4
10	Seepage Behavior	V-1-4
<u>SEEPAGE CONTROL</u> ;		
11	General	V-1-4
12	Bowers Seep Area	V-1-4
13	Downstream Seep Area	V-1-4
14	Dike Seep Area	V-1-5

Paragraph

Title

Page

→ STABILITY ANALYSIS. ←

15	Design Memorandum No. 10 "Soil Data and Embankment Design" Stability Studies	V-1-5
16	Stability Review of DM No. 10	V-1-5
17	Left Abutment Stability Investigation	V-1-6
18	Shale Seam in the Raytown Limestone	V-1-6
19	Raytown Limestone	V-1-6
20	Lower Foundation Overburden Material	V-1-6
21	Material Strength Approaches	V-1-7
22	Final Material Strength Approach	V-1-7
23	Estimation of Piezometric Surface	V-1-7
24	Method of Analyzing Stability	V-1-7
25	Recommended Remedial Measures	V-1-8
26	Current Stability Investigation	V-1-8
27	Method of Analyzing Stability	V-1-8
28	Determination of the Critical Section	V-1-8
29	Profiling the Critical Section	V-1-8
30	Shear Strengths of Materials	V-1-9
31	Shear Strength Envelopes	V-1-9
32	Determination of Water Pressures	V-1-9
33	Unit Weights of Materials	V-1-9
34	Cases Considered for the Stability Analyses	V-1-9
35	Partial Pool Case	V-1-9
36	Rapid Drawdown Case	V-1-10
37	Steady Seepage Case	V-1-10

FIGURES

FIGURE NO.

TITLE

1	Pool EL. vs PZ Level - P-106-4
2	Pool EL. vs PZ Level - P-106-5
3	Pool EL. vs PZ Level - P-108-2
4	Pool EL. vs PZ Level - P-109-1
5	Pool EL. vs PZ Level - P-110-3
6	Pool EL. vs PZ Level - P-110-4
7	Pool EL. vs PZ Level - P-110-6
8	Pool EL. vs PZ Level - P-110-7
9	Pool EL. vs PZ Level - P-110-8
10	Pool EL. vs PZ Level - P-110-9
11	Pool EL. vs PZ Level - P-112-1
12	Pool EL. vs PZ Level - P-112-3
13	Pool EL. vs RW-1 Flows
14	Pool EL. vs RW-2 Flows
15	Pool EL. vs RW-3 Flows
16	Pool EL. vs RW-4 Flows
17	Pool EL. vs RW-5 Flows
18	Pool EL. vs RW-6 Flows
19	Pool EL. vs RW-7 Flows
20	Pool EL. vs RW-8 Flows
21	Pool EL. vs RW-11 Flows
22	Pool EL. vs Total Well Flow (RW-3 - 11)

DRAWINGS

DRAWING NO.

TITLE

FILE

1	Location and Vicinity Map	RP-3-1751
2	Embankment Plan	RP-3-1752
3	Observation Devices Plan View and Schedule	RP-3-1753
4	Observation Devices, Plan View of Left Abutment	RP-3-1754
5	Inset From Preceding Plate, Plan of Exploration & Profile of PZ's on Left Abutment	RP-3-1755
6	Instrumentation Cross Sections	RP-3-1756
7	Bovers Seep Area	RP-3-1757
8	Geologic Column and Legend, Detached Borings and Typical Installation Diagrams	RP-3-1758
9	Left Abutment Dam Axis Profile; Embankment Section - Station 110+00	RP-3-1759
10	Main Dike Section - Station 20+00	RP-3-1760
11	Profiles - Left Abutment; Bovers Seep Area	RP-3-1761
12	Logs of Explorations and Legend	RP-3-1762
13	Logs of Explorations	RP-3-1763

<u>DRAWING NO.</u>	<u>TITLE</u>	<u>FILE #</u>
14	Relief Drains - Logs of Explorations	RP-3-1764
15	Profile Along Relief Wells and Logs of Explorations	RP-3-1765
16	Installation Details of RW-1 thru RW-11	RP-3-1766
17	Installation Details of RW-12 & RW-13, Pumped Wells and 2-inch and 4-inch Relief Drains	RP-3-1767
18	Details of Collector Ditch	RP-3-1768
19	Gradation Curves Left Abutment Borings	RP-3-1769
20	Test Data, Left Abutment, R Tests, Borings UC-527 (P-109-1), C-533 (P-112-2)	RP-3-1770
21	Test Data, Left Abutment, Residual S Tests Boring C-527 (P-109-1), S Test Boring C-532 (P-110-10)	RP-3-1771
22	Test Data, Left Abutment, Residual S Tests Boring C-526 (P-110-9)	RP-3-1772
23	Test Data, Left Abutment, Residual S Tests Borings UC-525 (P-108-1), C-528 (P-109-2)	RP-3-1773
24	Test Data, Left Abutment, Residual S Tests Borings UC-525 (P-108-1)	RP-3-1774
25	Test Data, Left Abutment, Residual S Tests Borings C-529 (P-108-2), I-108-1 And Summary of Tests	RP-3-1775
26	Piezometric Level at Sta 110+00 Before & After Wells Installed, at Record Pool, and at Projected Spillway Crest	RP-3-1776
27	Projected Piezometric Levels from Piezometer Data Obtained After Installation of Relief Wells - STA 110+00	RP-3-1777
28	Shear Strengths Parameters (Left Abutment)	RP-3-1778
29	Partial Pool Stability Analysis (Left Abutment)	RP-3-1779
30	Rapid Drawdown and Steady Seepage Stability Analyses (Left Abutment)	RP-3-1780

APPENDICES

<u>Appendix No.</u>	<u>Title</u>
A	Left Abutment Stability Report
B	Left Abutment Seepage Report

OPERATION AND MAINTENANCE MANUAL
SMITHVILLE LAKE
LITTLE PLATTE RIVER, MISSOURI

APPENDIX V

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

SUPPLEMENT NO. 1

LEFT ABUTMENT REMEDIAL MEASURES

INTRODUCTION

1. Purpose of Supplement. The purpose of this supplement is to present the general plan used to improve the stability of the dam at high pool levels. A brief description is included of the investigations performed, results, remedial measures installed, and performance to date. The results of the stability analyses conducted on the left abutment based on project performance since installation of the remedial measures is presented. Detailed descriptions of the field and laboratory investigations are found in the reports entitled 'Left Abutment Stability Report' and 'Left Abutment Seepage Report' contained in appendices A and B, respectively, of the supplement.

2. Impoundment history. Impoundment began in October 1979 but lake filling was delayed because of real estate acquisition problems. Multipurpose pool, at elevation 864.2, was first reached in June 1982. A pool at elevation 869.4 occurred in April 1983 and again in April 1984. The record high pool, at elevation 873.17, was reached on 16 and 17 October 1985. Flood control pool is at elevation 876.2, while spillway crest pool is at elevation 880.2.

3. Piezometric levels and seepage. During the fall of 1982, after the pool had attained multipurpose level, high piezometric response was noted in several of the piezometers downstream of the centerline. Comparisons between the observed and anticipated piezometric levels suggested the safety factor for the steady seepage case might be lower than the 1.6 shown in the embankment design memorandum. Close monitoring of the piezometric levels was recommended, particularly at station 110+00 where the piezometric level was about 3 feet above the ground surface. During April 1983 a seep area developed at the toe at station 110+00 with the pool at elevation 869.37, a record high pool at that time. Seepage was not enough to observe flowing quantities. In August 1983 an adjacent landowner, Roy Bowers, reported that a large wet area, located about 3,000 feet below the main dike, had developed on his property. Field reconnaissance revealed three general areas of seepage: (1) the downstream seep area at the toe of the left abutment; (2) the Bowers seep area; and (3) the dike seep area. These general areas are shown on plate 4. An investigation of seepage in the left abutment was then initiated. A brief overview of the investigation is presented below, a detailed description of these events are contained in appendices A and B.

4. Investigations and actions.

a. Initial. The initial investigation included the installation and monitoring of additional instrumentation devices and a review of the original design analysis. Fourteen piezometers and observation wells were installed in a line between the lake and the Bowers seep area and five piezometers were installed in the downstream seep area as shown on plates 5 and 7. Based upon

the observed data and a review of the original design stability analysis it was concluded that projected piezometric levels for the pool at spillway crest were higher, as much as 25 to 35 feet, than those assumed during design. Preliminary stability analysis indicated that the safety factor using the DM design strengths and the projected piezometric levels was well below the DM recommended 1.6.

b. Subsequent. Commencing in April 1984 a sequence of actions was begun to insure the safety of the structure while slope stability and seepage investigations were being conducted. A revised plan of lake releases was put into effect to reduce the probability of subjecting the dam to high pool levels and the frequency of monitoring the dam was increased for all pools above multipurpose. Because of immediate concerns regarding the stability of the dam at high pool levels an interim solution that would reduce piezometric levels by pumping from wells was implemented. After the wells were installed and operational the project was returned to a normal operating plan. Two inclinometers were installed in the most critical areas at stations 108+00 and 110+00 to monitor any movement. Ten relief drains were installed on the Bover's and adjacent Government properties to provide a measure of seepage control and to determine the effect on the piezometric surface. Four flowing test wells were subsequently installed at the toe of the embankment to evaluate whether pressure relief wells would provide a satisfactory permanent solution. During the installation of the wells and piezometers undisturbed sampling was being performed and a laboratory testing program being conducted. Of concern was the strength and continuity of a slickensided surface in a shale seam located near the top of bedrock. Appendices A and B contain additional detailed information regarding these investigations and actions.

c. Final. The installation of 13 pressure relief wells and a buried collector pipe in the left abutment was completed early in 1985. Plate 6 shows the location of these wells. With these wells controlling seepage and reducing uplift pressures the stability analysis had indicated that an adequate factor of safety would be obtained for the spillway crest steady seepage condition. A detailed description of the investigations regarding the stability analysis is contained in appendix A. Additional stability analysis performed after installation of the relief wells is presented in later paragraphs.

GEOLOGY

5. General. Smithville Lake is located near the southern limit of the Dissected Till Plains Section of the Central Lowlands Physiographic Province. Major topographic features are the maturely to submaturely developed valleys of the Little Platte River, Crows Creek, and Camp Branch. Drainage patterns typical of northern Missouri are developed on thick glacial deposits resulting in gently rolling topography. Bedrock exposures are not common but can occasionally be found along the bases of valley walls of major streams. Maximum relief in the area is about 160 feet.

6. Glacial history. Pleistocene glaciers extended into the northern region of Missouri approximately 750,000 years ago during the Kansan glacial episode and persisted for approximately 100,000 years. Glaciers may have also advanced into the area during the earlier Nebraskan episode. Both the Nebraskan and Kansan advances were from the north-northwest and are attributed

to the Iowa ice lobe from the Keewatin ice center in Canada. Since the same general regions were traversed during both episodes, the content of resultant drift materials is similar and difficult to distinguish. The southern limit of glaciation is generally recognized as being slightly south of, and approximately parallel to, the present course of the Missouri River.

Pleistocene ice sheets have been compared in size and extent to those of the Antarctic which have an average central thickness of about 6,500 feet. Estimated thicknesses of marginal masses are of the order of 1,600 feet. Glacial erosion was primarily by abrasion and quarrying whereby slabs of frozen ground were sheared from and dragged forward over nonfrozen ground. Magnitudes of erosion were dependent upon the thickness and velocity of the ice mass, the nature of materials incorporated into the basal ice, and the character of surfaces overridden. Glacial sediments include nonstratified till and, less frequently, fluvio-glacial deposits of stratified silts, sands, and gravels. Drift of variable thickness has been deposited upon essentially flat lying Pennsylvania bedrock and is the thickest in pre-Pleistocene topographic lows.

7. Overburden. Overburden in the vicinity of the dam is of three principal types; alluvium, glacial drift, and loess. Alluvium occupies the valleys of the Little Platte River and its tributaries and generally consists of lean and fat clays overlying clayey sands and sandy clays with minor amounts of basal gravel. Thicknesses range from 25 to 50 feet. Upland areas are deposits of glacial drift thinly mantled with loess. In the left abutment area, the drift ranges in thickness up to 85 feet and generally consists of 20 to 60 feet of till overlying 5 to 25 feet of coarser outwash sediments. Till, in general, is composed of unsorted, unconsolidated (geologically), nonstratified sediments deposited directly by and underneath glacial ice masses and consists of heterogeneous, random mixtures of clay, silt, sand, gravel, cobbles, and boulders. The overburden above approximately elevation 845 in the left abutment is predominantly lean clay glacial till with scattered gravel and cobbles and occasional isolated silty sand lenses. Below elevation 845, the material is much more heterogeneous with considerable lateral and vertical variation. Throughout most of the abutment area, the upper 11 to 20 feet of this lower unit is generally silt, however, silty clay or lean clay was encountered in some borings at this horizon. Below the silt zone, the material is coarser and consists of sand, gravel, and cobbles, generally with a significant amount of silt and clay. The coarser materials underlying the till are meltwater sediments deposited from advancing or retreating ice sheets. Loess overlying the till reaches a thickness up to 20 feet in the area. The maximum thicknesses occur on broad, gently sloping, interstream divides where erosion has been minimal.

8. Bedrock. Near surface bedrock strata are of the Pennsylvanian System, Lansing and Kansas City Groups and consist of alternating beds of shale and limestone. A geologic column for the left abutment is shown on plate 8. The essentially horizontal configuration of the left abutment bedrock surface is the result of a pre-Pleistocene stream channel trending generally east-west through the abutment. It is one of two major channels mapped in the reservoir area which are part of the ancestral Missouri River drainage system prior to the advance of Pleistocene glaciers. The other is located several miles upstream of the dam in the reservoir area. As ice masses traversed the area, existing sediments were scoured away and near-surface bedrock strata subjected to shear forces induced by ice thrusts.

9. Characterization of the left abutment foundation. The investigations more clearly defined a basal layer of glacially deposited materials that form a pervious zone beneath the entire left abutment. The basal layer, consisting of a heterogeneous mixture of silt and silty or clayey sands, varies in thickness from 40 feet beneath the upland portion of the abutment to less than 10 feet near the valleys. Plates 9 and 11 contain profiles and sections through the abutment and the embankment showing these trends. The thicker portions tend to be more gravelly and cobbly whereas sand becomes more predominate as the layer thins. Similar materials crop out in the bluffs along Crows Creek upstream of the dam. The natural pervious, in combination with the overlying lean clays and silts and underlying tight bedrock, forms a confined aquifer system that is recharged from the lake. The seep areas are characterized by a decreasing thickness of low permeability material with pockets or lenses of more pervious material extending to or near ground surface.

10. Seepage behavior. With impoundment the exposed basal material along Crows Creek was submerged, permitting saturation of the basal layer and subjecting the confined aquifer system to a hydrostatic pressure head corresponding to the pool level. The piezometric pressure gradient is relatively flat through the abutment, except for initial entrance pressure losses and near the seeps. As piezometric levels in the basal layer near the downstream base of the abutment increased to above ground level a sufficient vertical gradient was created to force seepage upwards through the thinning confining layer. Because a relatively impervious alluvium blocks this flow seepage quantities are low. Up to a 50 percent piezometric response to changes in the pool have been observed. A more detailed discussion of the seepage is contained in appendix B.

SEEPAGE CONTROL

11. General. The installation of the relief drains and relief wells was completed by early 1985. The investigation into the cause, extent, and chosen remedial solutions to the seepage and stability problems was briefly summarized in the previous paragraphs. The appendices contain reports that address these matters in greater detail. The following paragraphs discuss the performance of the remedial actions to date.

12. Bovers seep area. A total of 10 relief drains, 7 on the Bovers property and 3 on the adjacent Government land, were installed as shown on plate 7. Boring logs for the 10 drains are shown on plate 14. A temporary collector system consisting of an above ground plastic pipe currently connects relief drains 4, 5, 5A, 5B, 6, 7, and 8 while drains 1, 2, and 3 discharge onto the ground. The relief system on the Bovers property is estimated to be flowing at a rate of 30 gpm and has succeeded in significantly drying up the area. Plate 11 shows a profile through the Bovers seep area with the piezometric levels prior to and following drain installation. A buried collector system is to be installed after acquisition of the property and will result in a further lowering of the piezometric surface.

13. Downstream seep area. A total of 13 relief wells were installed in the left abutment as shown on plate 5. Boring logs and installation details are shown on plates 15, 16, and 17. Relief wells 3 through 11 discharge into a buried collector system that exits through a Parshall measuring flume into

the toe ditch. The collector system consists of a fabric lined trench with gravel surrounding a slotted, corrugated plastic pipe. Relief wells 1 and 2 discharge at the toe of the dam below the flume while wells 12 and 13, located on the downstream slope, discharge into the pervious drain. Figures 13 through 21 show the relief well flow versus pool for wells 3 through 11 and figure 22 shows the total combined flow of these wells. The collector system flows at a rate of 25 to 30 gpm which is generally about 5 gpm higher than that of the wells. This is attributed to any or all of the following: (1) infiltration of ground water into the collector system; (2) error in the calibration of the flume; or (3) well flow measurement errors. Flow from wells 12 and 13 are too small to measure but they have caused a reduction of the piezometric level in the immediate area. Figures 1 through 12 relate the pool elevation to the piezometric level in the left abutment. The plots show a fairly well defined relationship between the pool and the piezometric level for both prior to and following well installation. The result is that the wells have performed as anticipated and have effectively reduced the piezometric surface in the left abutment resulting in an improvement in the overall stability of the embankment. Periodic Inspection Report #5 contains detailed piezometric data obtained over the life of the project. Plate 26 shows the projected piezometric levels with and without the wells, at the record high pool, and projected levels at spillway crest.

14. Dike seep area. The projected piezometric levels at higher pools are above ground level, however, the thickness of the confining layer should prevent excessive seepage. No remedial measures were deemed necessary although some surface drainage may facilitate maintenance of the area for surveillance purposes.

STABILITY ANALYSIS

15. Design Memorandum No. 10 "Soil Data and Embankment Design" stability studies. Investigations for the DM included stability analysis for the left abutment steady seepage, rapid drawdown, and partial pool cases. Water pressures were computed to be hydrostatic below the saturation line. For the left abutment section the assumed saturation line under the downstream slope was in the foundation at elevation 825. High piezometric levels resulting from underseepage were not expected to be a problem "due to the thickness of the impervious foundation materials, the large amount of fines in the sands and gravels, and the scarcity of continuous pervious layers beneath the embankment." DM No. 10 plate A-37 summarizes design shear strengths and illustrates the critical shear surfaces.

16. Stability review of DM No. 10. An initial investigation of the left abutment stability was begun after seepage areas and high piezometric levels were observed. Additional piezometers were installed through the downstream slope and the original design stability analysis was reviewed. The investigation concluded that:

a. The piezometric level assumed for design in the left abutment was actually some 20 feet below the base of the horizontal pervious blanket in the area at station 110+00.

b. Projected piezometric levels for a spillway crest pool based on recorded data up to that time were about 25 to 35 feet above that assumed for design.

c. The higher than anticipated piezometric levels meant that available shear strength of the foundation shales become more critical.

d. The uppermost bedrock units in the left abutment have been exposed to erosional unloading and glacial loading. These factors, coupled with a nearly horizontal bedrock surface and essentially flat lying sedimentary rock units, suggested the possibility of the existence of a shear zone or zones near the bedrock surface.

e. Preliminary stability analysis conducted with the projected piezometric levels and Design Memorandum No. 10 design strengths showed the safety factor well below the desired 1.6 of the DM.

17. Left abutment stability investigation. Shale design strengths used in Design Memorandum No. 10 were obtained from samples of shale units of the right abutment, outlet works area, and valley. Additional sampling and testing for the left abutment was not done for the original design since the factor of safety for the left abutment appeared to be adequate. The shale shear strengths become more critical when higher than anticipated piezometric levels were recorded in the left abutment. Accordingly, a more comprehensive stability investigation was initiated. The investigation included additional sampling and testing of left abutment materials, particularly to determine if a weak zone or zones existed, a reanalysis of slope stability, and recommendations for remedial measures needed to insure the stability of the embankment. A detailed account of these results may be found in appendix A.

18. Shale seam in the Raytown limestone. Sampling efforts were initially directed towards obtaining samples for testing the Raytown limestone-Muncie Creek shale contact. However, core samples in the Raytown limestone revealed slickensided planes in a soft shale seam located in the lower part of the bed. The seam is about 0.4 to 0.5 feet thick, approximately 1.5 feet above the Muncie Creek contact, and is persistent throughout the abutment. Apparent shear planes were observed in every sample recovered from the shale seam. Several thin shale partings are present above the seam but not continuous. The shale seam rather than the Raytown-Muncie Creek contact was determined to be the material with the least resistance to shearing forces and is the critical material for stability considerations. Testing of the seam yielded a residual strength with $\tan \phi = 0.13$ ($\phi = 7.4^\circ$) and a peak strength with $c=250$ psf, $\tan \phi = 0.268$ ($\phi = 15^\circ$).

19. Raytown limestone. The Raytown limestone was assumed to have a vertical joint and was assigned a resisting shear strength of zero. This assumption is consistent with Design Memorandum No. 10. Higher strengths for the limestone were available but were not used in the analysis since it would require thin, partially weathered, jointed limestone to carry very large forces.

20. Lower foundation overburden material. Design strengths used in Design Memorandum No. 10 for left abutment overburden material were obtained from tests run on lean and fat clays. As noted previously additional exploratory borings through the left abutment foundation overburden consistently revealed a coarse-grained pervious layer present above bedrock. As a result, it was believed reasonable to increase the S strength of the pervious layer to $c=0.0$, $\tan \phi = 0.577$ ($\phi = 30^\circ$).

21. Material strength approaches. Stability analyses for Design Memorandum No. 10 and preliminary stability analyses for the investigation were conducted using two different strength approaches: (1) peak strengths were used along the entire critical shear surface, and (2) peak design strengths were used along the failure surface in the active and passive wedges, and the residual shear strength of the shale was used in the central block portion of the sliding surface. The use of the residual strength was considered to be overly conservative since it assumed the lowest possible strength. The use of peak strengths for the stability analysis was believed to be unconservative due to strain incompatibilities between a shear zone in the shale, the foundation overburden, and the embankment. With small strains the peak strength could be developed in the shale before the peak strengths could be developed in the embankment and overburden materials.

22. Final material strength approach. For the interim and final analyses of the investigation a third approach was considered which accounted for strain incompatibilities. Peak strengths were used for material in the active wedge portion of the critical shear surface and in the shale seam, but not in the passive wedge since relative large displacements would be required to develop full passive resistance. It was considered reasonable to use strengths in the passive wedge (foundation overburden) which correspond to 0.5 percent strain development. Strains somewhat larger than this were required to develop the peak strength in tests on the slickensided shale surface.

23. Estimation of piezometric surface. The final stability analysis of the investigation consisted of locating the most critical shear surface of the left abutment for the steady seepage case at spillway crest (elevation 880.2). Stability analyses were considered with and without pressure relief wells at the toe of the embankment. Piezometric surfaces were determined by projecting piezometer levels for a pool at spillway crest and subtracting drawdown levels observed after installation of the four test relief wells at the toe of the embankment near station 110+00.

24. Method of analyzing stability. The stability analysis was conducted with the hand wedge method in accordance with EM 1110-2-1902 (April 1970) and with a computer program, SLOPESR, developed at the University of California Berkeley. SLOPESR uses Spencer's procedure to calculate the factor of safety for specified noncircular shear surfaces. In all cases, the factor of safety by the hand wedge method was lower than those obtained by SLOPESR because the side force inclinations computed by SLOPESR were greater than that assumed in the hand wedge analysis. When the same side force inclination was used in both the hand wedge analysis and SLOPESR the computed factors of safety were similar. The results of the stability analyses performed at station 110+00 are as follows:

	Safety Factor	
	Hand Wedge	SLOPESR
Spillway Crest Pool (without relief wells)	1.25	1.47
Spillway Crest Pool (with relief wells)	1.30	1.53

25. Recommended remedial measures. As a result of the stability analysis, it was concluded that the installation of pressure relief wells at the toe of embankment would provide an adequate factor of safety for steady seepage conditions with the pool at spillway crest. Recommendations included the installation of additional pressure relief wells and a buried collector system.

26. Current stability investigation. The stability of the left abutment following installation of remedial measures, consisting of 13 relief wells and a collector system, has been reviewed. Uplift pressures in the lower foundation materials are projections based on piezometric levels observed since installation of remedial measures.

27. Method of analyzing stability. A computer program entitled UTEXAS2 (September 1985) was used to analyze the slopes for the stability investigation. The program allows the user to select either a circular or noncircular shear surface with or without a search option to locate the shear surface with the minimum factor of safety. Reservoir pools are represented as an external surface load. The program, though relatively new, has been manually checked by the hand wedge method several times to assure its accuracy. It was also checked against previous SLOPESR results. For this stability investigation, a noncircular shear surface with the search option was chosen. For each analysis the factor of safety against sliding was determined using Spencer's procedure.

28. Determination of the critical section. The first step of the stability investigation was to determine which cross-section of the left abutment would be the most critical. Cross-sections of stations 110+00, 111+00, 112+00, 113+00, and 114+00 were drawn from construction surveys, drill logs of exploratory borings, and piezometer and relief well installations. The slope height above natural ground decreased and the thickness of the passive wedge increased as the stations increased. The piezometric level does not increase significantly upstation. Therefore, based upon visual inspection, the cross-section at station 110+00 was determined to be the most critical.

29. Profiling the critical section. The embankment at station 110+00 consists of berm fill and compacted fill materials. Seepage through the embankment is intercepted by an inclined pervious wick which conducts it to the downstream toe of the embankment. Foundation overburden generally consists of two materials, finer grained upper overburden material and a lower coarser grained material. Bedrock under most of the embankment at station 110+00 is the Raytown limestone member which contains the soft shale seam. Raytown limestone is shown to extend 267 feet upstream from the centerline of the embankment although the scarcity of borings on the upstream side of the left abutment makes it difficult to determine the exact extent of the Raytown limestone. However, a drill log detailing the installation of piezometer P-110-2 (station 110+00, range 267 US) lists Muncie Creek shale as bedrock material; the Raytown limestone was not encountered. For the stability investigation, the Raytown limestone member containing the soft shale seam was conservatively assumed to terminate 267 feet upstream from the centerline of the embankment.

30. Shear strengths of materials. Shear strength parameters for the berm fill, compacted fill, and upper foundation overburden material were taken from Design Memorandum No. 10, "Soil Data and Embankment Design." Shear strength parameters for the lower foundation overburden (coarser material), the Raytown limestone, and the shale seam were obtained from the Left Abutment Stability Report (July 1984) and are referenced in appendix A.

31. Shear strength envelopes. For steady seepage and partial pool cases the shear strength parameters for the berm fill, compacted fill, and upper foundation overburden materials were defined by a nonlinear $(S, (R+S)/2)$ envelope. The minimum nonlinear (S, R) envelope was used to define the shear strengths of these materials for the rapid drawdown case. The shear strength parameters for the lower (coarse) foundation overburden material, the Raytown limestone, and the shale seam remained the same for each analyses. The peak "S" (drained) strength was used for all cases for the coarse foundation overburden material, the residual "S" strength for the shale seam.

32. Determination of water pressures. For steady seepage conditions, the pore pressures acting on materials comprising the left abutment were determined from two piezometric surfaces. The pore pressures acting on the berm fill and compacted fill materials were defined by a line of seepage through the embankment. The line is assumed to extend horizontally at the pool elevation through the embankment to the inclined pervious wick. The wick and the horizontal pervious blanket conduct the seepage to the downstream toe of the embankment. Pore pressures acting on the lower (coarser) foundation overburden material, the Raytown limestone, and the shale seam were determined from water levels measured in a line of piezometers located at station 110+00. The corresponding piezometric surface for each pool elevation was projected from recorded levels read in each piezometer after installation of remedial measures. Pore pressures acting on the upper foundation overburden material were calculated by interpolating and averaging pore pressure values from the line of seepage and from pressures in the lower foundation overburden material.

33. Unit weights of materials. The unit weights used for the embankment and foundation materials are shown on plate No. 28. Unit weights for the berm fill, compacted fill, and upper foundation overburden materials were considered to be moist above the assumed line of seepage and saturated below. Saturated unit weights were used for the lower coarser foundation overburden material. The Raytown limestone and the shale seam were given unit weights of 140 pcf.

34. Cases considered for the stability analyses. This stability investigation considered both the upstream and downstream slopes of the embankment at station 110+00. The downstream slope was analyzed for a steady seepage condition with the pool at spillway crest (El. 880.2). The upstream slope was analyzed for steady seepage partial pool conditions and for a rapid drawdown from spillway crest pool to multipurpose pool (El. 864.2).

35. Partial pool case. A wide range of pool elevations was considered for the partial pool case. For a "no" pool condition the saturation line was assumed to be horizontal at an elevation of 820 feet. This places the piezometric level at the top of the lower (coarser) foundation overburden material for most of the section at station 110+00. This elevation for the

saturation line was determined from recorded piezometer levels after the embankment was constructed to its final height but before a measured pool elevation was recorded. The highest pool elevation considered was spillway crest at an elevation of 880.2 feet. The most critical failure surface for the partial pool analysis occurred at a pool elevation of 845 feet with a corresponding factor of safety of 1.37. The critical shear surface and the computed factor of safety for each considered pool elevation is shown on plate No. 29.

36. Rapid drawdown case. For the rapid drawdown analysis the pool was assumed to drop from spillway crest (El. 880.2) to multipurpose pool (El. 864.2). During rapid drawdown it was assumed that the drawdown occurred instantaneously and that no pore pressure dissipation occurred during drawdown. Therefore, the line of seepage through the embankment intercepted the upstream slope of the embankment at multipurpose pool and followed the upstream slope of the embankment to spillway crest pool. At this point the line extended horizontally to the inclined pervious wick. Water pressures acting on the lower foundation overburden and bedrock materials corresponded to the projected piezometric surface for spillway crest. Water pressures acting on upper foundation overburden material were calculated by interpolating pressures from the assumed line of seepage and from the piezometric surface. As previously mentioned, the shear strengths for the berm fill, compacted fill, and upper overburden materials were determined by the composite (S,R) envelope. Shear strengths for the remainder of materials remained the same as for the other stability analyses. The minimum factor of safety for the rapid drawdown case was calculated to be 1.30. The critical shear surface is illustrated on plate No. 30.

37. Steady seepage case. The downstream slope stability was analysed for a steady seepage condition with the pool at spillway crest (El. 880.2). Assigning the residual strength to the shale seam was considered for the previous stability analysis but not included because it was believed to be an overly conservative approach. In this investigation the residual strength was assigned to the shale seam but the required factor of safety lowered. A minimum factor of safety equal to 1.0 was sought. For the steady seepage case the calculated factor of safety was 1.00. Previously using peak strengths for the shale, reduced strengths in the other zones for strain compatibility, an estimated piezometric reduction from the relief wells, a factor of safety of 1.30 was obtained. The critical shear surface is illustrated on plate No. 30.

FIGURES

FIGURE

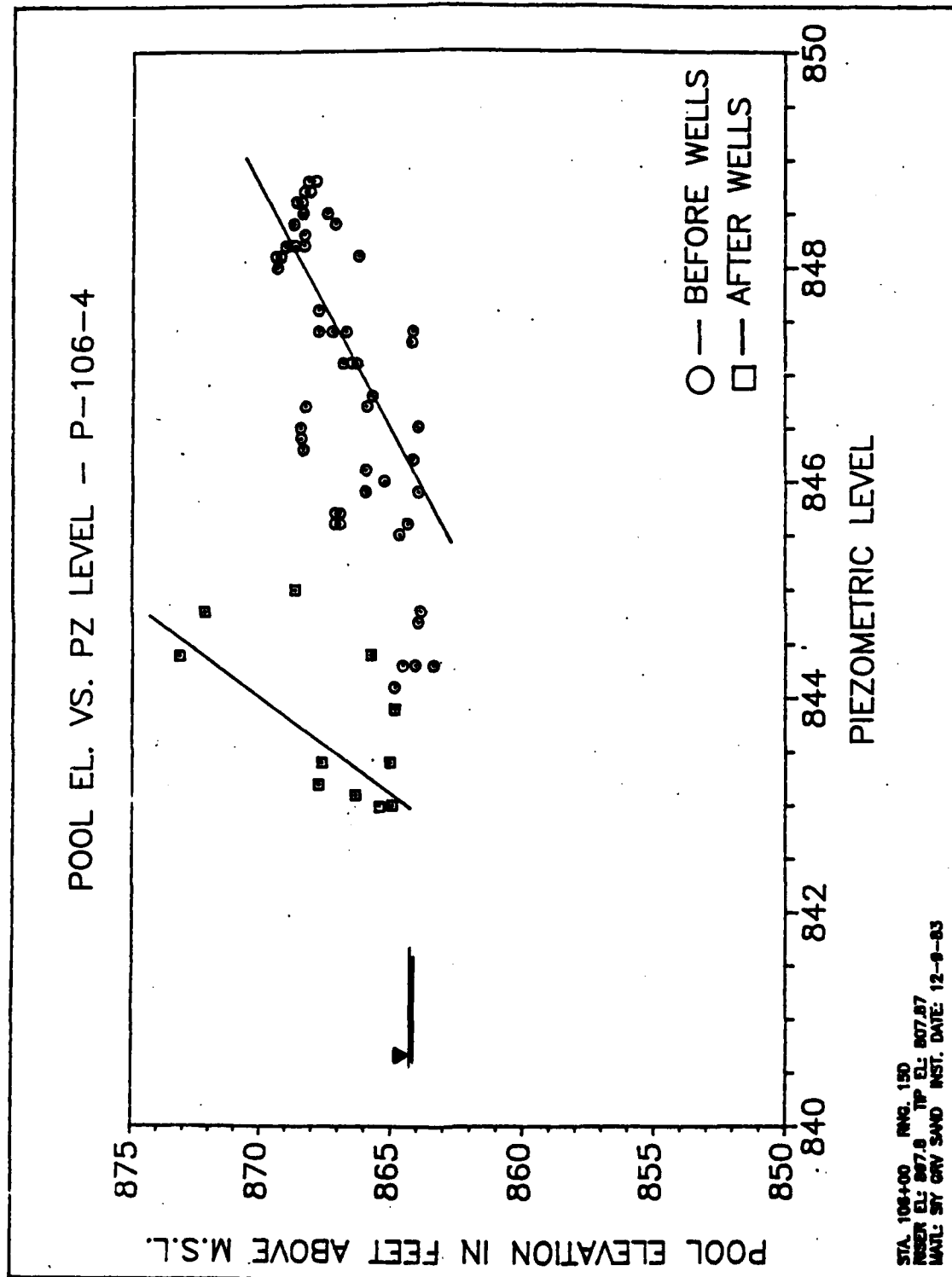
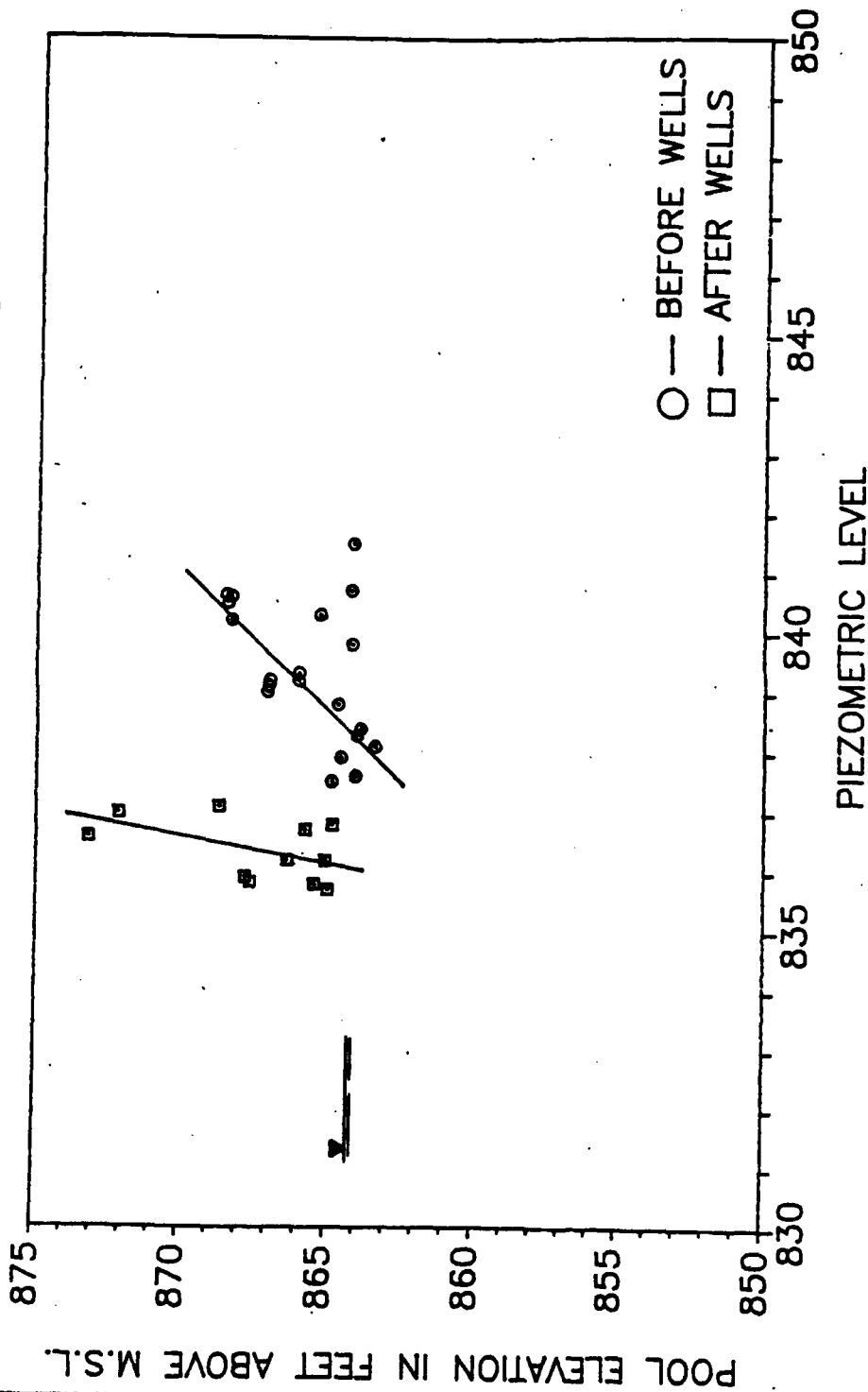


FIGURE NO. 1

Figure No. 1

POOL EL. VS. PZ LEVEL - P-106-5



STA. 108+00 RVC. 1500
 PZ EL. 859.84 TP EL. 806.7
 MATL: CLAY W/GRV INST. DATE: 17 MAY 84

FIGURE NO. 2

Figure No. 2

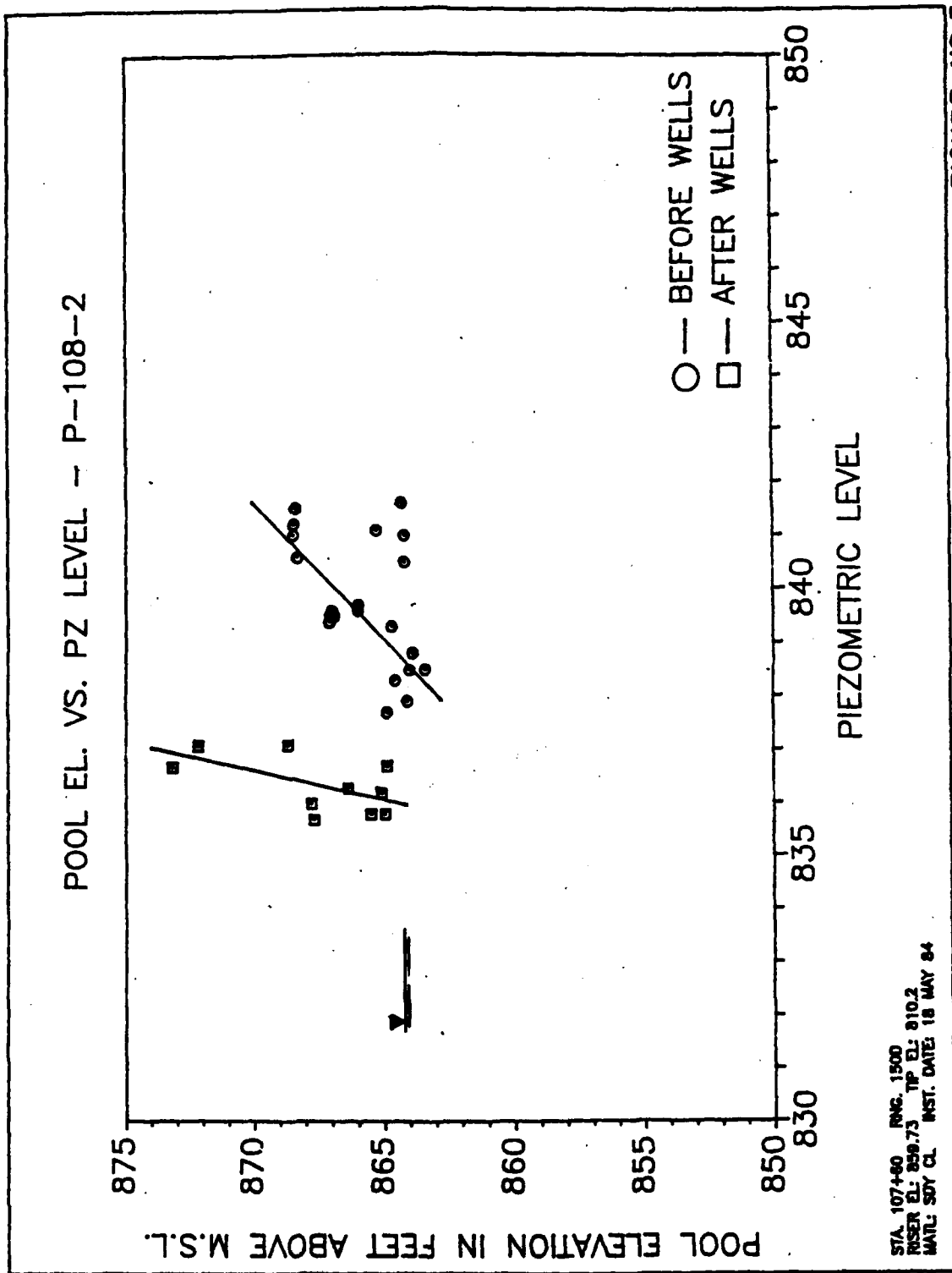


FIGURE NO. 3

Figure No. 3

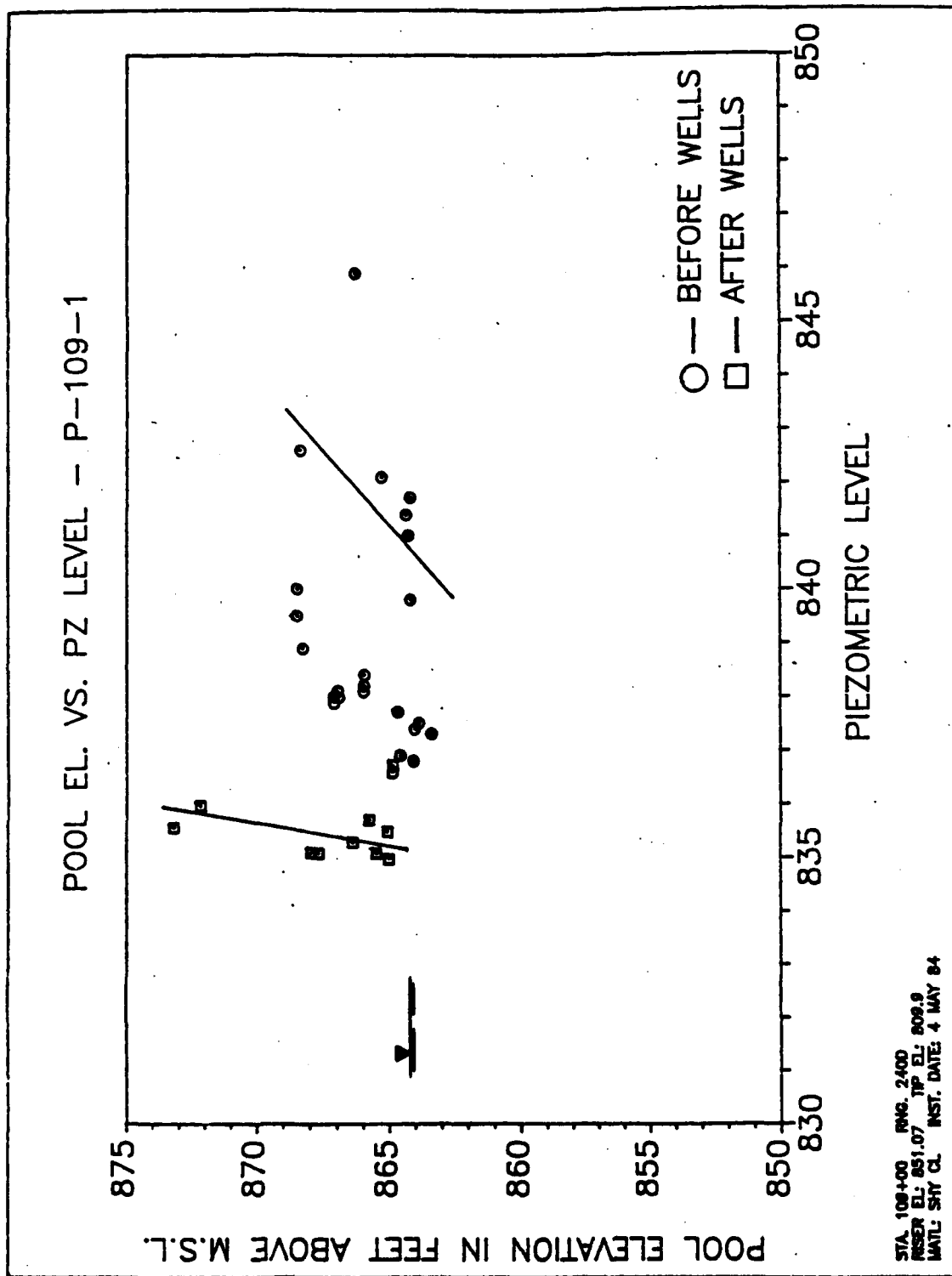


FIGURE NO. 4

Figure No. 4

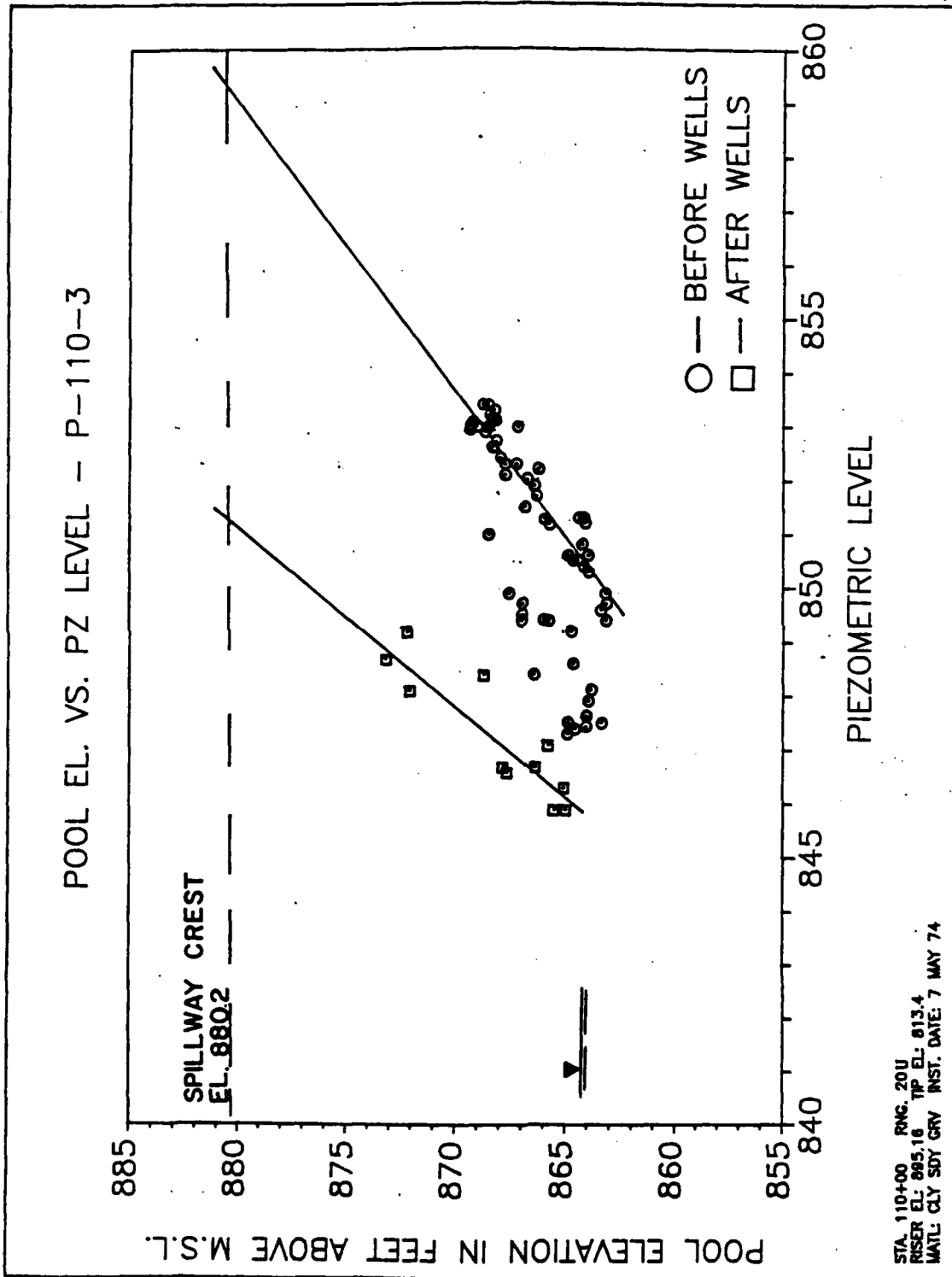


FIGURE NO. 5

Figure No. 5

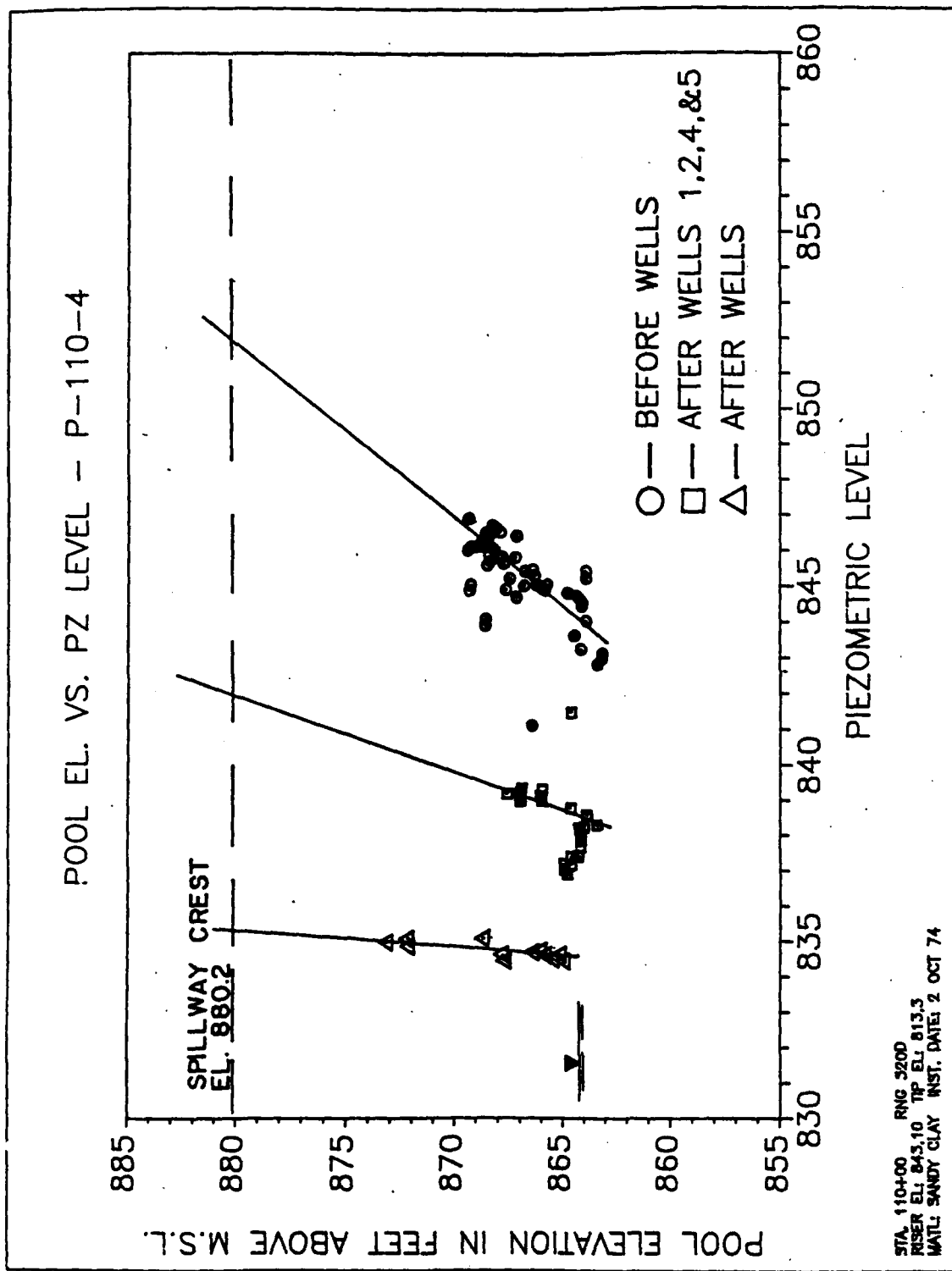


FIGURE NO. 6

Figure No. 6

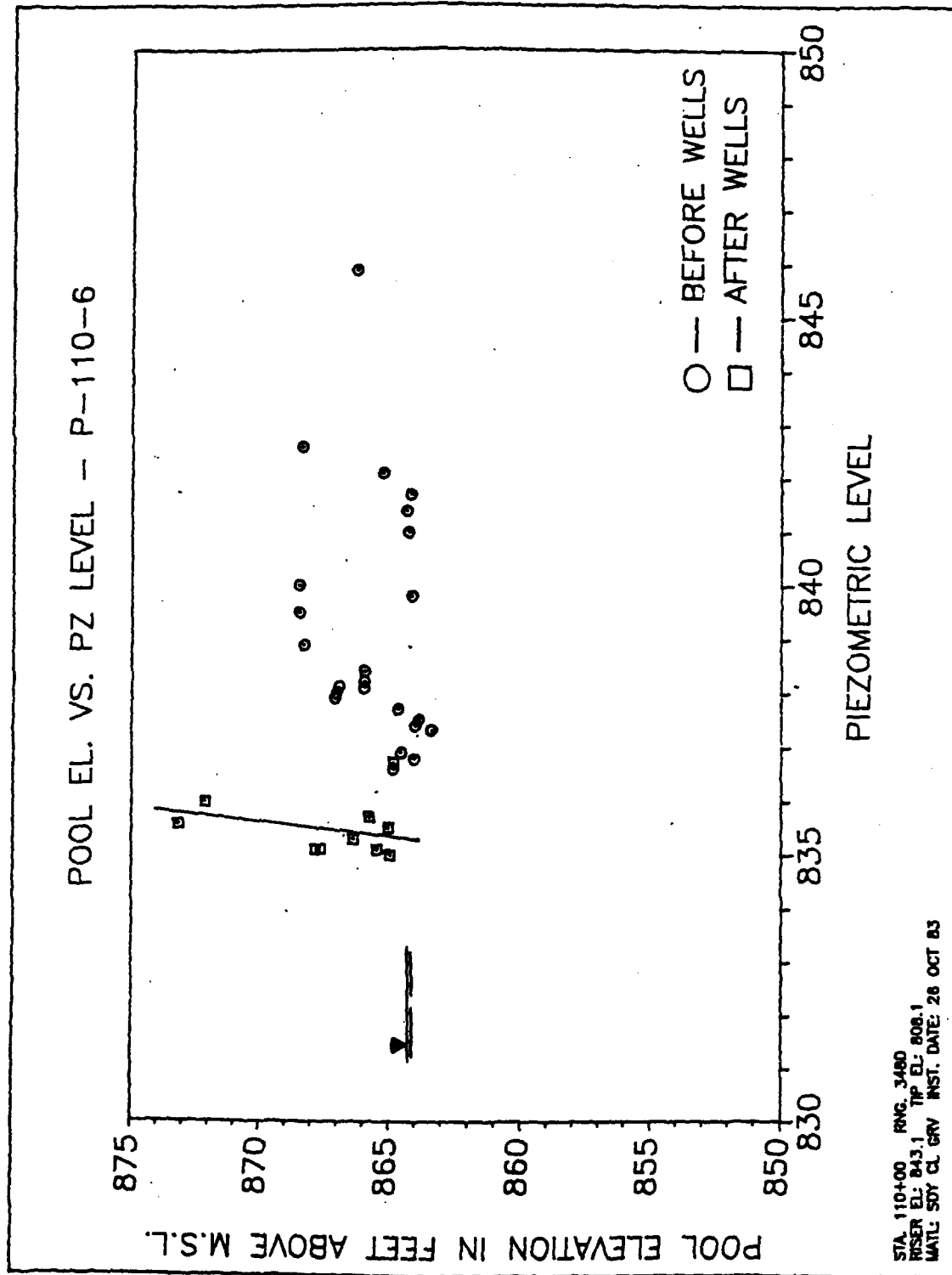
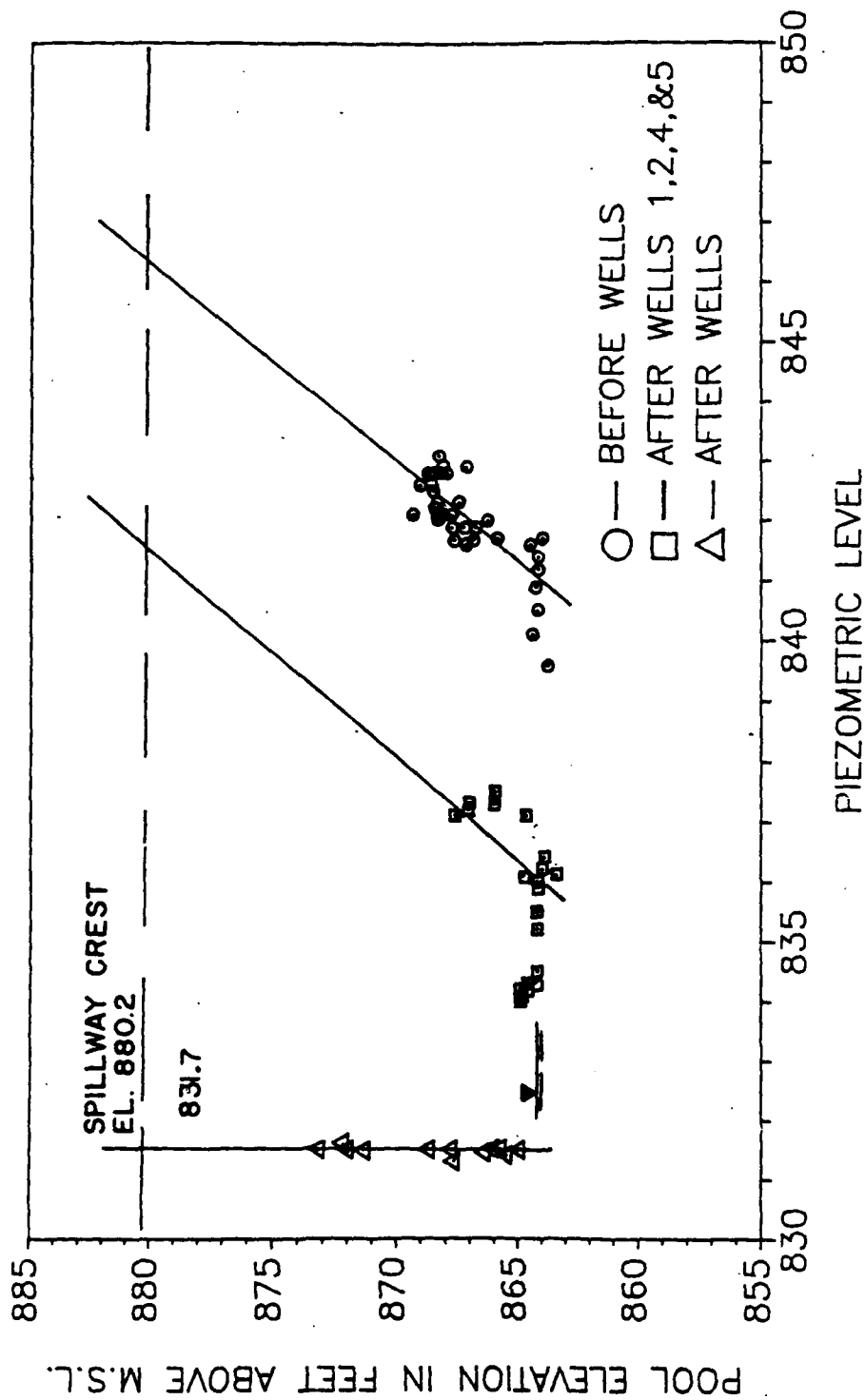


FIGURE NO. 7

Figure No. 7

POOL EL. VS. PZ LEVEL - P-110-7



STA. 110+00 RING. 423D
 RISER EL: 835.44 TIP EL: 809.5
 MATL: CL GRV INST. DATE: 22 NOV 83

FIGURE NO. 8

Figure No. 8

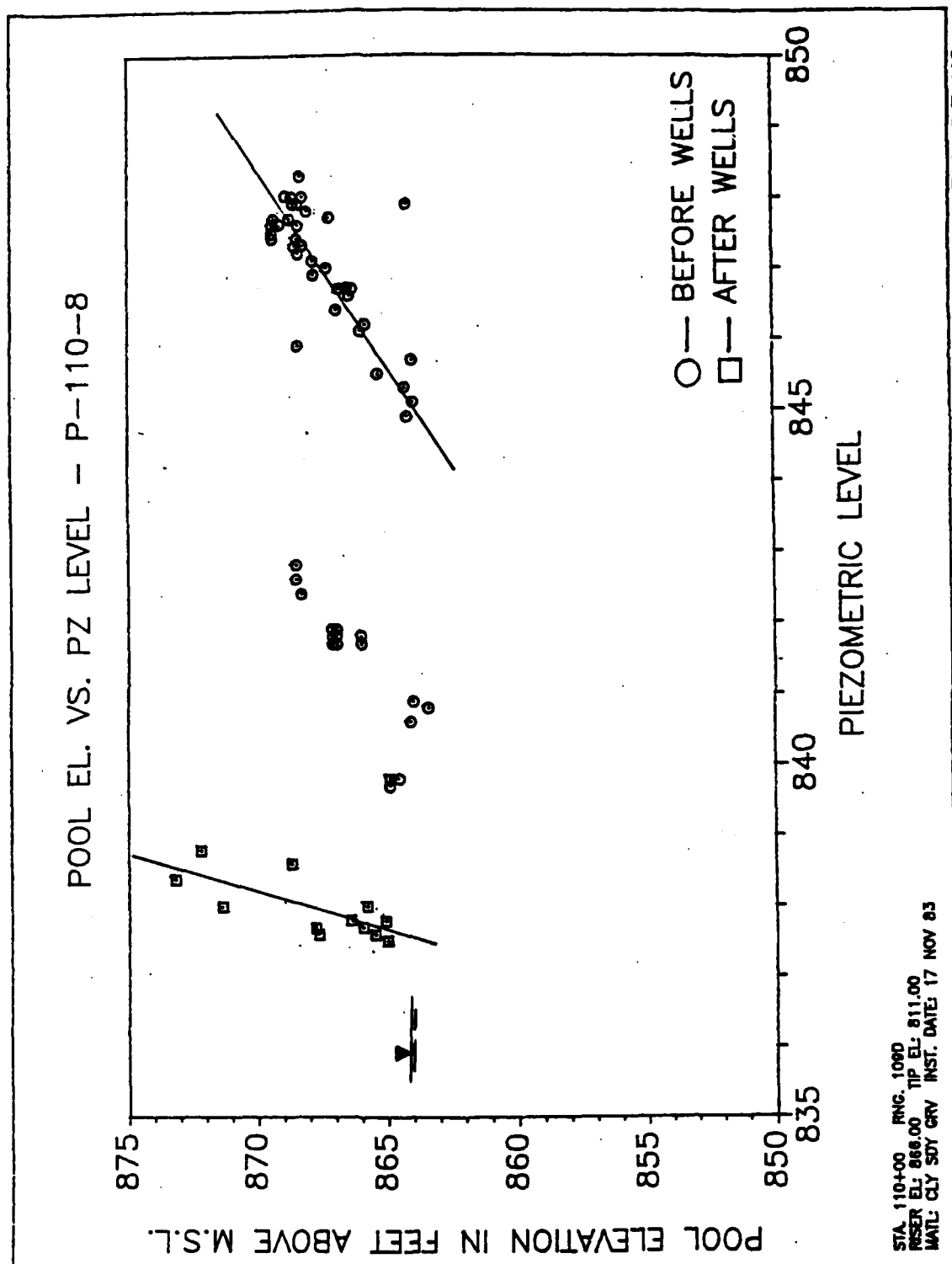


FIGURE NO. 9

Figure No. 9

POOL EL. VS. PZ LEVEL - P-110-9

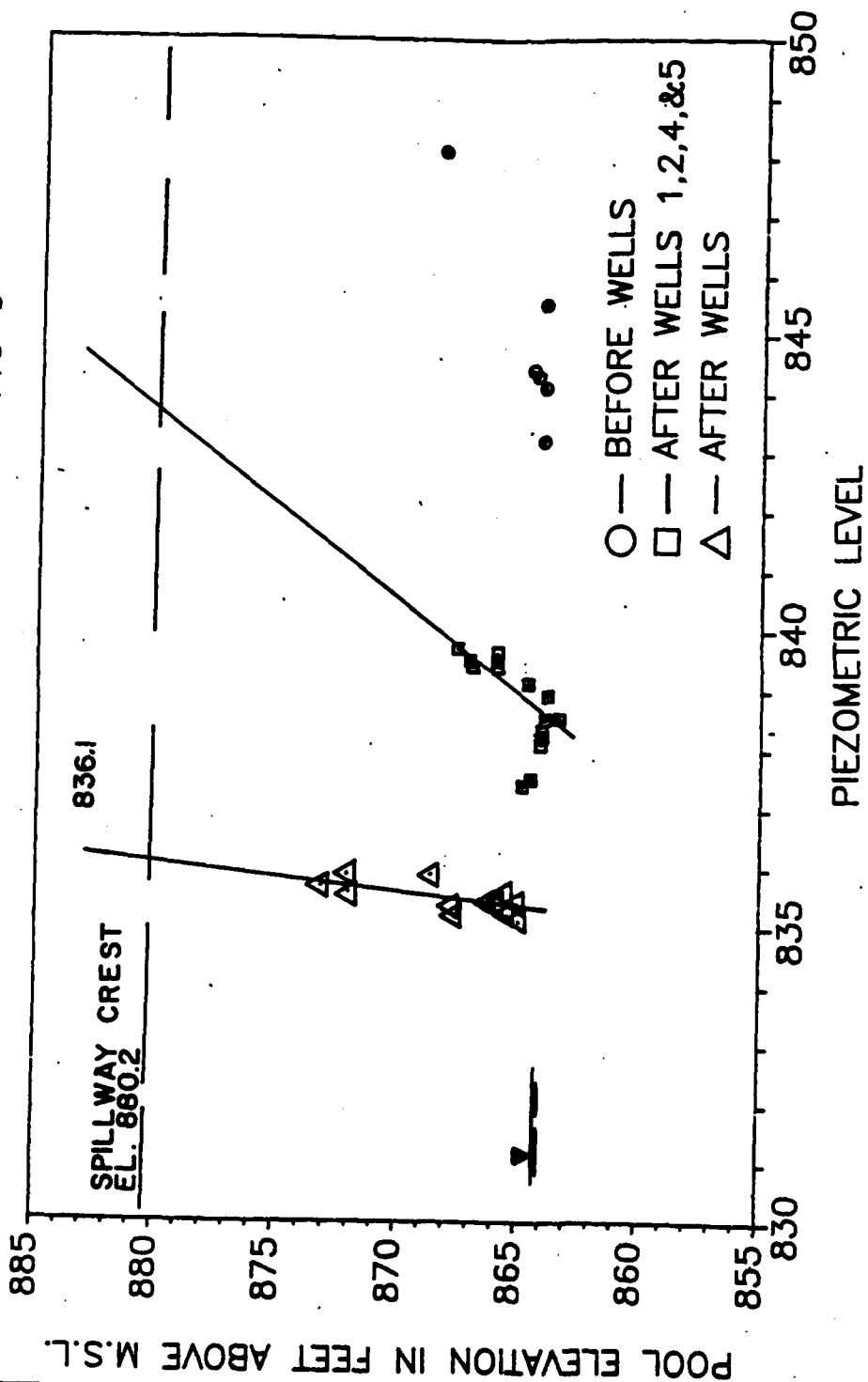


FIGURE NO. 10

STA. 110+10 INV. 2400
 REEF EL. 849.52 TP EL. 808.49
 MAIL: LS COBBLES INST. DATE: 3 MAY 84

Figure No. 10

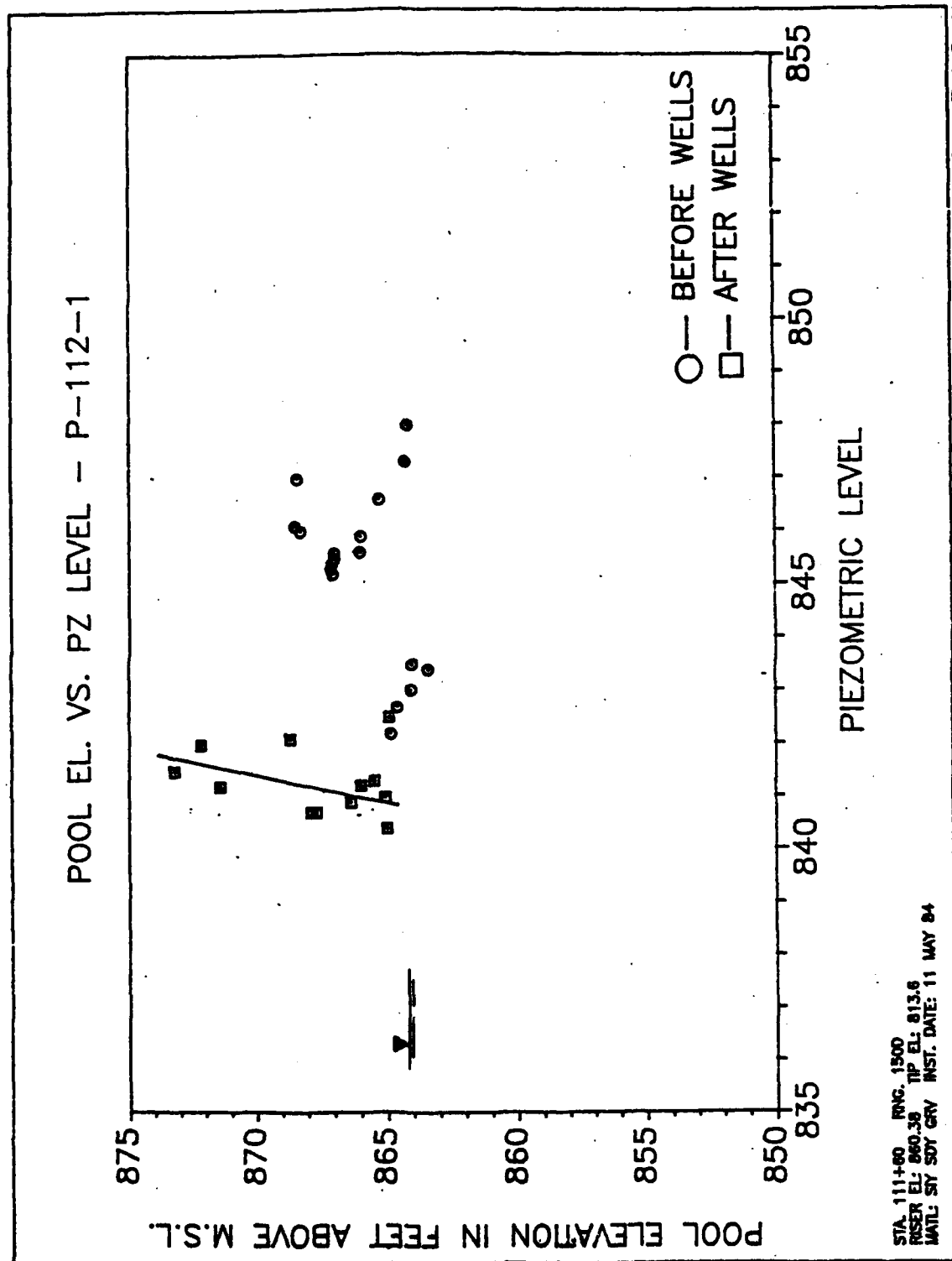


FIGURE NO. 11

Figure No. 11

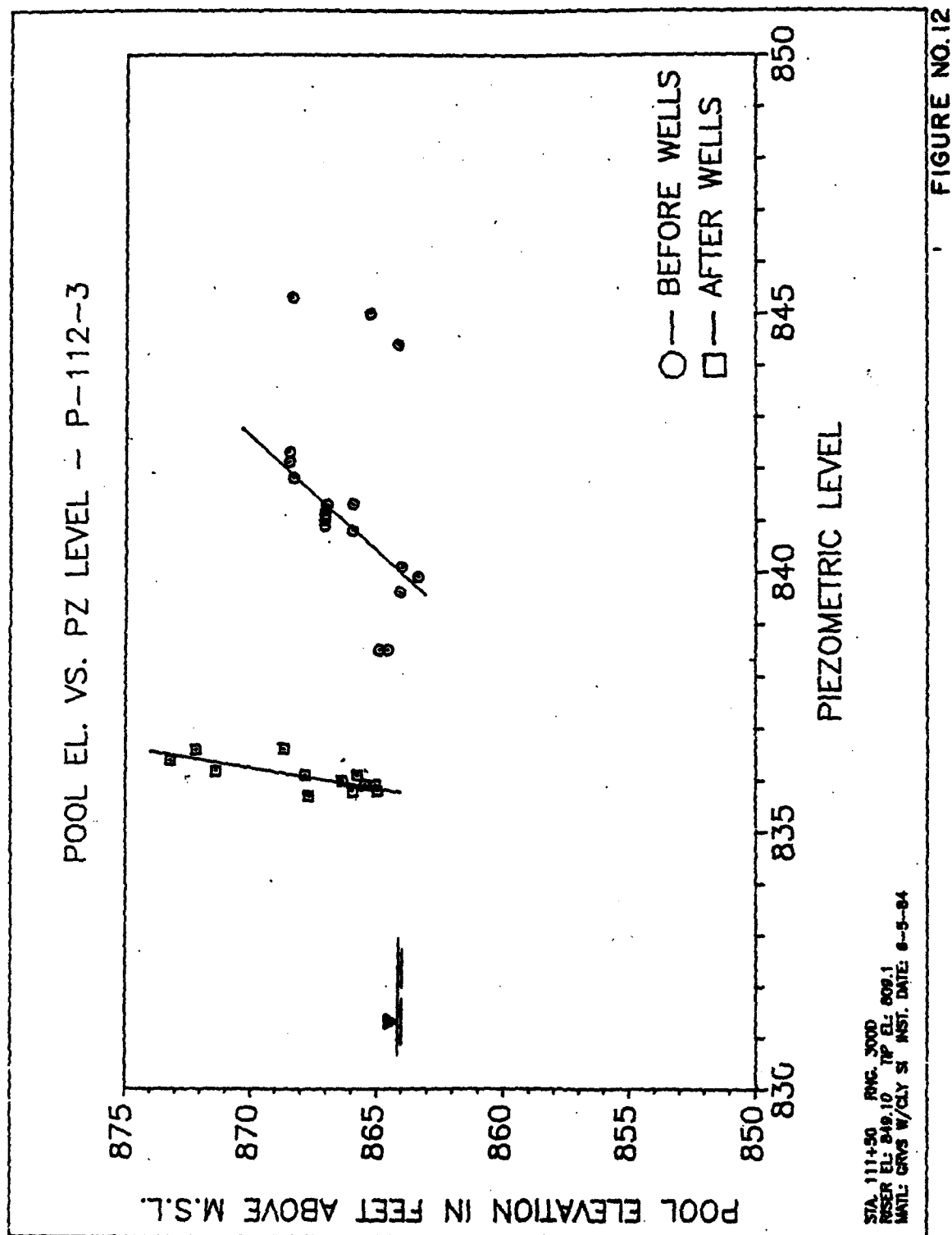
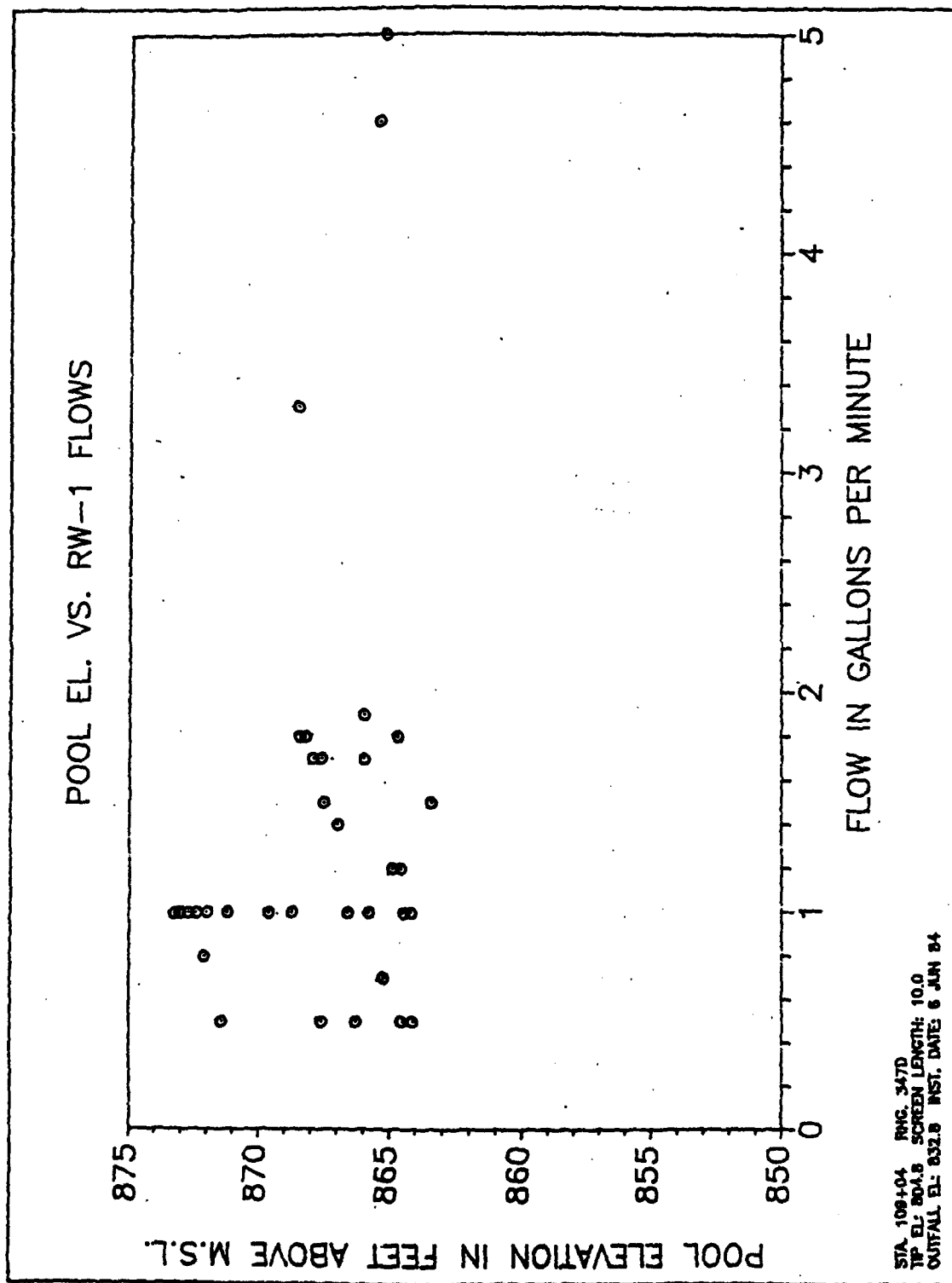


FIGURE NO.12

Figure No. 12



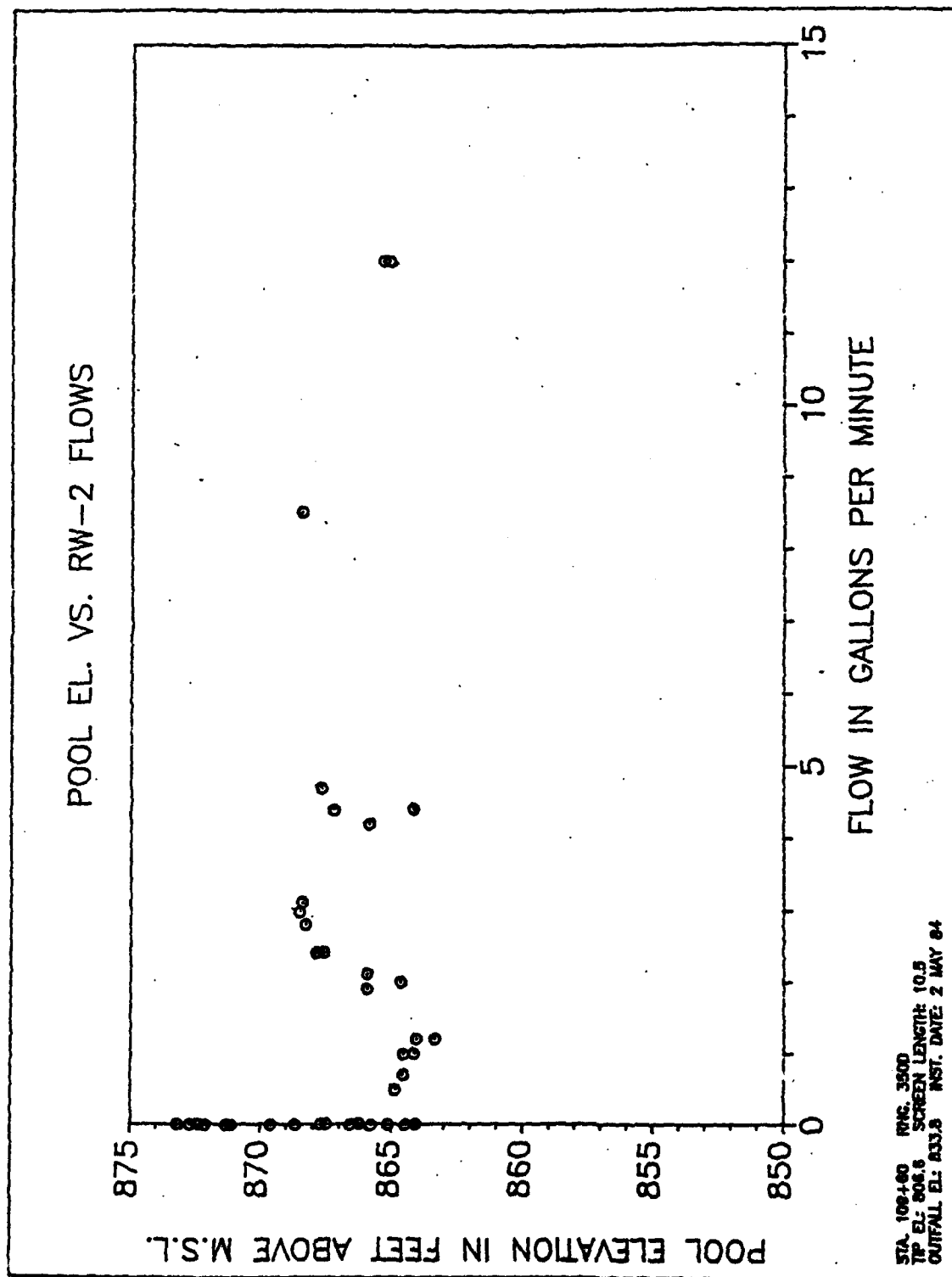


FIGURE NO. 14

Figure No. 14

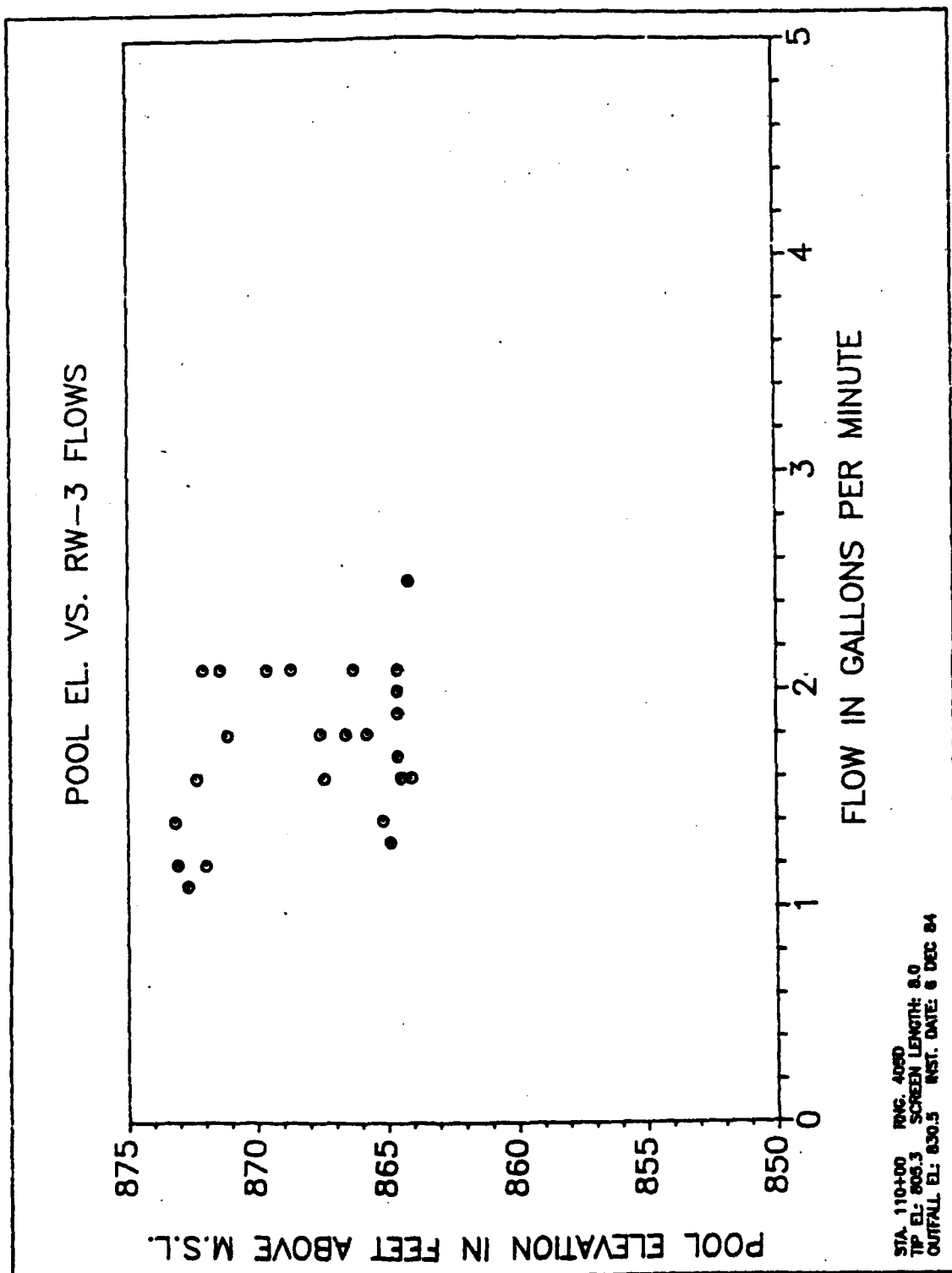


FIGURE NO.15

Figure No. 15

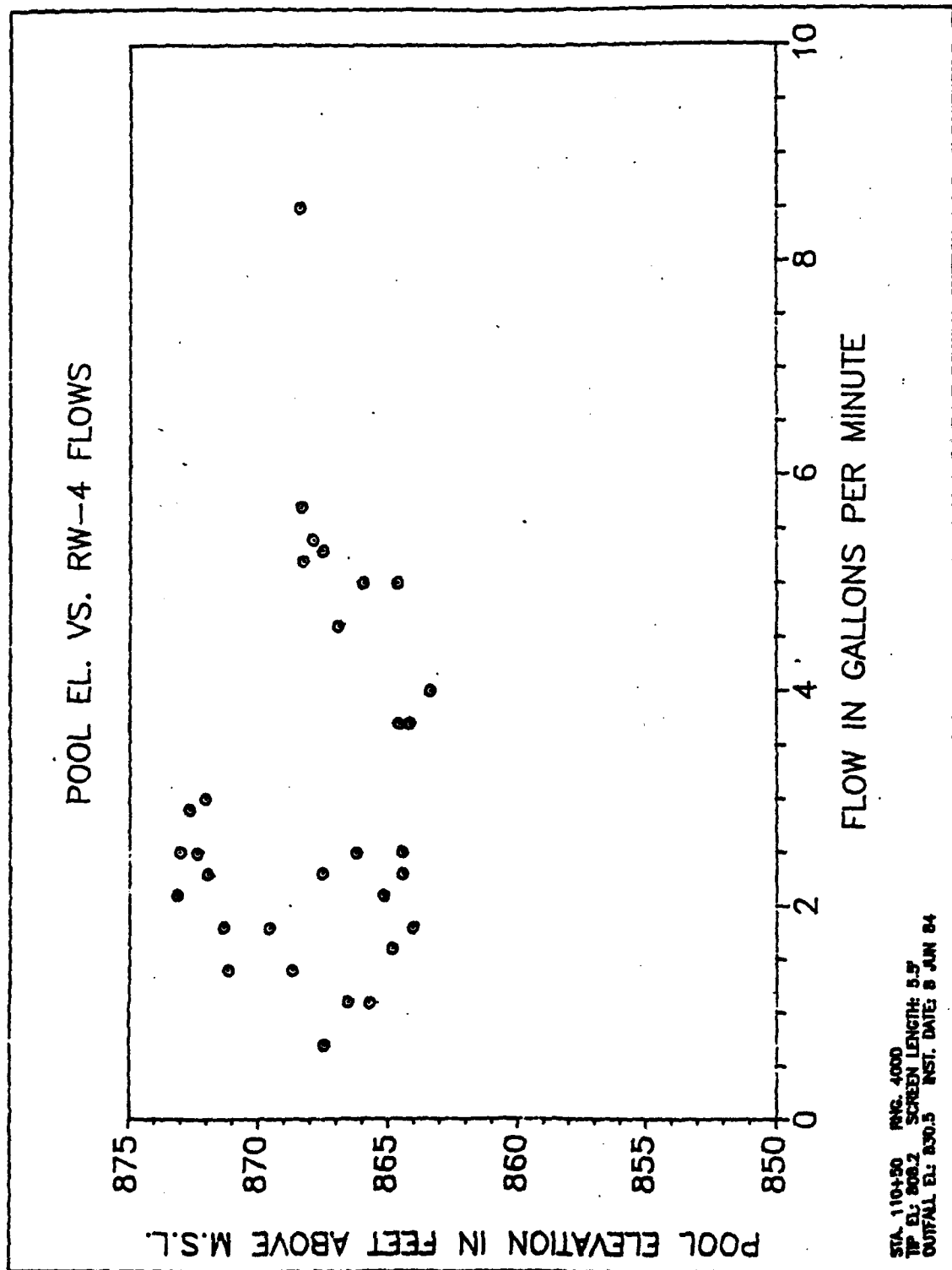


FIGURE NO. 16

Figure No. 16

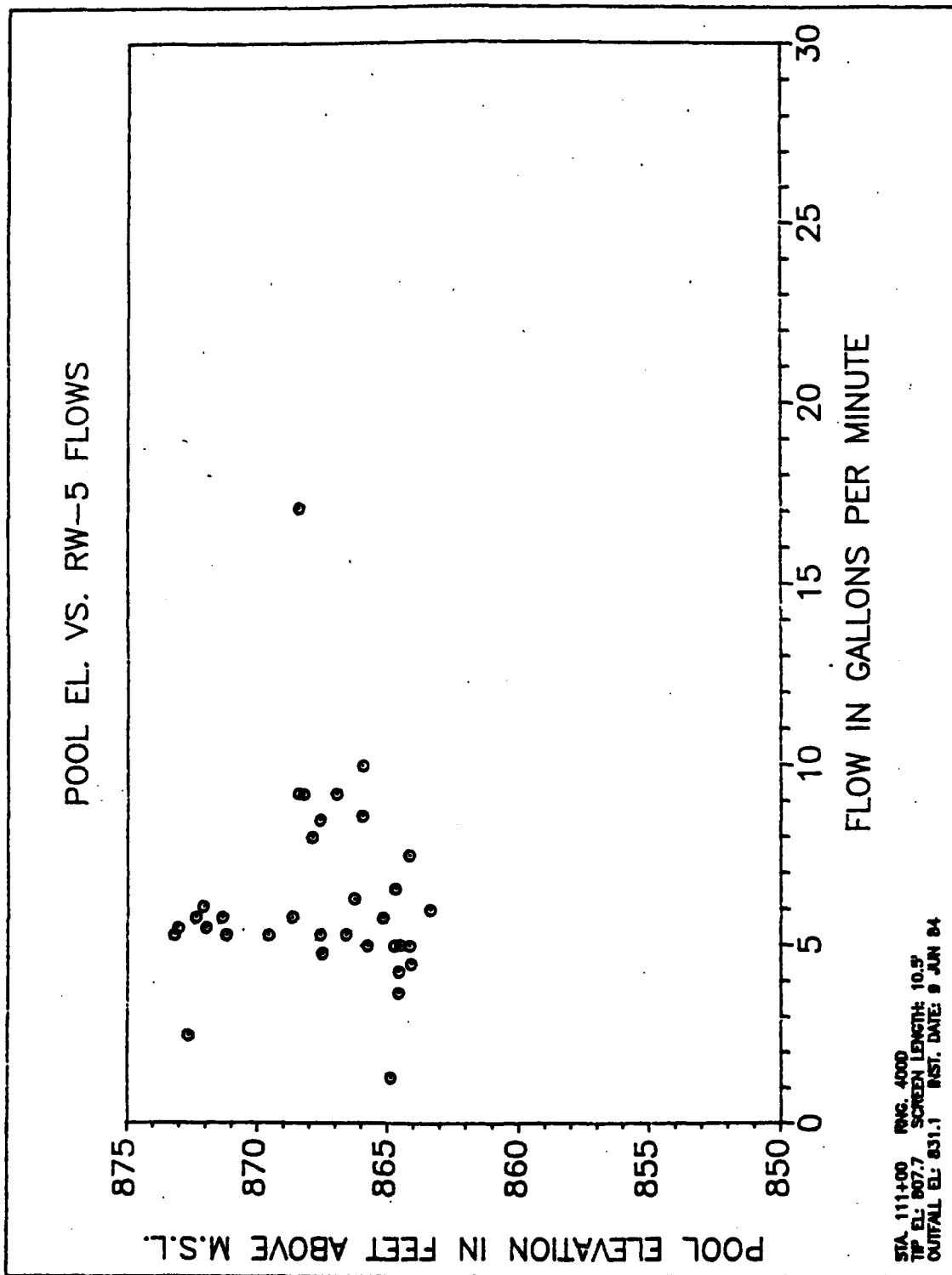


FIGURE NO. 17

Figure No. 17

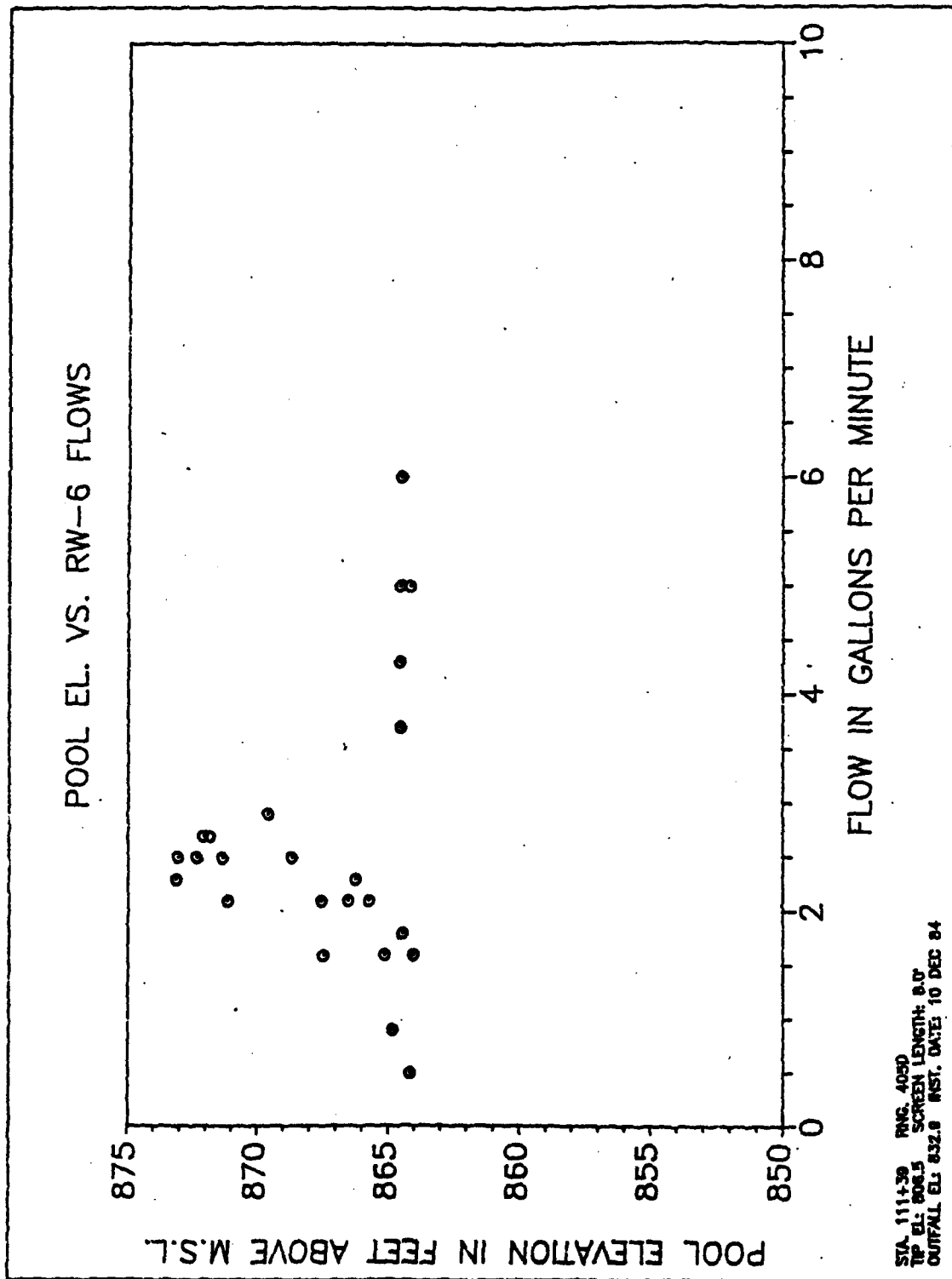


FIGURE NO. 18

Figure No. 18

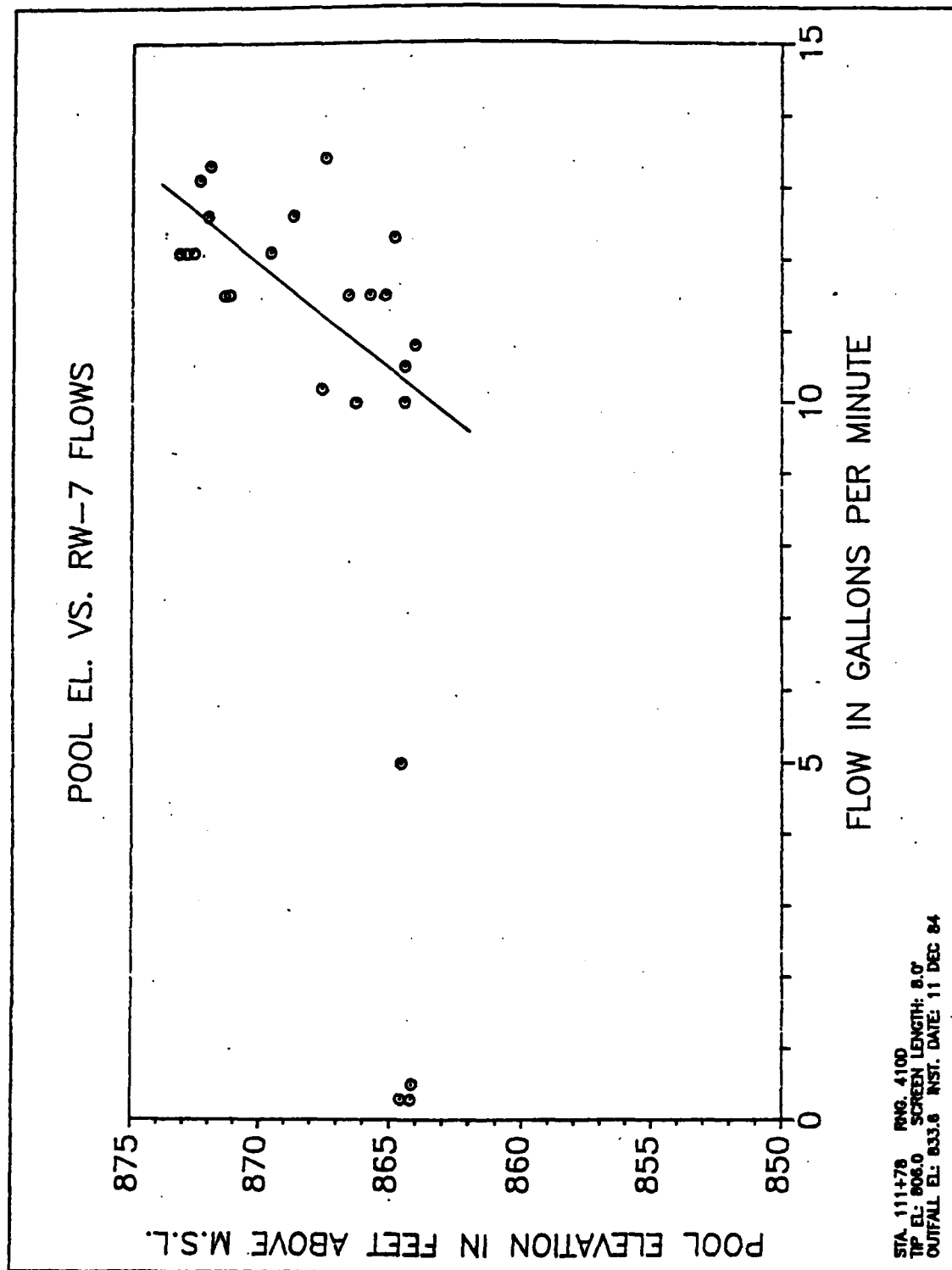


FIGURE NO. 19

Figure No. 19

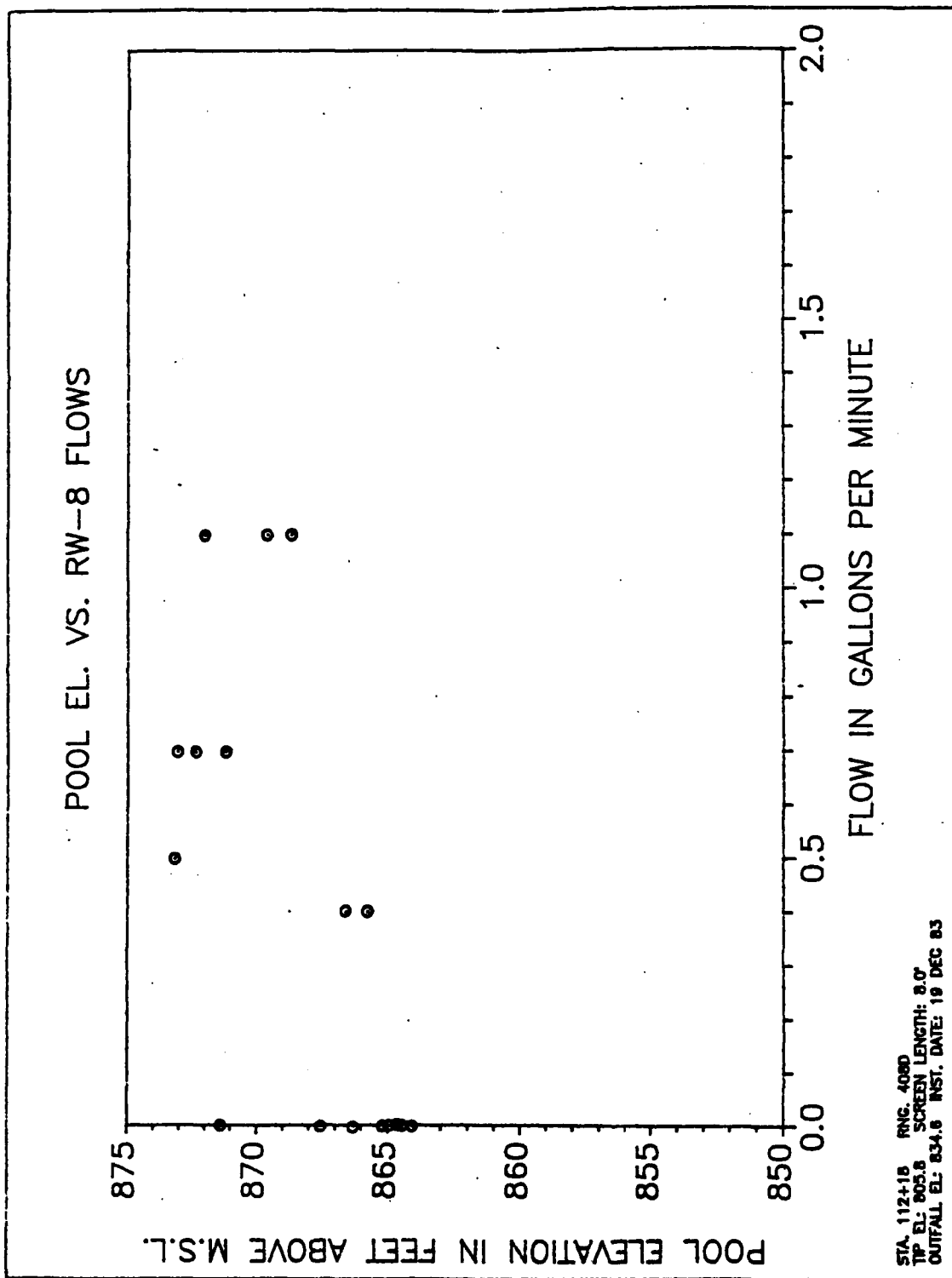


FIGURE NO. 20

Figure No. 20

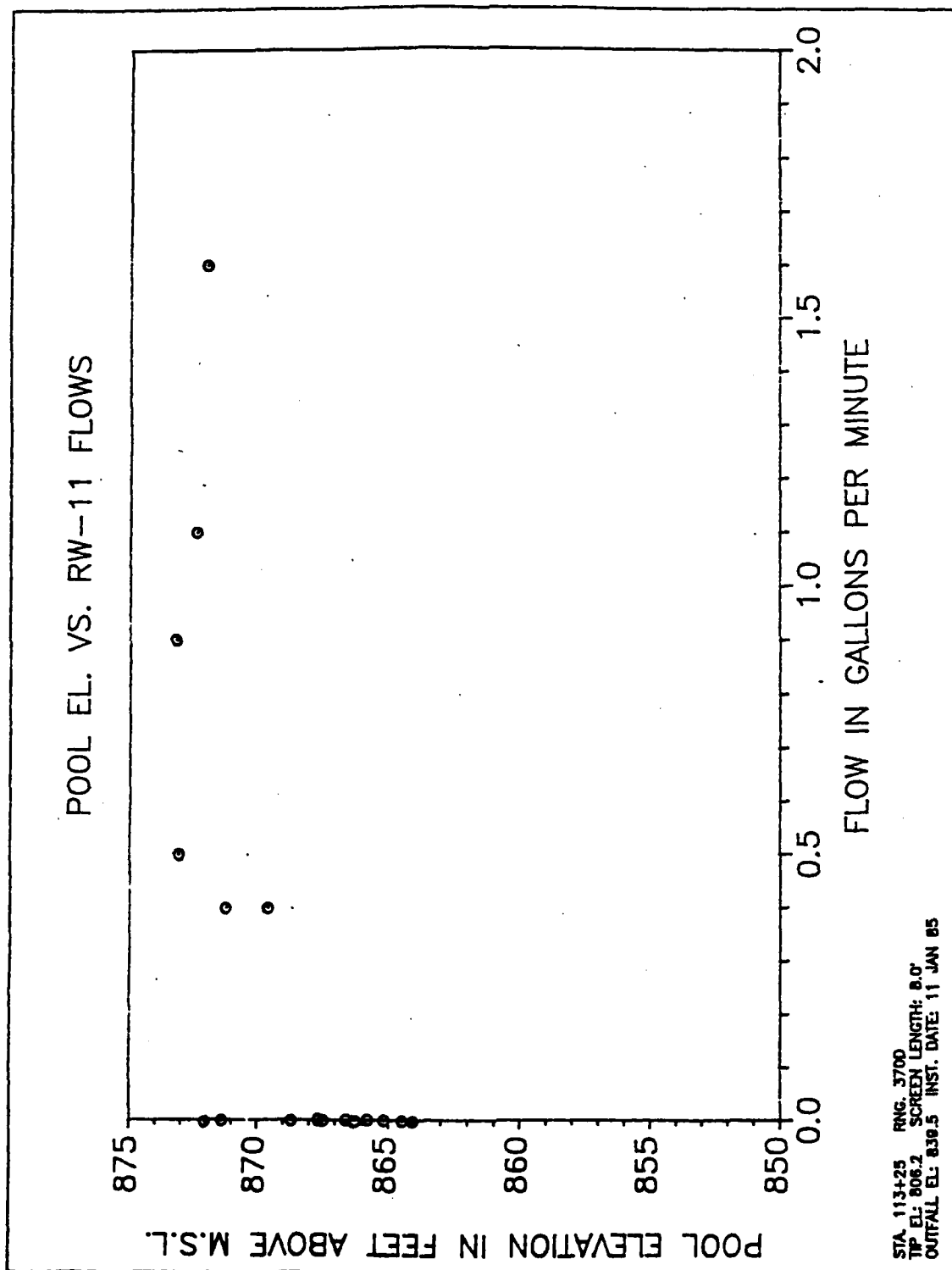


FIGURE NO. 21

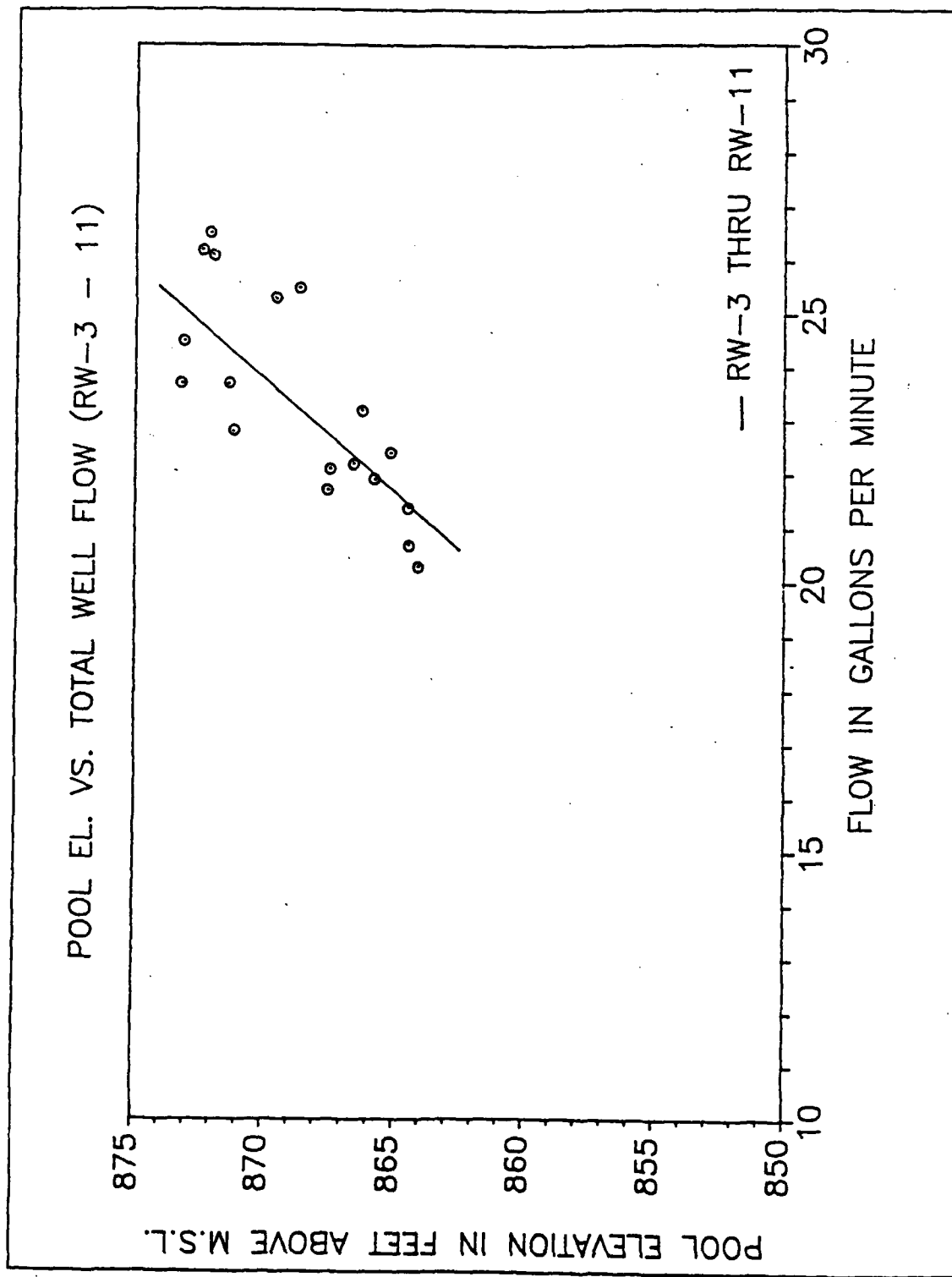
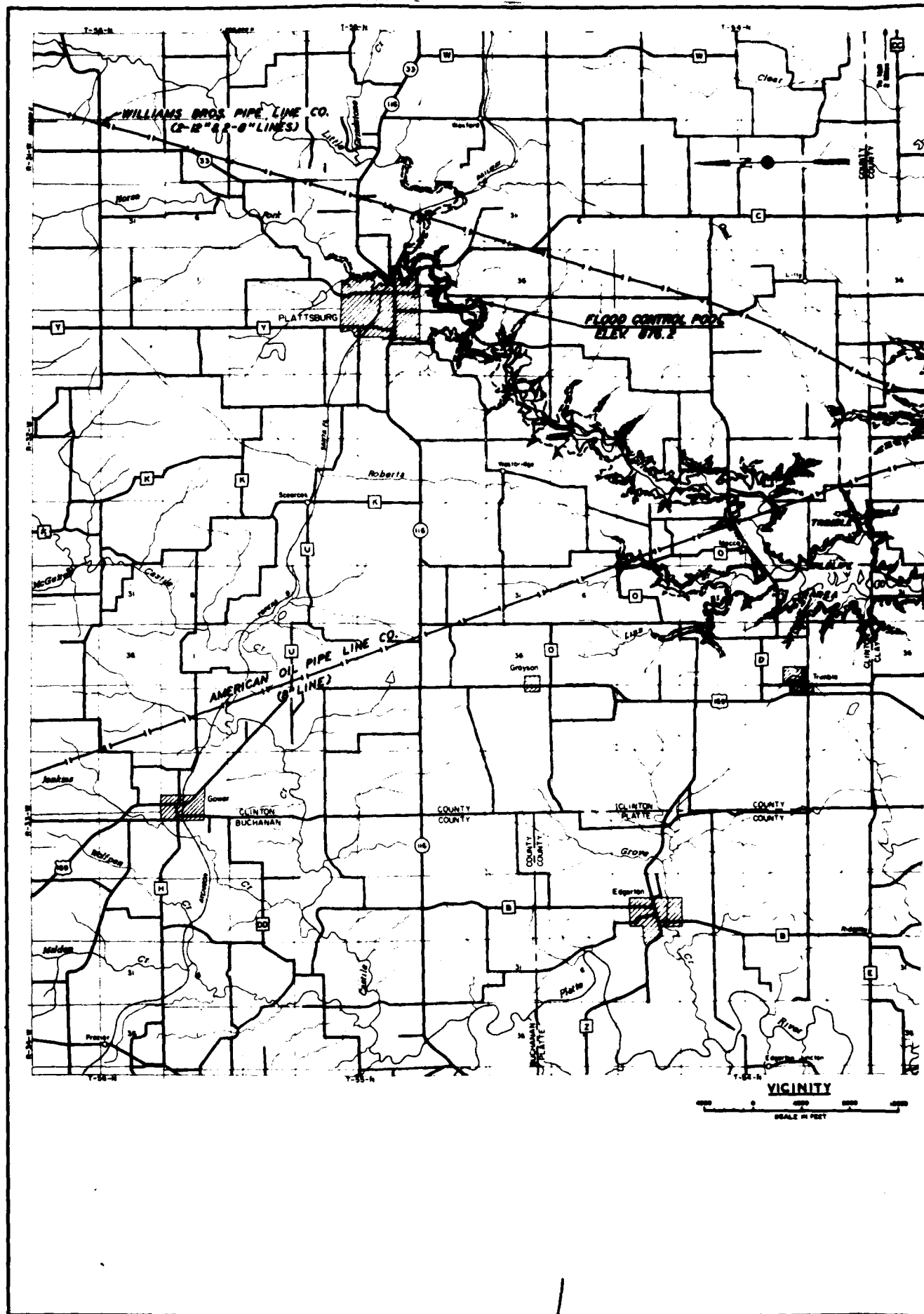
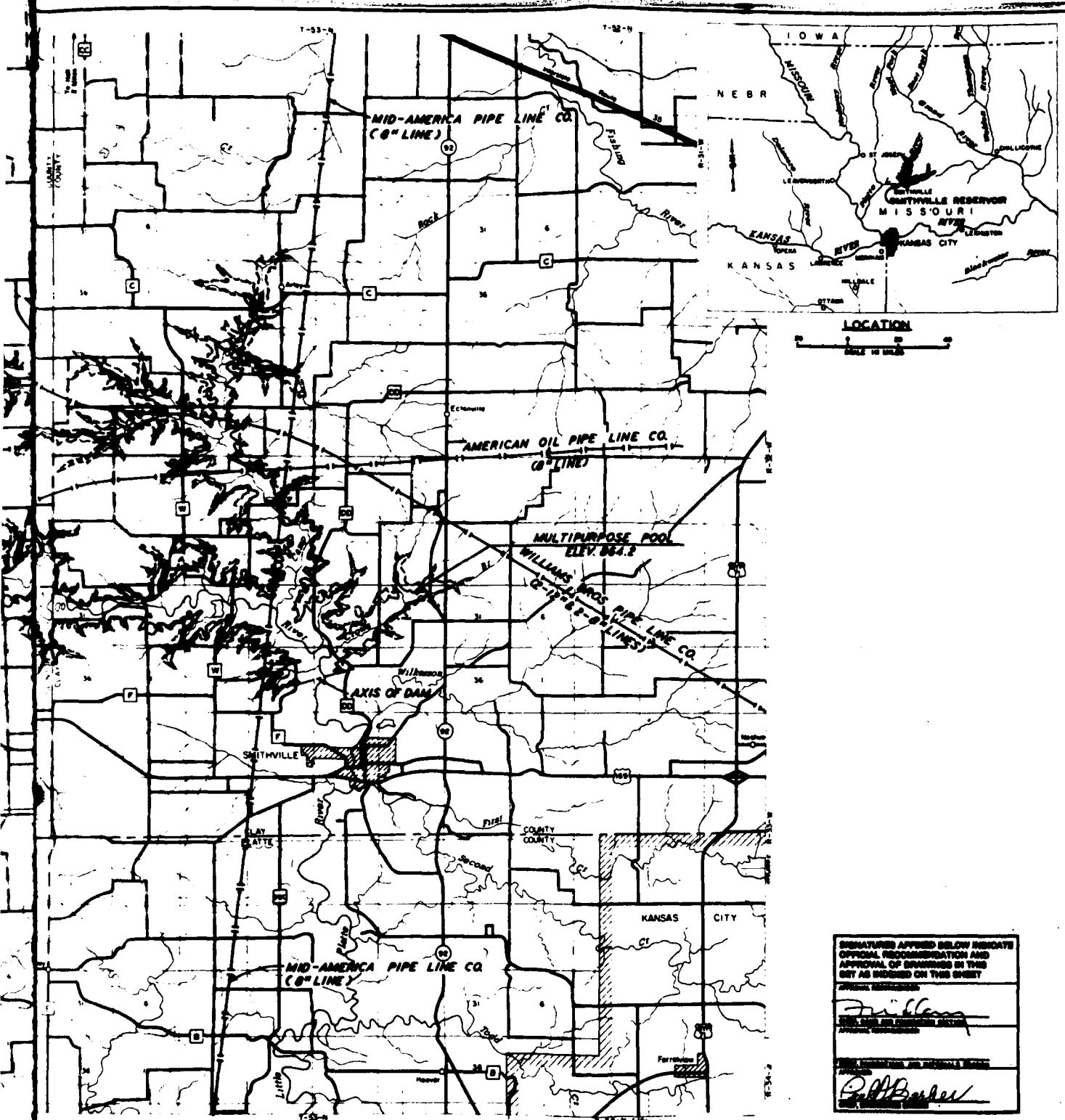


FIGURE NO. 22

DRAWINGS

DRAWINGS





LOCATION
SCALE IN MILES

SIGNATURES AFFIXED BELOW INDICATE
OFFICIAL RECOMMENDATION AND
APPROVAL OF DRAWINGS IN THIS
SHEET AS SHOWN ON THIS SHEET

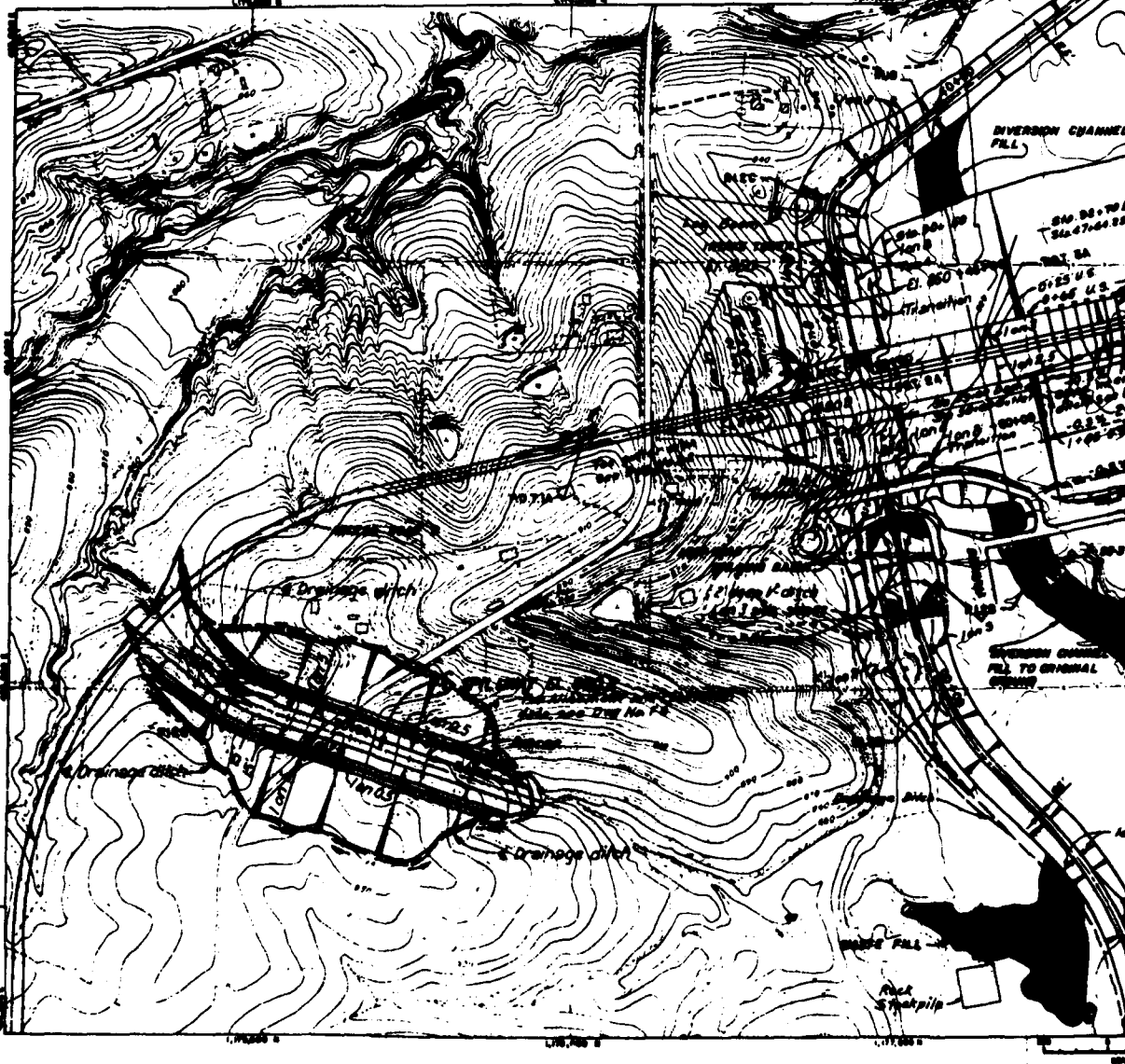
OFFICIAL RECOMMENDATION
[Signature]
DATE: 10-10-87
POSITION: ENGINEER

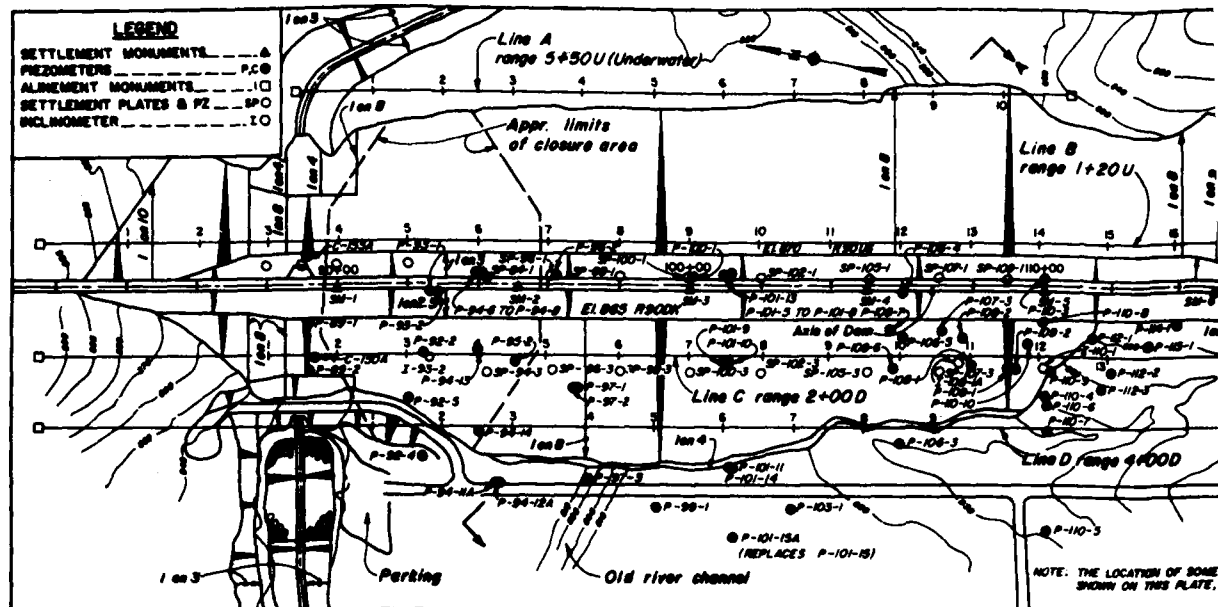
OFFICIAL APPROVAL
[Signature]
DATE: 10-10-87
POSITION: CHIEF ENGINEER

Revised	Revisions	Date	Approved

U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI	
Designed by: A.D.H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT LOCATION VICINITY MAP
Drawn by: R.A.C.	
Checked by: A.D.H.	
Submitted by: P.C.M.	Date: AS SHOWN MARCH 1987 RP-3-175

ALIGNMENT DATA										
R.L.	DOT	LATITUDE	DEPARTURE	AZIMUTH	DISTANCE	STATION	P.C. STA.	P.T. STA.	A	D
AXIS DAM										
1A	1172.97.67	488.70.28		87° 40'	900.00	80+00				
2A	1172.98.00	488.9° 00'		87° 30'	370.00	89+00 - Station 80+00	Outlet Works			
3A	1176.752.77	483.225.05		847° 30'	2281.65	92+70 - Station 47+64.29	Diversion Channel			
4A	1176.504.33	484.488.33		7° 30'	640.00	14+00.00	17+00.00	20° 31'	6° 40'	151.34' 859.41' 800.00'
5A	1176.878.70	484.488.33		11° 30'	800.00	19+00.47	123+00.47	4° 31'	1°	200.00 5729.58 400.00'
6A	1176.576.77	484.225.57				124+00.00				
DIVERSION CHANNEL										
7C	1176.002.23	483.461.86		82° 30'	1401.78	24+5.48	33+63.48	44° 30' 31"	5°	466.81' 148.32' 890.00'
8C	1176.806.04	484.783.78		77° 30'	753.34	39+5.78	41+39.48	63° 21'	30°	121.67' 190.98' 216.67'
9A	1176.732.27	483.186.08		77° 30'	700.00	47+64.29 - Station 92+70	Dam Axis			
10A	1176.820.00	483.82.67		31° 00'	440.00	53+00.18	56+10.18	46° 30' 11"	15°	164.11' 381.97' 310.00'
11A	1176.878.70	483.186.08		77° 30'		56+10.18	60+01.96	46° 30' 31"	15°	164.11' 381.97' 310.00'
OUTLET WORKS										
12C	1176.012.28	483.461.86		82° 30'	1610.00	24+5.48	33+65.48	44° 30' 31"	5°	466.81' 148.32' 890.00'
13C	1176.806.43	484.783.78		77° 30'	580.00	43+25.00	43+41.67	63° 21'	30°	121.67' 190.98' 216.67'
14A	1177.004.00	483.88.00		77° 30'	1020.78	50+00 - Station 89+00	Dam Axis			
15C	1176.878.67	483.225.67		48° 30'	1010.21	57+50	62+70	32° 21'	6° 03'	266.37' 531.03' 520.00'
16C	1176.878.67	483.225.67		52° 30'		65+34	70+04	31° 31'	6°	485.48' 382.33' 430.00'





PLAN OF OBSERVATION DEVICES

ALIGNMENT MONUMENTS		
NUMBER	STATION	ORIGINAL TOP ELEVATION
ALIGNMENT LINE "A" IS LOCATED 5+50 US AND IS PERMANENTLY UNDERWATER		
A-1	91+00	823.93
A-2	93+00	822.10
A-3	95+00	821.71
A-4	97+00	819.73
A-5	99+00	820.06
A-6	101+00	816.70
A-7	103+00	816.38
A-8	106+00	816.74
A-9	107+00	816.32
A-10	109+00	815.79
ALIGNMENT LINE "B" IS LOCATED 1+20 US		
B-1	84+00	825.51
B-2	86+00	821.40
B-3	88+00	827.23
B-4	90+00	827.22
B-5	92+00	827.23
B-6	94+00	827.23
B-7	96+00	827.29
B-8	98+00	826.77
B-9	100+00	826.70
B-10	102+00	826.96
B-11	104+00	827.06
B-12	106+00	827.19
B-13	108+00	827.28
B-14	110+00	827.32
B-15	112+00	827.30
B-16	114+00	827.32
B-17	116+00	827.44
B-18	118+00	827.90
B-19	120+00	827.38
ALIGNMENT LINE "C" IS LOCATED 2+00 DS		
C-1	89+00	823.64
C-2	90+00	822.23
C-3	92+00	822.03
C-4	94+00	821.59
C-5	96+00	822.15
C-6	98+00	822.18
C-7	100+00	822.30
C-8	102+00	822.33
C-9	104+00	822.34
C-10	106+00	822.35
C-11	108+00	822.39
C-12	110+00	822.41
C-13	112+00	822.49
ALIGNMENT LINE "D" IS LOCATED 4+00 DS		
D-1	91+00	827.37
D-2	93+00	827.30
D-3	95+00	827.41
D-4	97+00	827.31
D-5	99+00	827.32
D-6	101+00	827.37
D-7	103+00	827.62
D-8	106+00	NO PLATE
D-9	107+00	827.15

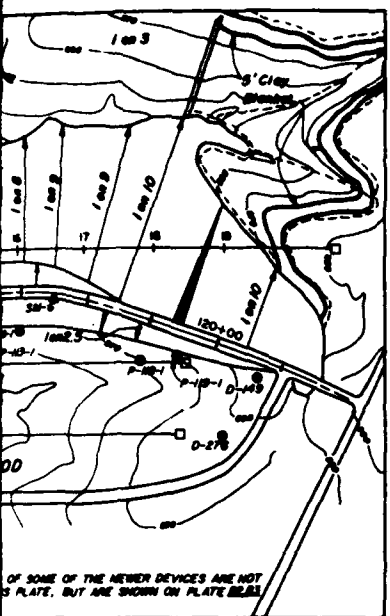
TRILATERATION SURVEY			
ALIGNMENT LINE "B" MONUMENTS AND ONE POINT ON THE UPSTREAM EDGE OF THE INTAKE TOWER ROOF ARE USED FOR THE TRILATERATION SURVEY. B1 IS DESIGNATED A1, B2 IS DESIGNATED A2, ETC. B19 IS DESIGNATED A19. THE TOWER IS DESIGNATED AS A20.			

INCLINOMETER SCHEDULE			
NUMBER	STATION	RANGE	BOTTOM ELEVATION
I-93-2	92+70	2+030	763.8
I-108-1A	108+03	2+34.50	782.9
I-110-1	110+00	2+400	781.1

CREST SETTLEMENT MONUMENTS			
NUMBER	STATION	RANGE	ORIGINAL TOP ELEVATION
SM-1	90+00	0+140	894.93
SM-2	95+00	0+140	894.94
SM-3	100+00	0+140	894.91
SM-4	105+00	0+140	894.94
SM-5	110+00	0+140	895.00
SM-6	115+00	0+140	894.95

SETTLEMENT PLATES W/PIEZOMETERS				
NUMBER	STATION	RANGE	ORIGINAL PLATE ELEVATION	TOP ELEVATION
SP-94-1	94+00	0+200	813.67	893.01
SP-94-3	94+20	2+300	814.11	894.43
SP-96-1	96+00	0+200	813.54	894.68
SP-96-3	96+00	2+300	813.54	895.62
SP-98-1	98+00	0+200	813.27	893.81
SP-98-3	98+00	2+300	813.15	895.85
SP-100-1	100+00	0+200	811.43	893.92
SP-100-3	100+00	2+300	813.67	897.26
SP-102-1	102+00	0+200	811.31	893.62
SP-102-3	102+00	2+300	811.24	894.62
SP-104-1	104+00	0+200	814.80	894.66
SP-106-1	106+00	2+300	816.30	897.33
SP-107-1	107+00	0+200	839.99	895.00
SP-107-3	107+00	2+300	822.30	897.05
SP-109-1	109+00	0+200	841.81	894.73

PIEZOMETERS			
NUMBER	STATION	RANGE	BL
P-98-1	88+25	1+000	
P-98-2	88+25	2+300	
C-100A	88+30.5	2+100	
P-92-2	92+50	2+000	
P-92-4	92+60	4+000	
P-92-5	92+00	3+230	
P-93-1	92+64	0+180	
P-93-2	92+75	0+120	
P-94-8	94+00	0+300	
P-94-7	94+10	0+200	
P-94-8	94+10	0+200	
P-94-11A	94+36	0+020	
P-94-12A	94+66	0+020	
P-94-13	94+04	1+000	
P-94-14	94+00	4+000	
P-95-2	95+05	2+000	
P-95-3	95+20.5	0+200	
P-97-1	96+70	3+000	
P-97-2	96+80	2+000	
P-97-3	97+00	0+000	
P-98-1	98+00	0+300	
P-100-1	100+00	0+200	
P-101-5	100+85	0+200	
P-101-6	100+85	0+200	
P-101-7	101+05	0+200	
P-101-8	101+05	0+200	
P-101-9	100+85	2+200	
P-101-10	101+05	2+200	
P-101-11	101+00	3+300	
P-101-13	101+00	0+200	
P-101-14	101+10	3+300	
P-102-5A	101+00	7+000	
P-103-1	103+00	0+300	
P-106-3	106+00	4+000	
P-106-4	106+00	0+100	
P-106-5	106+00	1+300	
P-106-6	106+00	2+400	
P-106-7	106+00	1+300	
P-107-3	107+00	1+300	
P-108-1	108+00	2+300	
P-108-3	107+30	3+000	
P-109-1	109+00	3+400	
P-109-2	109+10	2+400	
P-110-3	110+00	0+200	
P-110-4	110+00	2+200	
P-110-5	110+00	7+000	
P-110-6	110+00	3+400	
P-110-7	110+00	4+000	
P-110-8	110+00	1+000	
P-110-9	110+10	2+300	
P-110-10	89+47.5	0+200	
P-110-11	110+00	0+200	
P-112-1	111+00	1+000	
P-112-2	112+00	2+000	
P-112-3	111+30	2+000	
P-112-4	111+30	2+000	
P-112-5	111+30	2+000	
P-113-1	113+00	1+770	
P-113-2	112+75	3+790	



PIEZOMETER SCHEDULE (FOR DAM)

NUMBER	STATION	RANGE	TIP ELEVATION	MATERIAL	TOP ELEVATION	DATE INSTALLED	GROUND ELEVATION
P-114-1	114+00	1+100	817.1	SANDY CLAY	808.08	10-1-74	808.1
P-114-2	114+30	2+800	800.8	CLY GRV/SH	804.8	10-25-84	804.8
P-114-3	114+90	3+600	807.0	GRVY SAND	808.0	11-13-84	808.1
P-114-4	113+90	4+800	806.7	GRAVEL	801.8	11-7-84	806.8
P-118-1	118+00	1+000	807.00	GRVY CLY SD	878.33	11-9-83	808.0
P-118-2	118+00	0+700	804.0	CLY SDY GRAVEL	800.87	10-1-74	877.0
D-147R1			878.3	SHY CLY SD	802.11	4-18-71	808.3
D-147R2			882.5	SHY GRVY SD	802.11	4-18-71	808.3
D-148			888.2	LEAN CLAY/SHLT	808.4	4-18-71	808.0
D-148R1	120+15	0+200	808.7	CLY GRV	808.30	4-8-71	808.8
D-278	120+00	3+000	808.88	SHY LEAN CL	808.46	8-9-76	807.7
DC-803			810.0	CL SDY GRV	804.73	9-21-80	801.8
D-314			817.88	CLY SAND	846.03	10-13-83	840.98
DC-814-A			808.88	CLY GRVY SD	846.01	10-14-88	841.88
D-345			807.6	SHY GRVY SD	878.27	9-28-82	877.1
D-316			812.7	SHY GRVY SD	898.37	10-3-83	898.9
DC-317			812.9	SHY GRVY SD	883.88	10-7-83	880.0
D-318			823.3	SHY CLY GRV	882.88	10-20-83	889.3
D-319A			820.85	SHY LEAN CL	842.4	4-3-84	838.88
A-320			812.13	SHY CL	831.58	2-9-84	828.83
A-321			810.23	SHY SD	834.30	2-10-84	830.23
D-321A			808.88	SHY SAND	838.38	4-3-84	829.88
AD-322			812.8	CLY SAND	834.30	2-22-84	824.88
AD-323			793.38	SHY CLY	821.88	2-27-84	821.88
AD-324			808.88	LEAN CLAY	827.3	2-29-84	829.88

*MATERIAL DESCRIPTION FROM FIELD VISUAL CLASSIFICATION

PIEZOMETER SCHEDULE (FOR DAM)

NO.	TIP ELEVATION	MATERIAL	TOP ELEVATION	DATE INSTALLED	GROUND ELEVATION
200	805.7	SAND GRAVEL	808.28	9-20-76	804.3
300	797.1	SAND GRAVEL	845.44	9-29-76	845.3
400	790.3	CHALKY SHALE	838+00	9-8-76	846.7
500	776.0	SANDY SHALE	844.35	9-8-76	850.4
600	766.7	WEA SHALE	846.37	10-22-76	817.7
700	761.41	SHY GRV SD	838.23	11-20-83	836.4
800	766.38	SD	888.62	12-8-83	886.1
900	829.23	SHY CL	888.29	12-2-88	894.7
1000	801.8	LEAN CLAY	884.84	4-30-74	887.3
1100	788.7	CLY GRVY SAND	884.01	4-20-74	881.3
1200	775.5	CHALKY SHALE	881.08	4-29-74	887.4
1300	804.0	SILTY CLAY	880.29	9-29-76	883.5
1400	791.0	LEAN CLAY	818.27	9-19-76	814.3
1500	787.81	CLY SDY GRV	888.88	10-28-88	888.0
1600	764.7	SHY GRV SD	888.48	11-1-88	882.7
1700	791.0	LEAN CLAY	888.20	3-28-76	888.2
1800	802.0	SHY LEAN CLAY	884.38	3-18-76	880.7
1900	790.8	SHY CLAY	841.01	3-11-76	828.4
2000	788.2	LEAN CLAY	847.78	3-7-76	828.3
2100	787.4	CLAYEY SILT	847.58	3-28-76	813.4
2200	788.3	CLAYEY SILT	848.80	9-28-81	810.3
2300	788.1	WEA SHALE	888.84	4-1-78	881.8
2400	823.3	SAND-LEAN CLAY	888.38	10-2-74	882.1
2500	790.0	LEAN CLAY	881.02	4-10-74	887.1
2600	788.2	SANDY GRAVEL	884.97	4-17-74	881.5
2700	780.1	WEA SHALE	881.83	4-18-74	887.1
2800	788.8	LEAN CLAY	881.72	3-28-76	849.3
2900	790.2	LEAN CLAY	888.80	3-27-76	848.8
3000	788.4	LEAN CLAY	888.88	3-8-74	859.0
3100	788.1	WEA SHALE	888.88	4-18-74	888.5
3200	776.2	CHALKY SHALE	888.17	9-9-74	815.4
3300	788.8	CLAYEY SILT	888.88	2-18-81	815.8
3400	794.0	LEAN CLAY	881.88	2-18-81	815.0
3500	809.9	CLAYEY SAND	888.88	10-4-74	888.1
3600	807.87	SHY GRV SD	887.88	12-9-88	884.9
3700	808.7	CLAY GRVY SD	888.84	3-17-88	887.4
3800	808.0	SHY GRV CL	848.7	3-28-84	848.4
3900	798.0	CHALKY SH	888.87	3-28-84	888.0
4000	804.7	SHY CLAY	888.38	9-19-84	888.3
4100	801.4	CHALKY SH	888.78	4-30-84	847.4
4200	810.2	SHY CL	888.73	3-18-84	887.7
4300	811.3	SHY SAND	888.8	11-2-84	888.3
4400	809.9	SHY CL	888.87	2-1-85	888.9
4500	798.8	CHALKY SH	888.48	9-18-85	888.9
4600	813.4	CLY GRVY SAND	888.18	9-7-74	888.0
4700	813.2	SHY CLAY	888.82	12-2-74	888.1
4800	812.0	SANDY CLAY	888.84	9-2-74	881.8
4900	809.1	SHY CL GRV	848.1	12-28-88	888.1
5000	809.5	CL GRV	888.44	11-28-88	888.8
5100	811.88	CLY GRVY SD	888.88	11-17-88	888.0
5200	808.98	LA CHALKY	888.88	9-2-81	888.78
5300	808.88	SHY LA SH GRV	888.84	9-2-81	888.88
5400	812.4	SHY GRV	888.8	11-28-88	888.8
5500	813.4	SHY GRV SD	888.88	5-11-84	888.8
5600	808.78	SHY LA SH GRV	888.88	9-7-84	888.78
5700	888.1	SHY GRVY SD	848.8	9-6-84	848.88
5800	811.7	SHY GRV SD	848.8	11-1-84	888.8
5900	804.0	SHY GRV SD	848.4	11-8-84	888.3
6000	804.4	SHY GRV	848.48	9-7-84	887.8
6100	802.2	SHY GRV	848.8	12-18-84	881.2

PIEZOMETER SCHEDULE (FOR MAIN DIKE)

NUMBER	STATION	RANGE	TIP ELEVATION	MATERIAL	TOP ELEVATION	DATE INSTALLED	GROUND ELEVATION
P-18-1	8+00	1+400	821.5	CLY SHY SD	882.57	11-9-84	882.1
P-20-1	20+00	2+000	840.0	SANDY CLAY	888.68	9-21-76	878.0
P-20-2	18+00	0+100	840.1	SANDY CLAY	887.88	9-27-76	886.0
P-20-3	20+00	0+100	881.7	WASH CRACK SH	887.80	9-2-76	886.0
P-20-4	20+00	2+800	835.0	SANDY CLAY	883.38	9-28-76	886.4
P-22-1	22+00	0+1000	883.8	CLY SD	870.88	11-12-84	887.8
P-22-2	22+00	2+000	828.4	SHY CL/CLY SD	880.88	11-12-84	886.7

*MATERIAL DESCRIPTION FROM FIELD PERMEABILITY TESTS

RELIEF WELL INSTALLATION SCHEDULE RW-1 THRU RW-13

RELIEF WELL NO.	STATION	RANGE	APPROX. DEPTH (FT)	APPROX. BOTTOM OF WELL (ELEV.)	BOTTOM OF WELL SCREEN (ELEV.)	SCREEN LENGTH (FT)	SCREEN PACK (FT)	BENTONITE BALL SEAL (FT)	GROUT BACKFILL (FT)	PVC RISER TO TOP OF WELL (FT)	TOP OF PVC RISER (ELEV.)	OUTFLOW AT THE WELL	DATE INSTALLED
1	108+04	3+700	30.0	802.8	804.8	10.0	18.7	3.0	10.3	17.4	883.9	881.8	8-2-84
2	108+80	3+800	38.0	786.6	804.8	10.5	23.5	0.0	15.3	15.9	887.07	882.9	8-2-84
3	109+00	4+000	36.0	808.8	808.3	8.0	16.4	1.8	12.8	16.7	888.5	888.5	12-9-84
4	110+80	4+000	30.1	803.9	808.2	5.3	16.3	3.7	10.1	16.8	888.3	880.4	8-2-84
5	111+00	4+000	28.9	805.1	807.7	10.3	17.7	2.7	9.3	12.9	888.2	881.1	8-2-84
6	111+30	4+000	32.0	803.5	804.5	8.0	17.0	2.0	13.0	17.9	887.08	888.9	12-10-84
7	111+78	4+100	38.0	801.6	808.5	8.0	17.9	2.3	14.9	18.8	888.04	888.9	12-11-84
8	112+18	4+000	38.0	801.6	805.1	8.0	17.0	2.3	16.8	20.8	888.58	888.4	12-9-84
9	112+58	3+800	38.0	802.3	804.8	8.0	18.2	2.0	20.8	23.8	884.88	887.8	12-17-84
10	112+88	3+780	40.0	801.3	807.2	8.0	18.8	2.1	18.4	22.4	888.78	888.3	1-3-88
11	113+28	3+700	42.0	800.4	804.2	8.0	20.0	3.1	18.9	24.8	888.88	888.3	1-12-88
12	107+15	4+300	48.0	804.5	888.7	8.0	18.8	2.0	18.4	23.8	888.70	88	1-28-88
13	108+00	4+300	38.5	802.5	888.2	8.0	18.0	3.0	13.0	20.0	888.25	88	1-28-88

WHERE TWO VALUES ARE GIVEN, THE ONE BELOW THE LINE IS PLACED LOWER IN THE RELIEF WELL. GROUTED WELLS 12 AND 13 DISCHARGE INTO SAND BLANKET

PUMPED WELL INSTALLATION SCHEDULE

WELL NO.	STATION	RANGE	APPROX. DEPTH (FT)	APPROX. BOTTOM OF WELL (ELEV.)	BOTTOM OF WELL SCREEN (ELEV.)	SCREEN LENGTH (FT)	SCREEN PACK (FT)	BENTONITE BALL SEAL (FT)	GROUT BACKFILL (FT)	6" PVC RISER (FT)	TOP OF PVC RISER (ELEV.)	DATE INSTALLED
1	109+70	1+070	84.3	788.7	808.2	10.0	24.3	0.0	40.0	81.5	884.88	4-10-84
2	112+00	1+080	82.4	801.5	804.6	21.0	24.2	2.0	35.0	86.9	888.88	1-9-84
4	108+78	1+885	48.8	804.3	811.3	1.0	18.1	2.0	27.3	88.3	884.48	1-10-84

**U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI**

**LITTLE PLATTE RIVER, MISSOURI
BENTONVILLE LAKE
EMBAKMENT CRITERIA REPORT**

**OBSERVATION DEVICES
PLAN VIEW AND SCHEDULE**

Designed by: J.B.M.

Drawn by: R.B.T.

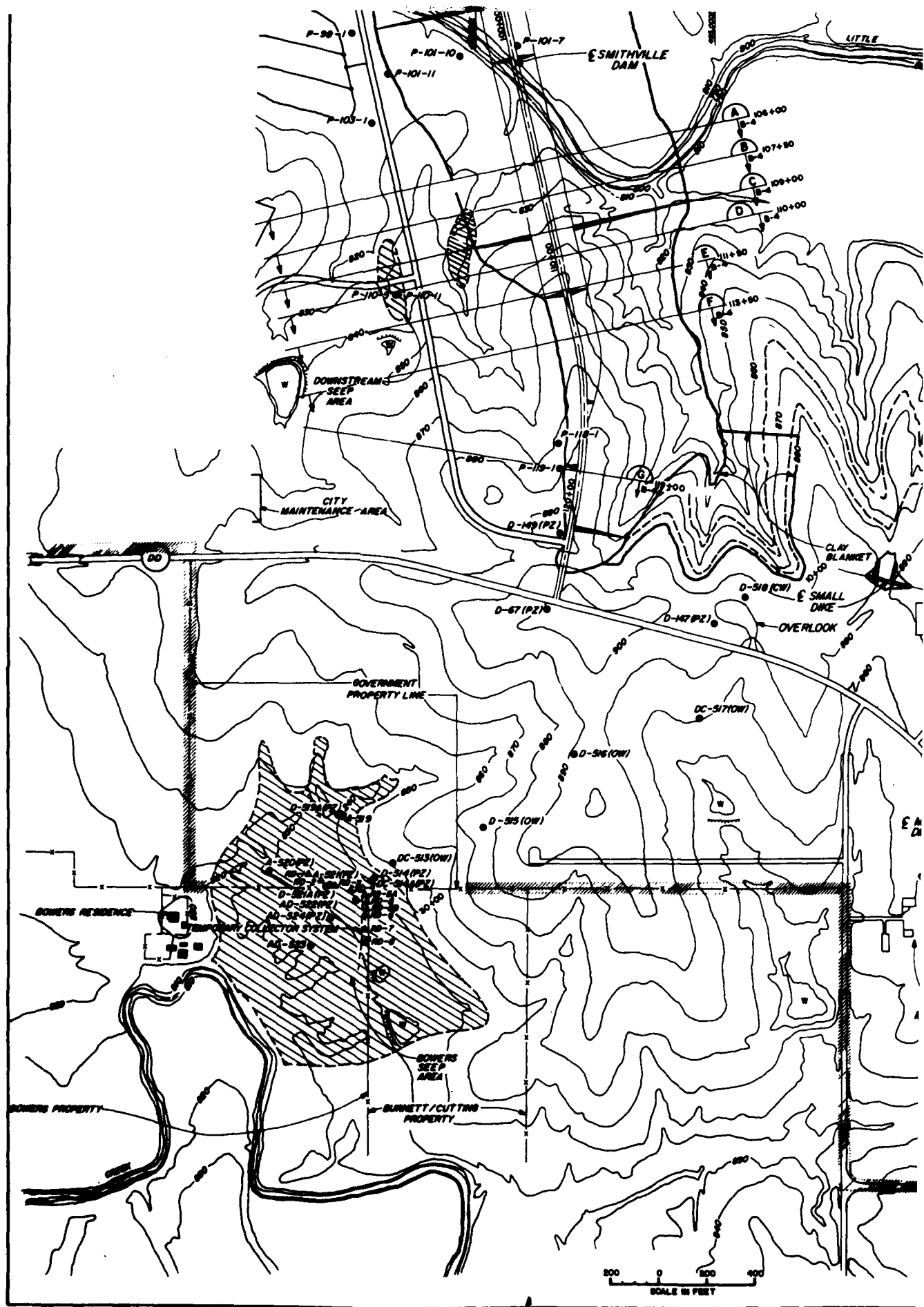
Checked by: J.B.M.

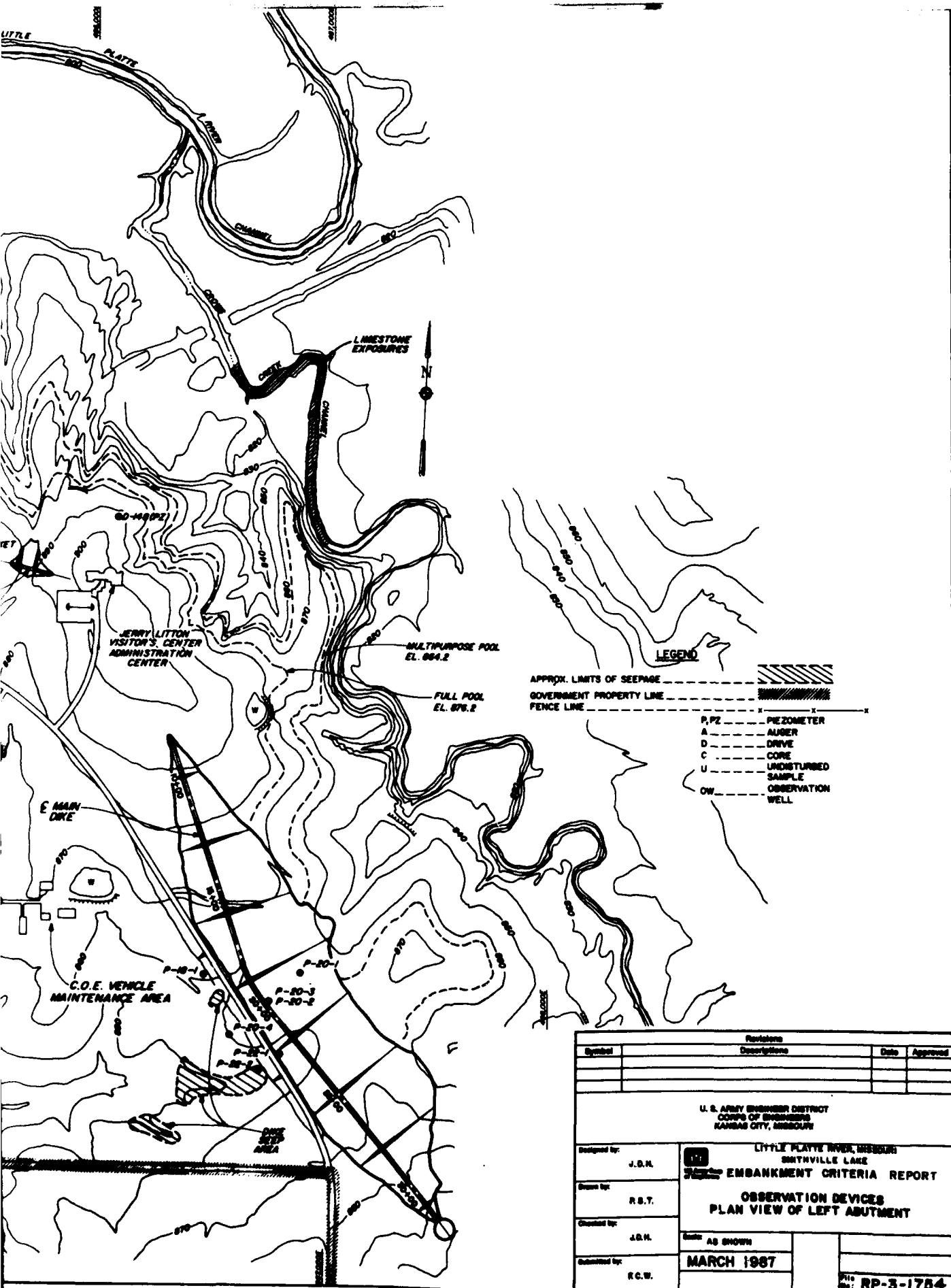
Reviewed by: P.C.M.

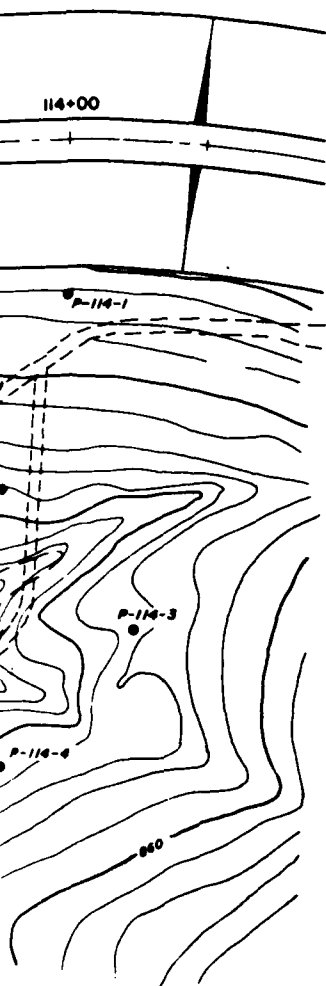
Date: AS SHOWN

MARCH 1987

File No: **RP-3-1763**



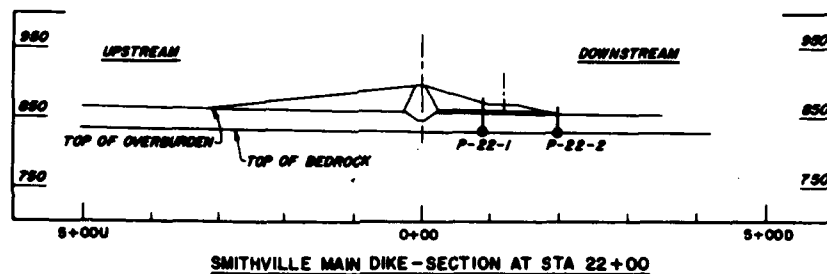
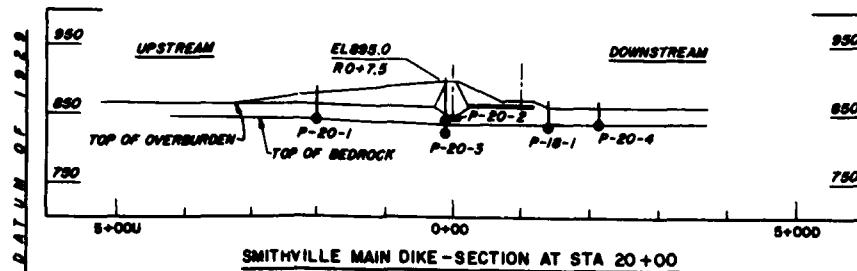
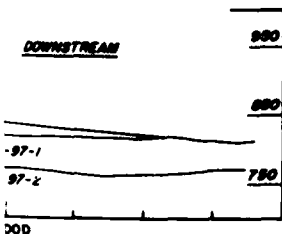




LEGEND

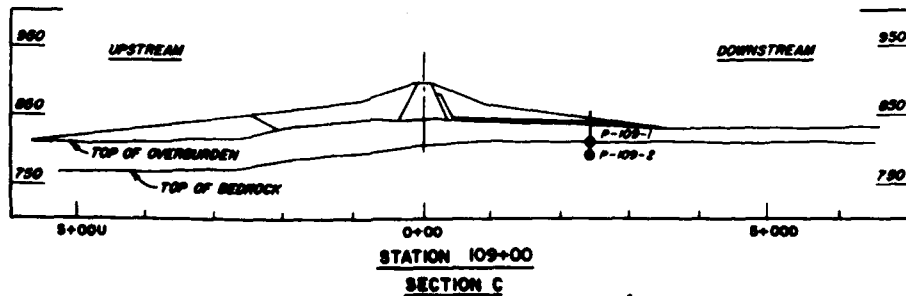
•-----PIEZOMETERS
 ---RELIEF WELLS

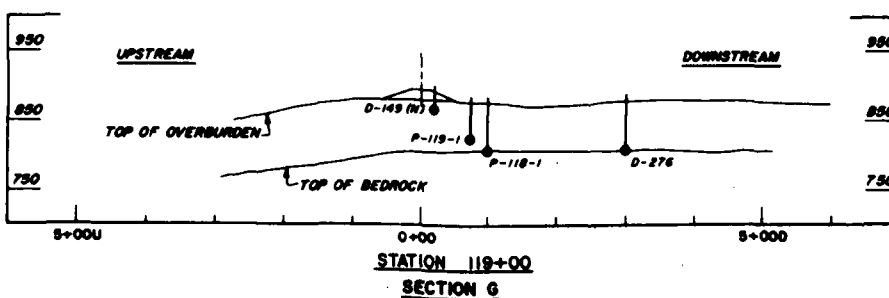
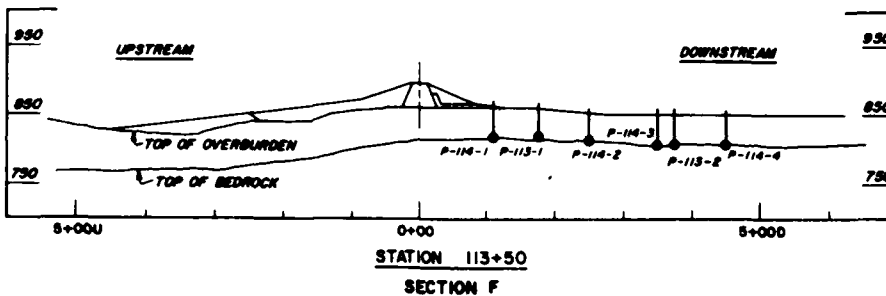
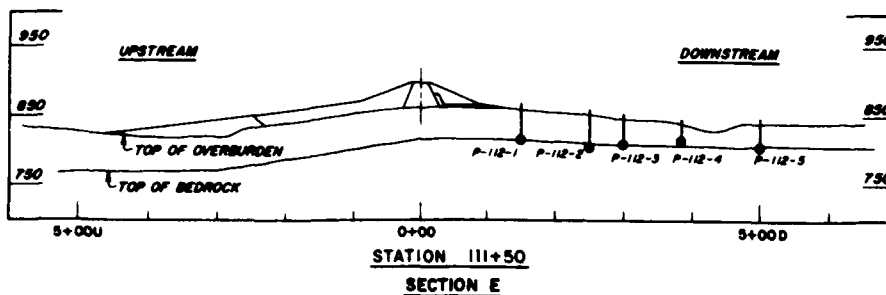
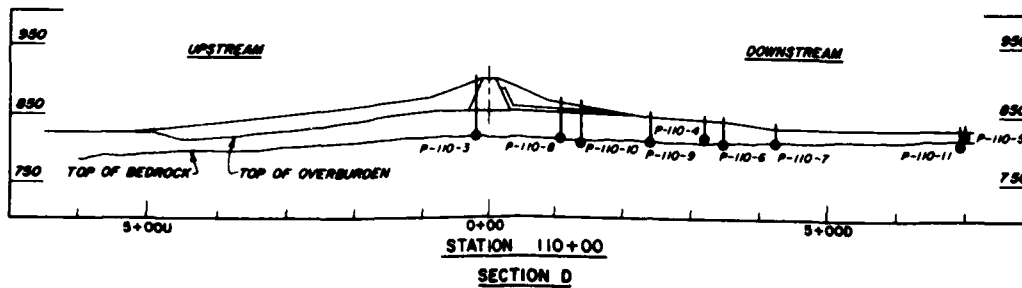
0 50' 100'
 SCALE 1" = 50'



Symbol	Revisions	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	J. B. H.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT INSET FROM PRECEDING PLATE PLAN OF EXPLORATIONS & PROFILE OF PZ'S ON LEFT ABUTMENT	
Drawn by:	R. B. T.		
Checked by:	J. B. H.		
Submitted by:	P. C. W.		
Scale: AS SHOWN		MARCH 1967 RP-3-1755	

SUPPLEMENT NO. 1 PLATE NO. 5

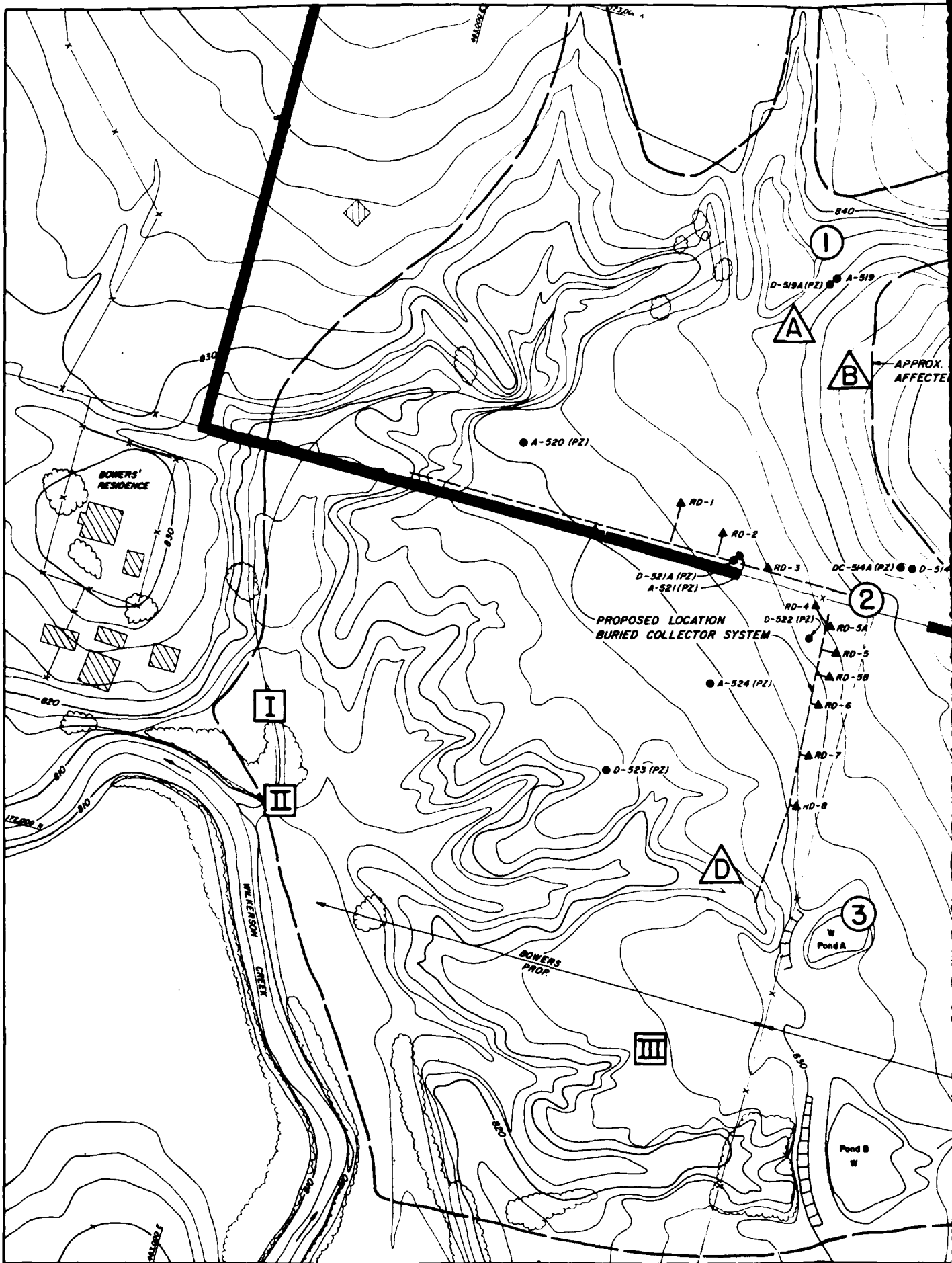


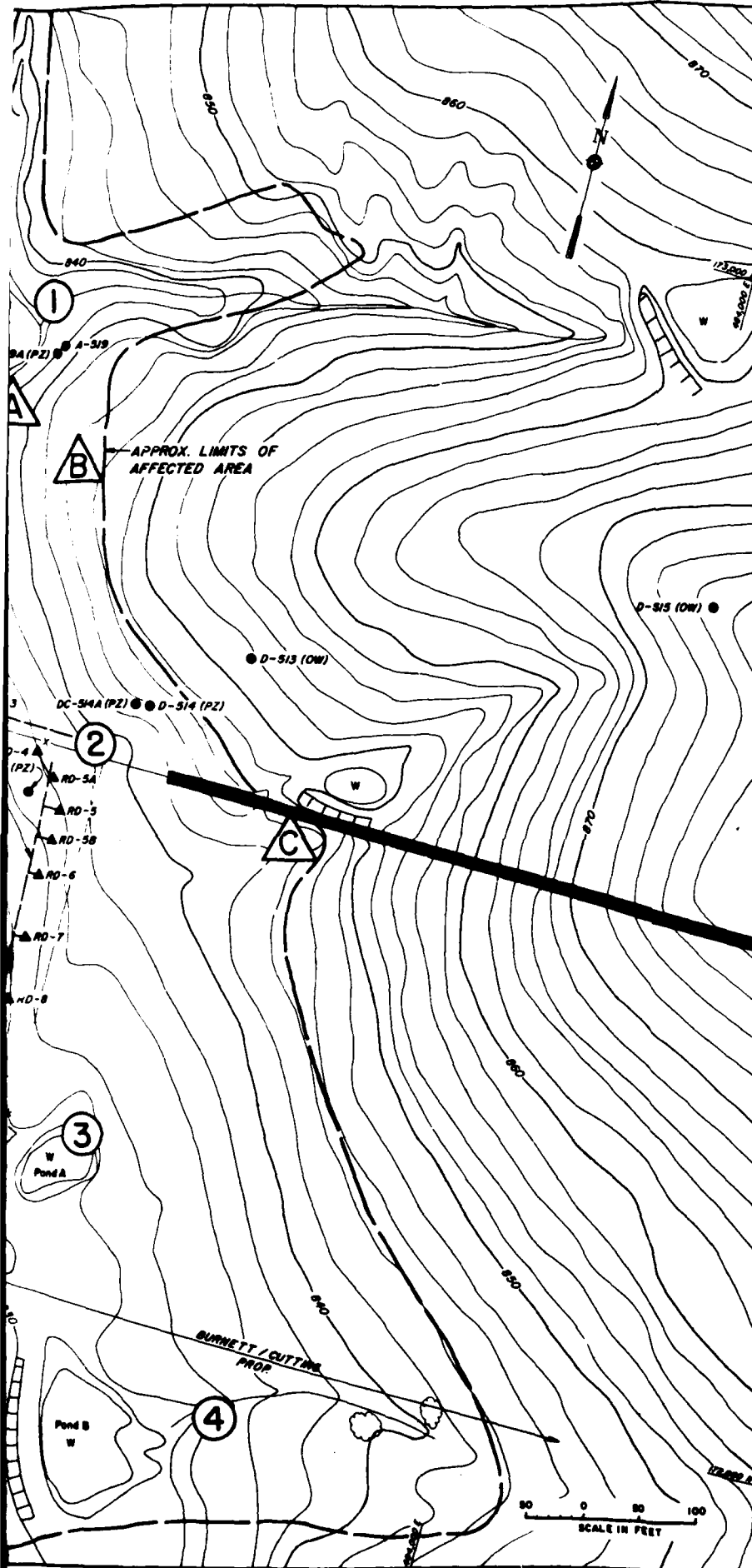


NOTE:
1. CROSS SECTIONS SHOWING LOCATION OF P.Z.'S
AT 10 STATIONS ACROSS THE DAM. LOCATION
SECTIONS A THROUGH G SHOWN ON PLATE NO.
4.

Revisions			
Symbol	Description	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	J. S. H.	LITTLE PLATTE RIVER, MISSOURI BENTVILLE LAKE EMBANKMENT CRITERIA REPORT	
Drawn by:	R. S. T.	INSTRUMENTATION CROSS SECTIONS	
Checked by:	J. S. H.	Scale	AS SHOWN
Submitted by:	P. C. M.	DATE	MARCH 1987
		File No.	RP-3-1756

SUPPLEMENT NO. 1 PLATE NO. 6





LEGEND

- RD RELIEF DRAIN
- (OW) OBSERVATION WELL
- (PZ) PIEZOMETER
- GOVT. PROP. LINE
- SEEPAGE - FLOWING
- SEEPAGE - DAMP
- EROSION - SEVERE HEADCUTTING

Revisions			
Symbol	Descriptions	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	J.M.M.	LITTLE PLATE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT	
Drawn by:	J.M.P.	BOWERS SEEP AREA	
Checked by:	J.D.H.	Scale:	AS SHOWN
Submitted by:	P.C.W.	MARCH 1987	
		7th Ed. RP-3-1757	

SUPPLEMENT NO. 1 PLATE NO. 7

OVERBURN variable thickness, loam clays.
Silty clays w/occasional lenses of fine sand
& occasional fine to coarse gravel, underlain
by clayey silts & fine to medium sands w/clay
& occasional gravel. Clayey, sandy gravels
w/limestone chert & quartzite cobbles &
boulders.

WYANDOTTE FM - ISLAND CREEK SHALE 26'
Soft, platy, silty clay, calcareous, few phosphatic nodules in upper 15', gray to gray. Soft to soft, clayey, w/ high angle fractures (approximately 15 to 16' above base of Island Creek). Moderately hard to hard, dense, medium to thin bedded, brown - gray mudstone w/ few sand partings & occasional very thin partings (approximately 5 to 7' above base of Island Creek).

WYANDOTTE FM-ARGENTINE LIMESTONE 5
Moderately hard to hard, dense, thin wavy
bedded, brown-gray, w/ numerous soft
dk gray wavy shale partings & bands.

WYANDOTTE FM - QUINDARO SHALE . 0.5'
Soft, very calcareous, gray, fossiliferous.

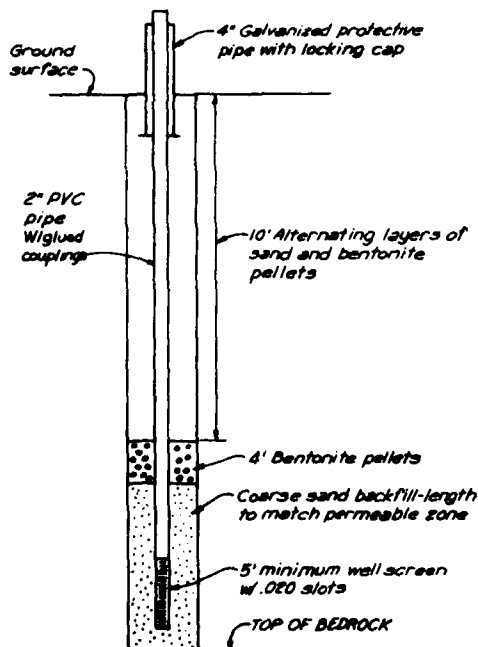
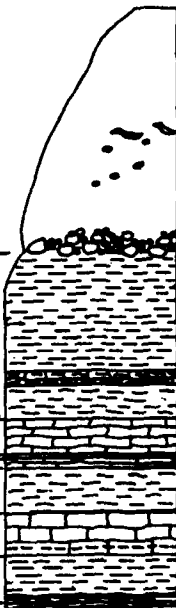
WYANDOTTE FM - FRISBIE LIMESTONE 1'
Moderately hard to hard, dense, thick-bedded,
subtle & fossiliferous inclusions, gray-brown.

LANE FM - SHALE average thickness 7'
Soft, fissile, slightly calcareous, dark gray.

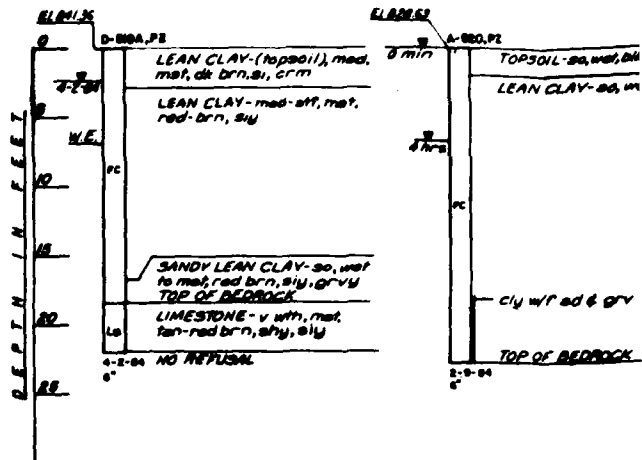
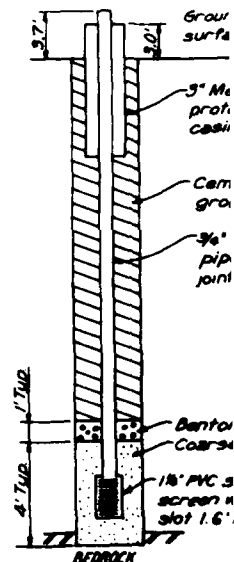
NOLA FM - RAYTOWN LIMESTONE 6'
Moderately hard to hard, crystalline to dense fossiliferous & massive in upper part. Thin-bedded w/interlaminated soft, calcareous shale w/persistent soft to soft clayey shale (approximately 1' to 1.5' above base of Raytown) overlying moderately hard, dense to finely crystalline fossiliferous limestone w/shaly partings in lower part.

7' IDLA FM-MUNCIE CREEK SHALE
Soft to moderately hard w/ occasional v soft
thin laminae, platy, dk gray to black,
calcareous, anal to black carbonaceous sh
near base.

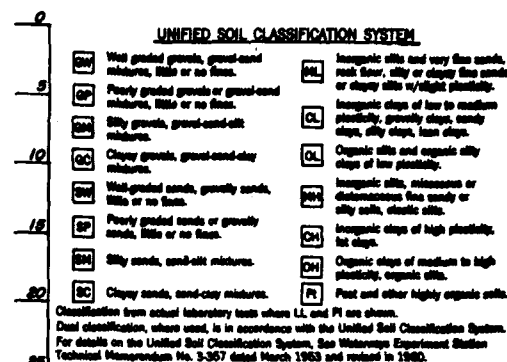
104A FM-PAOLA LIMESTONE
Moderately hard to hard, finely crystalline,
massive w/ occasional argillaceous partings,
grey fossiliferous.



TYPICAL 2-INCH INSTALLATION
OBSERVATION WELL

[illegible]

TYPICAL 3/4"-HIGH PIEZ INSTALLATION



**TERMS FOR CONSISTENCY OF
SOIL AND HARDNESS OF BEDROCK**

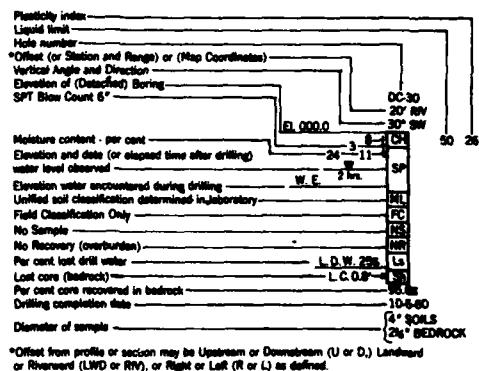
SOIL

Consistency	Estimated Unconfined Compressive Strength (Tons per square foot)
Very soft	< 0.25
Soft	0.25-0.5
Medium	0.5-1.0
Stiff	1.0-2.0
Very stiff	2.0-4.0
Hard	> 4.0

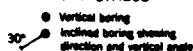
SCALE OF HARDNESS

Very soft or plastic	Can be indented easily with thumb.
Soft	Can be scratched with fingernail.
Moderately hard	Can be scratched easily with nail; cannot be scratched with fingernail.
Hard	Difficult to scratch with nail.
Very Hard	Cannot be scratched with nail.

LEGEND FOR LOGS OF BORINGS

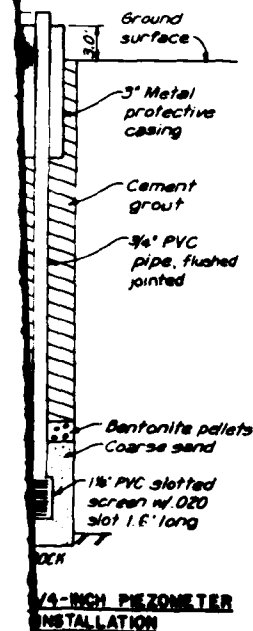


MAP SYMBOL



CODE DESIGNATION

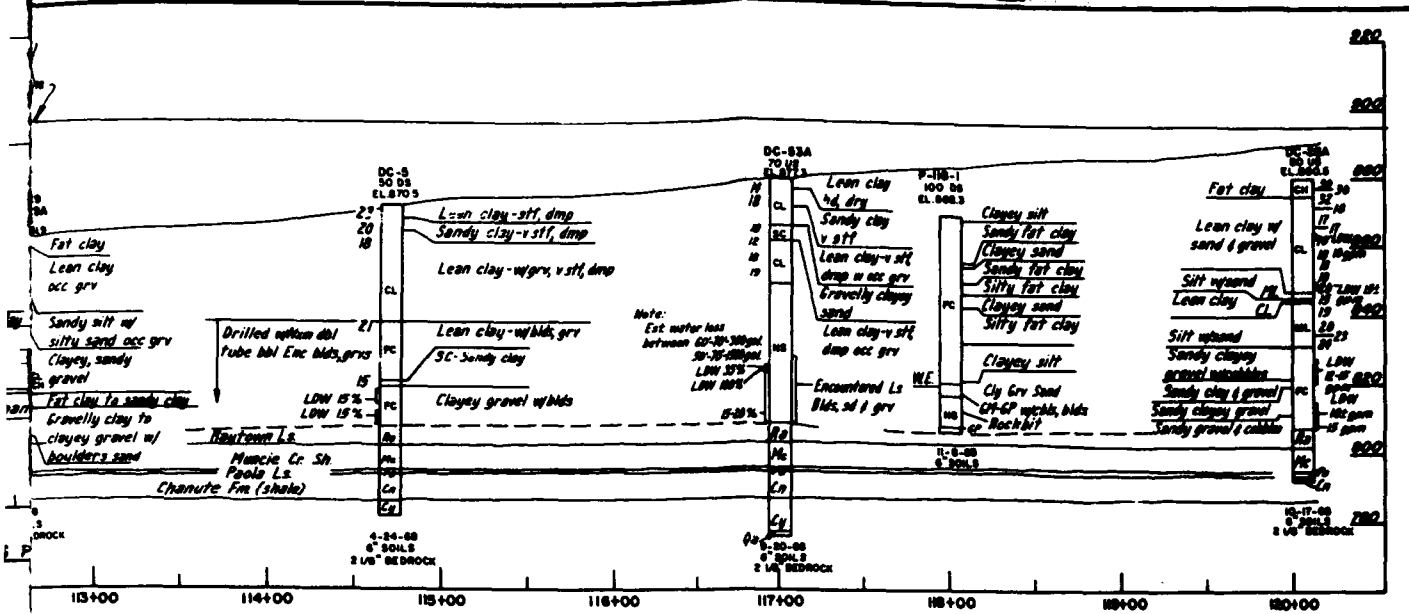
- D Drive sample hole
C Core hole
TP Test pit (includes power auger
24" or larger diameter)
U Undisturbed sample hole
A Auger hole-hand or power
sugar less than 24" diameter.
NS Not Sampled
(Field Classification from
cuttings only)
FS Field Section of outcrop.
O/O Observation well
RD Rollid drain
P Piezometer no.
PZ Piezometer



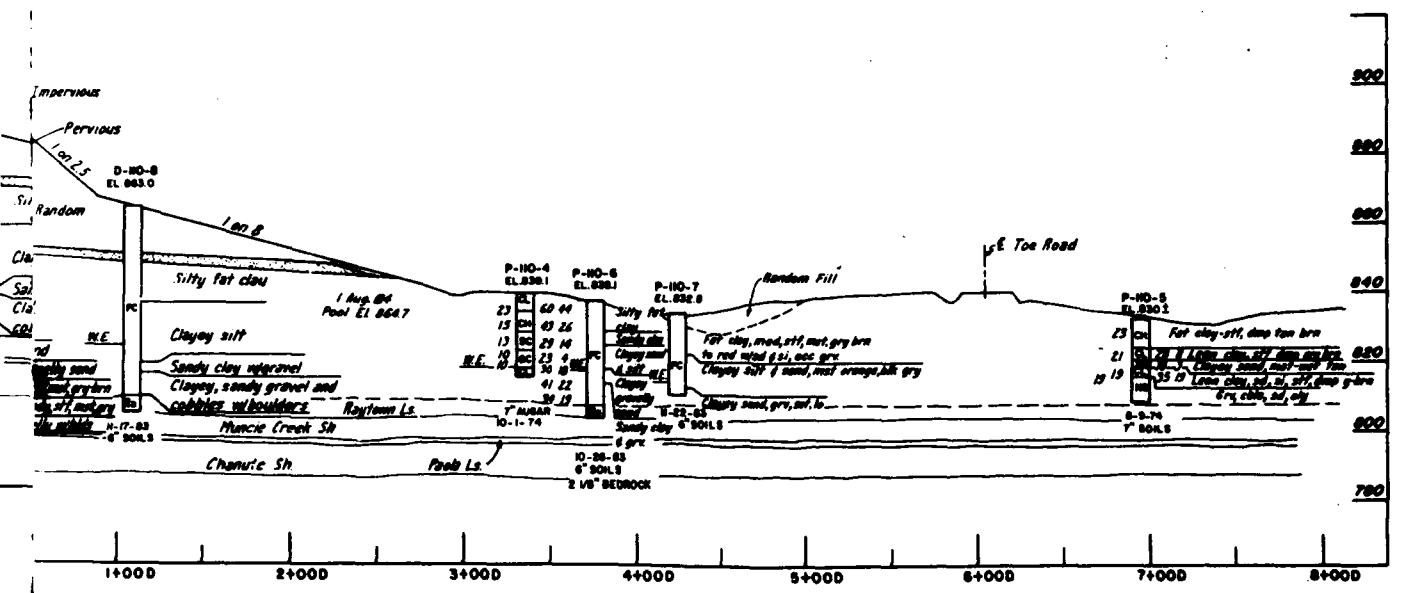
Symbol	Descriptions	Date	Approved

**U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI**

Designed by: J. M. M. Drawn by: R. L. D. Checked by: J. D. H. Submitted by: P. C. M.	LITTLE PLATTE RIVER, MISSOURI- SMITHVILLE LAKE  EMBANKMENT CRITERIA REPORT GEOLOGIC COLUMN AND LEGEND, DETACHED BORINGS, AND TYPICAL INSTALLATION DIAGRAMS		
	Status: AS SHOWN		File:
	MARCH 1967		RP-3-1758



MENT DAM AXIS PROFILE



WT
00
C
114

Revisions			
Symbol	Description	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: W.F.L.

Drawn by: E.E.R.

Checked by: J.B.M.

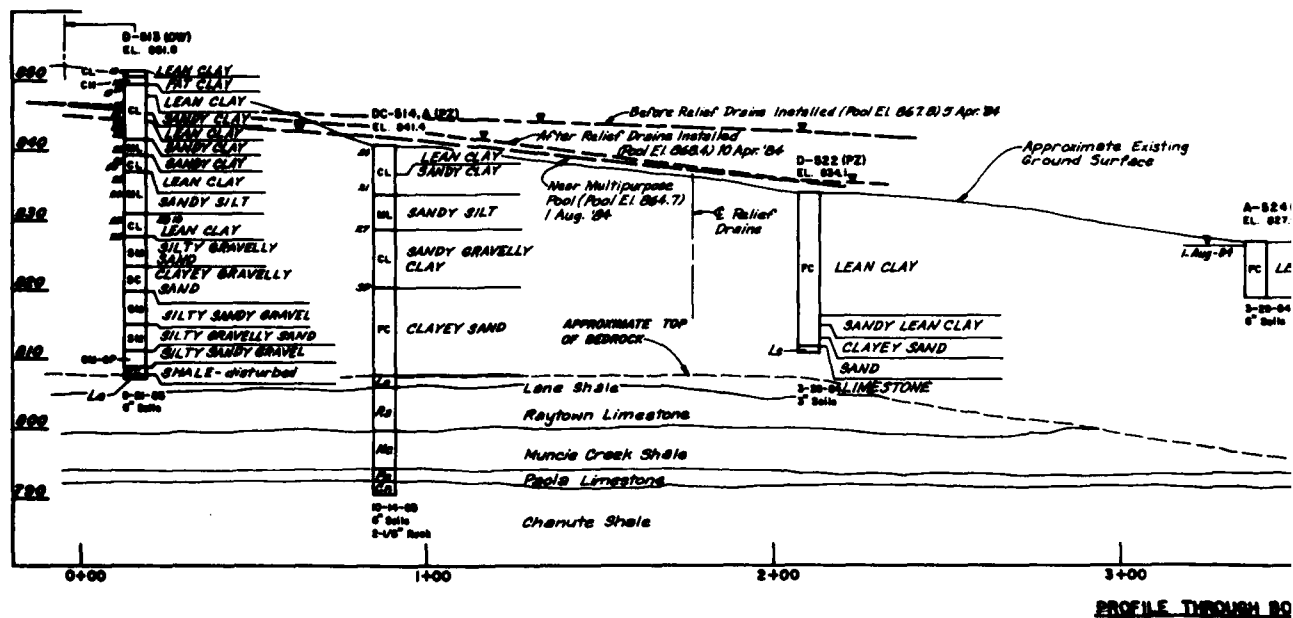
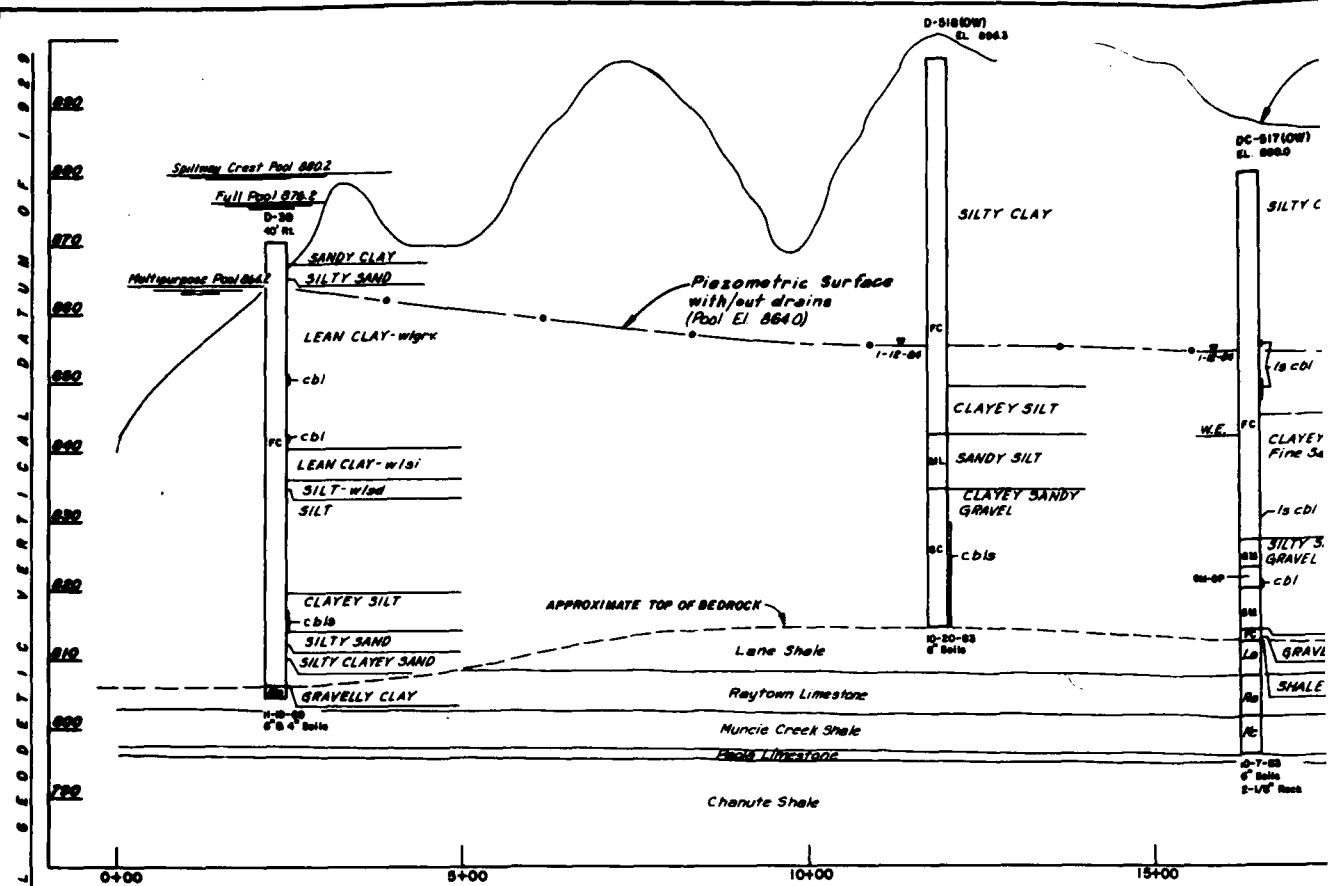
Submitted by: R.C.W.

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
EMBAKMENT CRITERIA REPORT
LEFT ABUTMENT DAM AXIS PROFILE;
EMBAKMENT SECTION-STATION 110+00

Scale: AS SHOWN

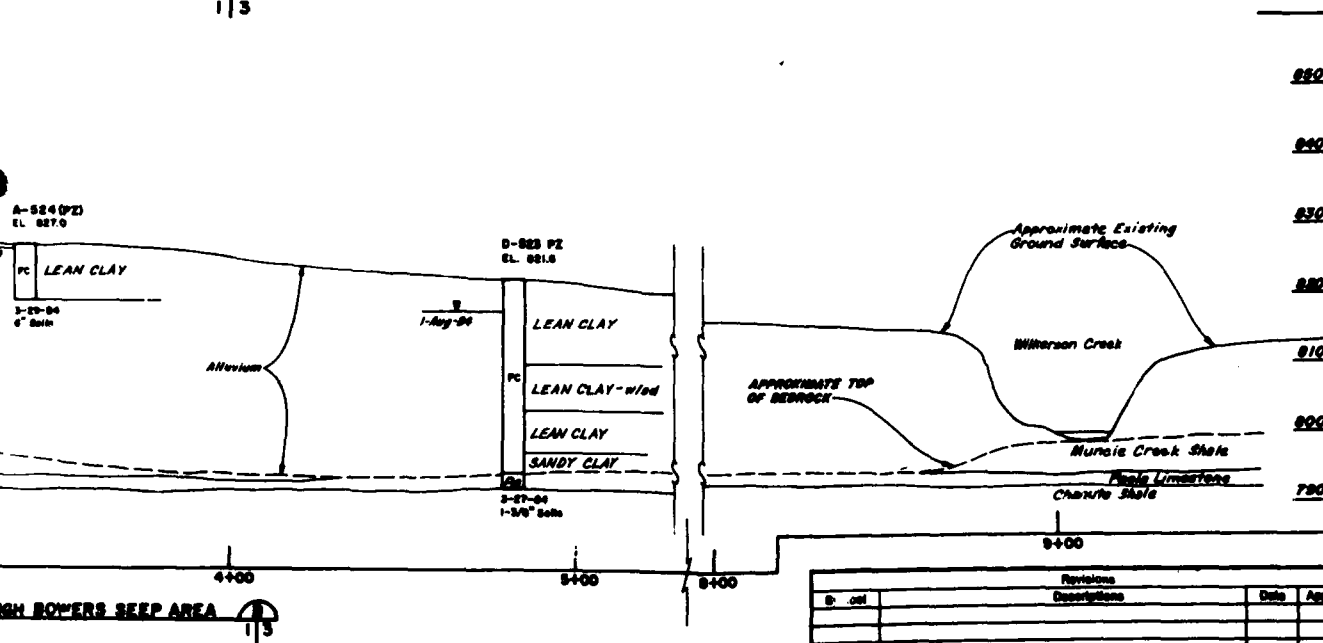
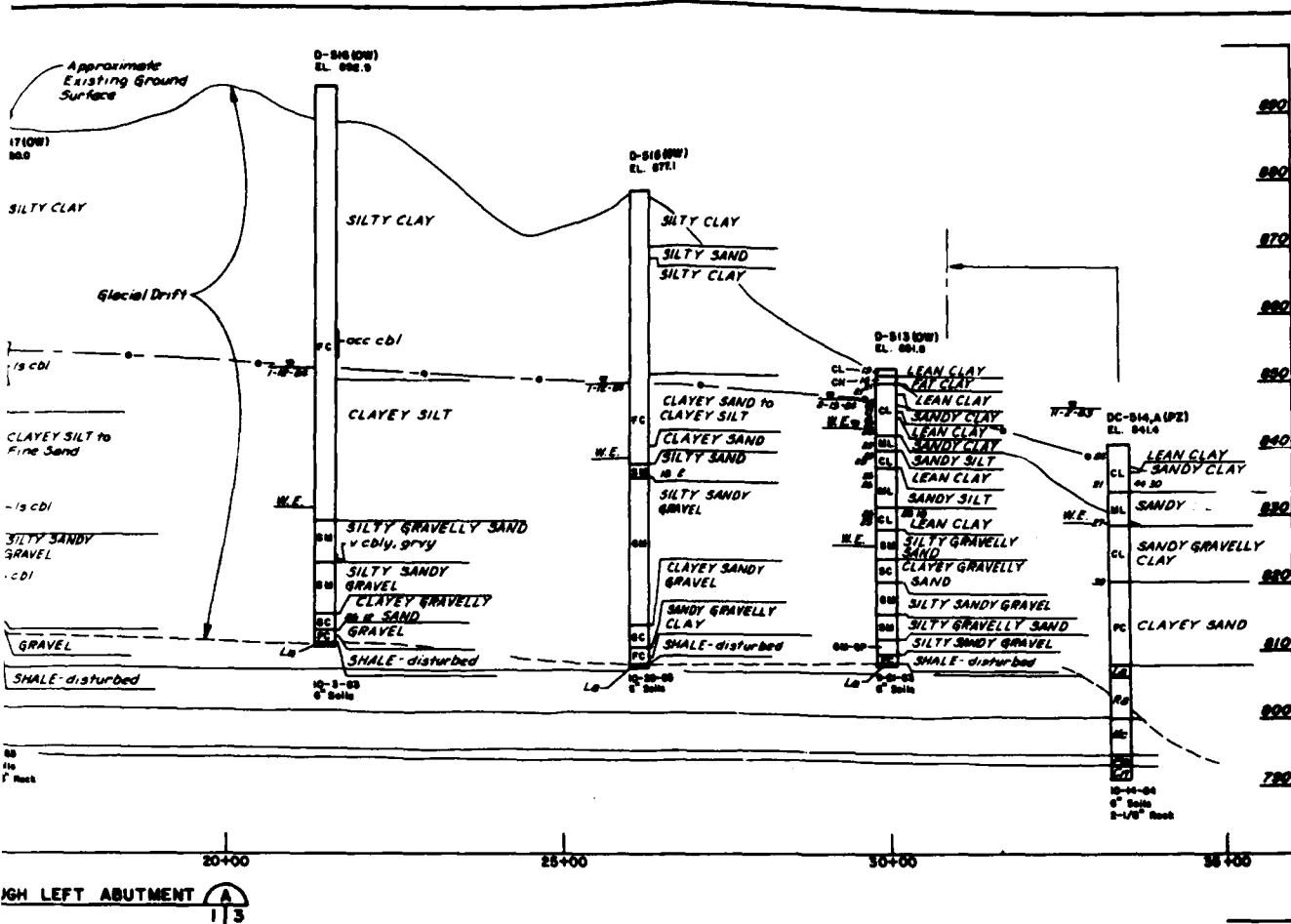
MARCH 1987

RP-3-1759



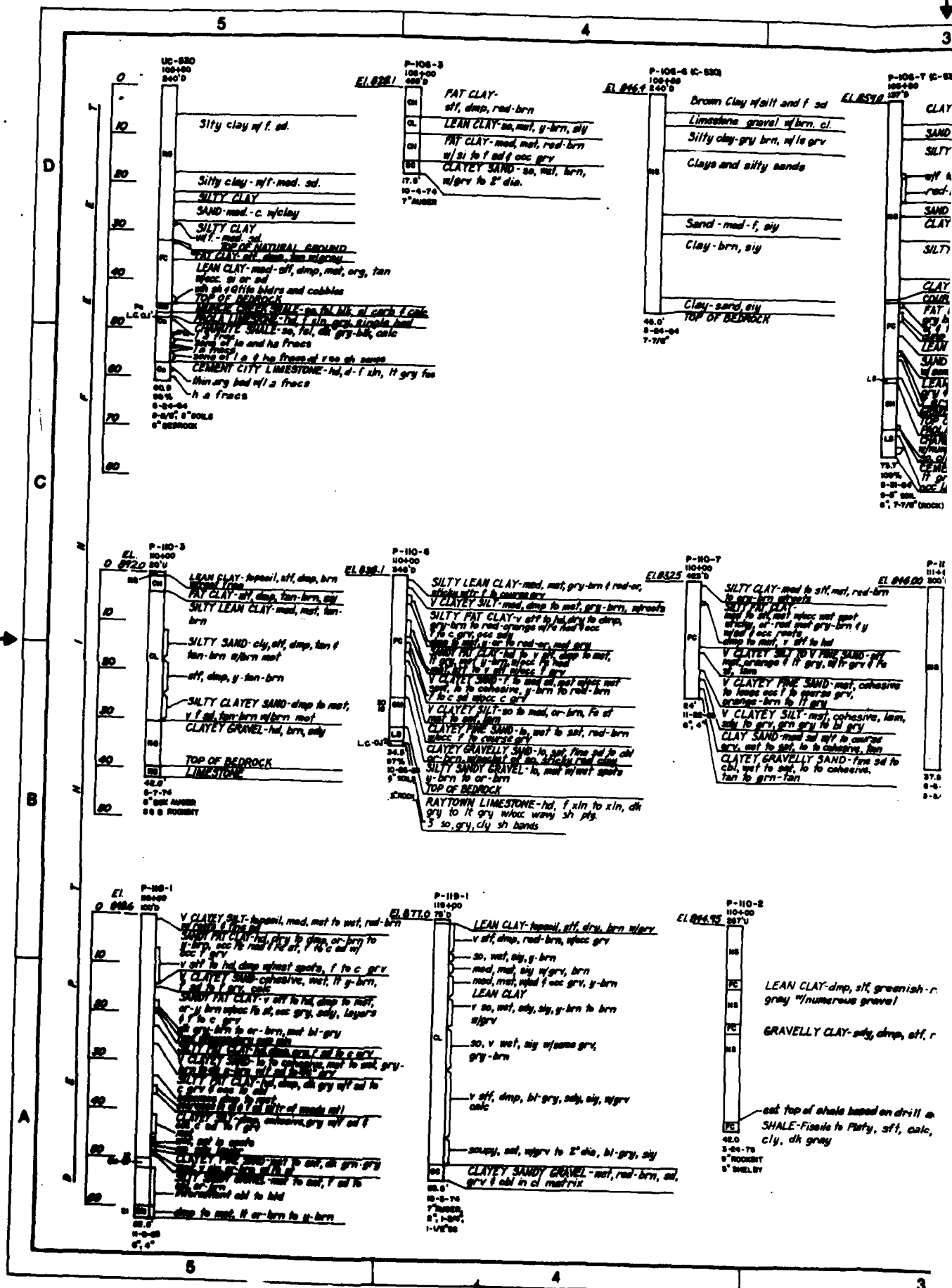
Notes:

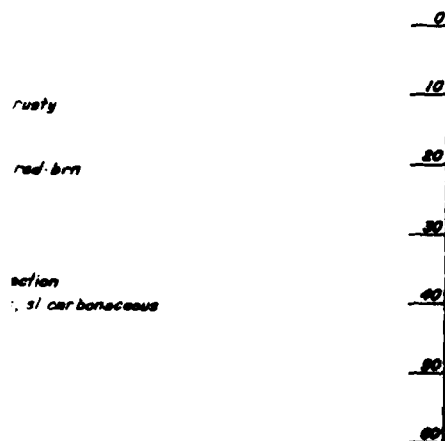
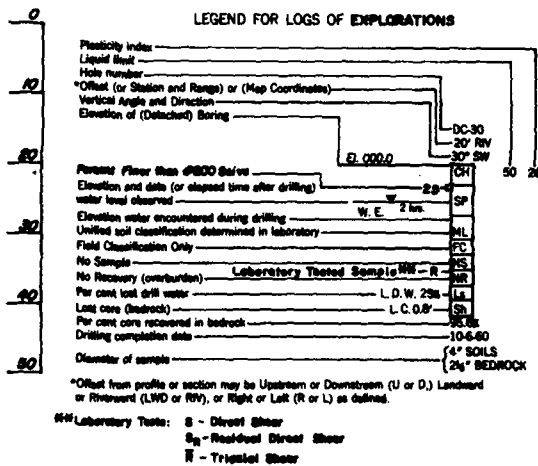
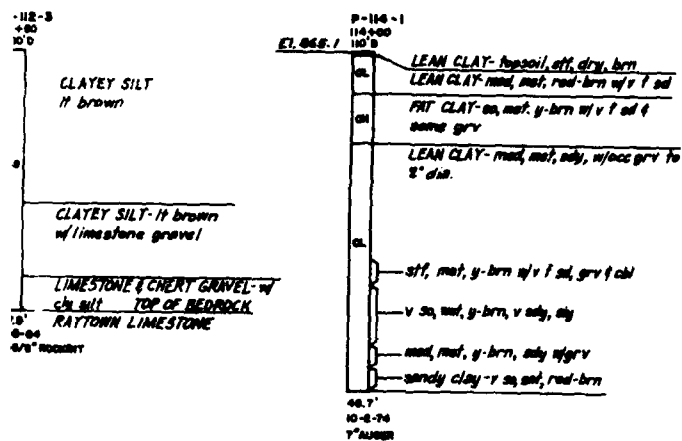
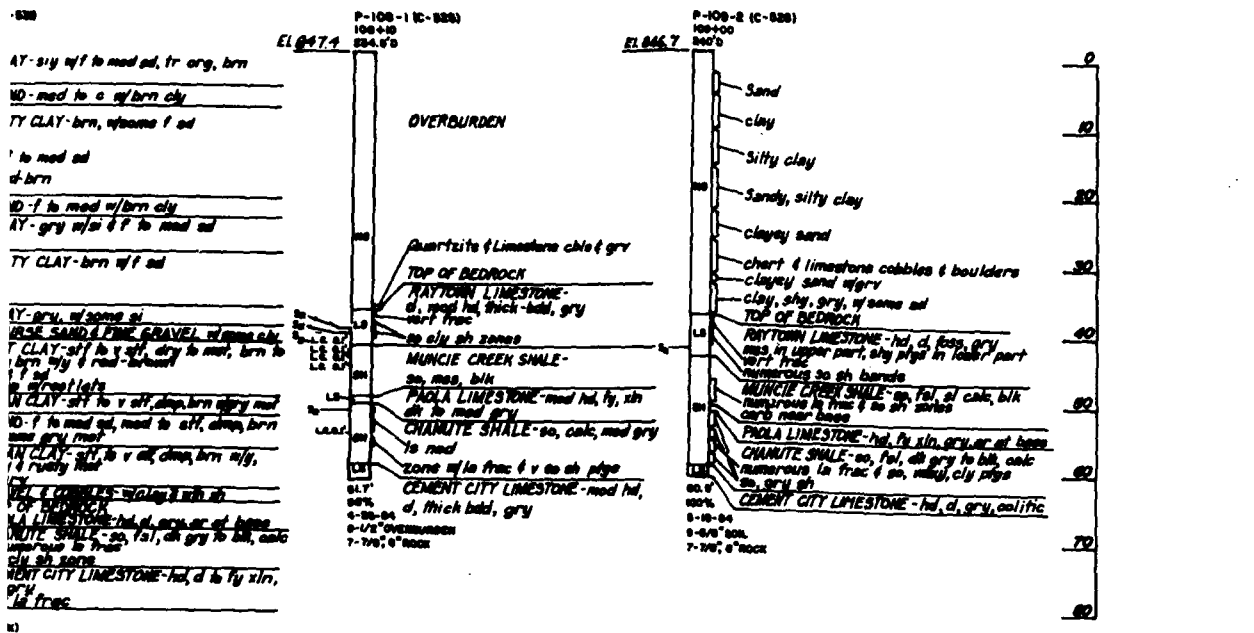
1. Boundary between glacial drift and alluvium is not defined.
2. Piezometric surface shown is measured in the pervious basal layer of the overburden.



Revisions		Date		Approved	
No.	Description	Date	Approved		

U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI					
Date: and by: J. M. M.		 LITTLE PLATTE RIVER, MISSOURI SANTIVILLE LAKE EMBANKMENT CRITERIA REPORT			
Drawn by: J. G. B.		PROFILES-LEFT ABUTMENT BOWERS SEEP AREA			
Checked by: J. G. B.		Scale: AS SHOWN			
Submitted by: F. C. M.		MARCH 1967 FOR FILE RP-3-1761			

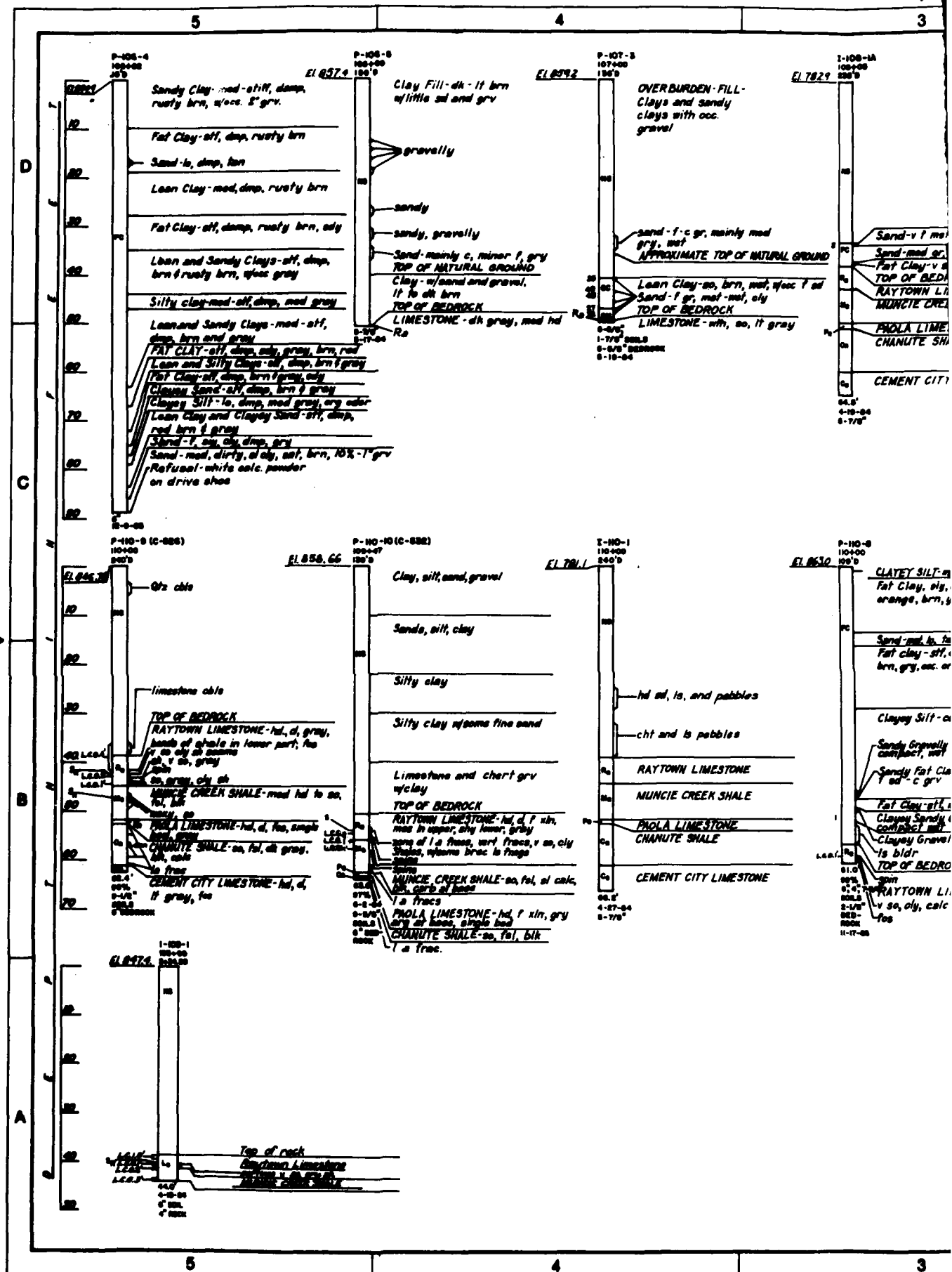


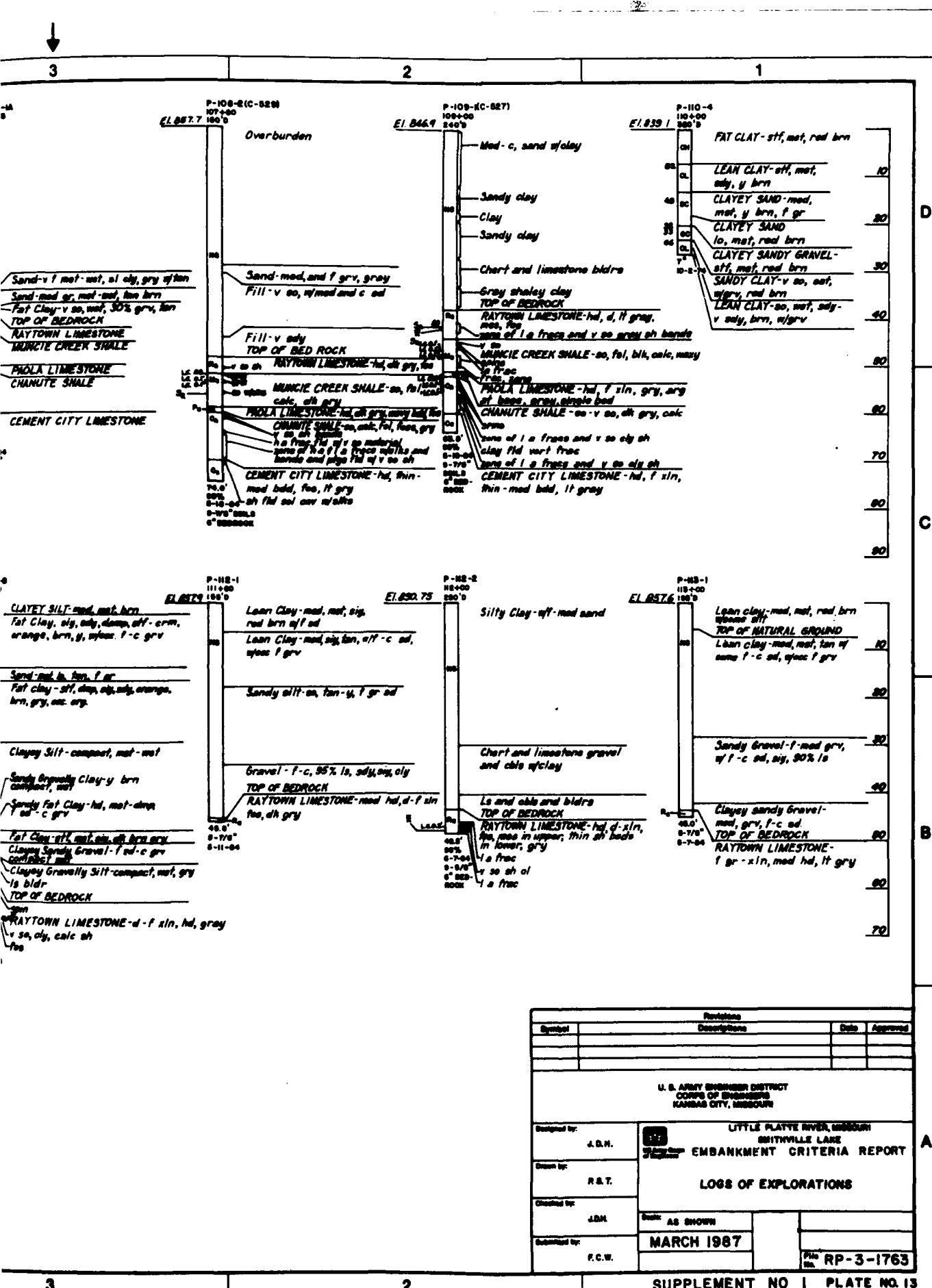


Symbol		Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI				
Designed by:	J. D. H.	 LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT		
Drawn by:	P. S. T.			
Checked by:	JDM			
Submitted by:	P. C. M.			

LOGS OF EXPLORATIONS AND LEGEND	Date: MARCH 1987	Scale: AS SHOWN	File No. RP-3-1782
---	--------------------------------	-------------------------------	----------------------------------





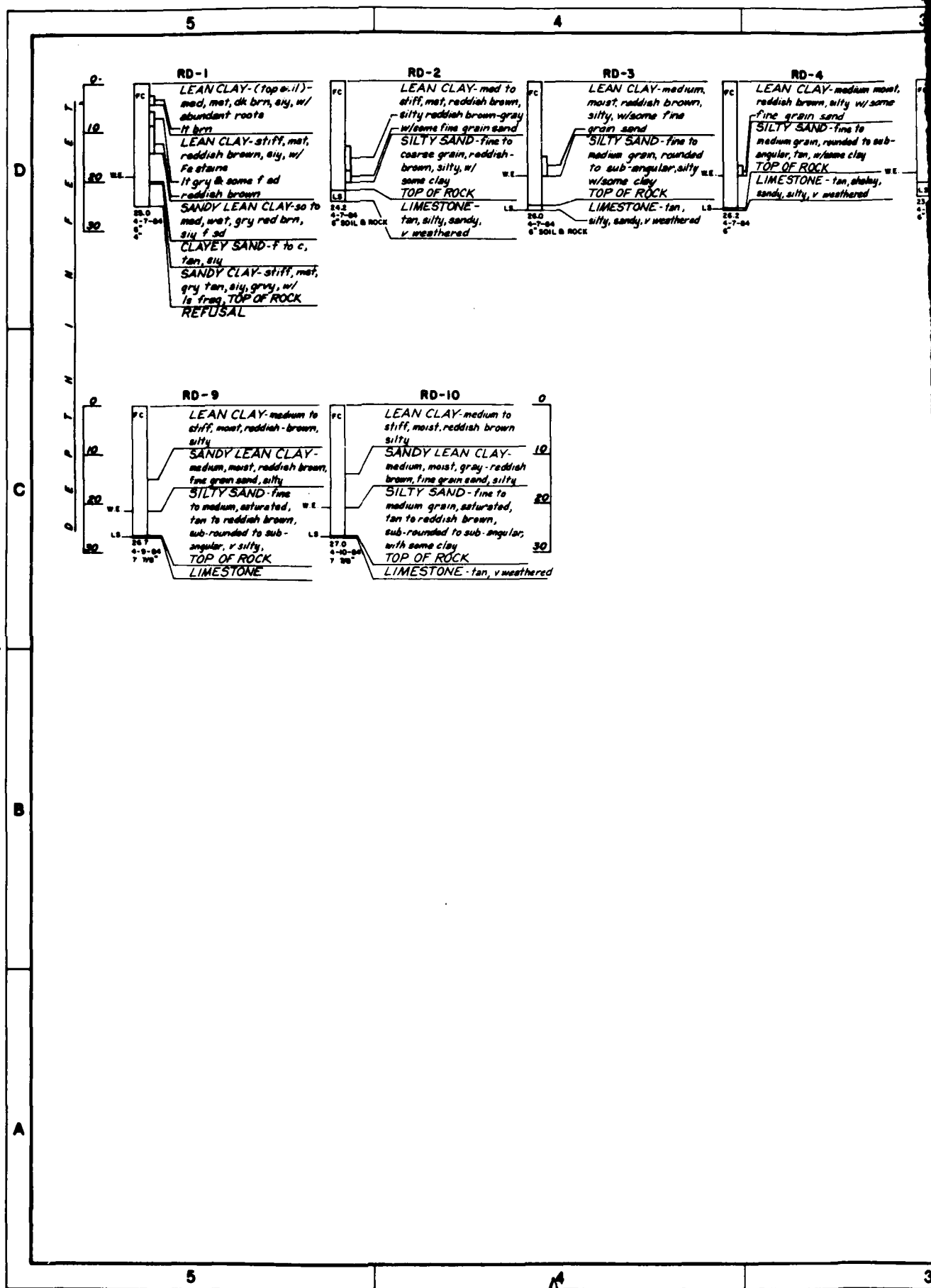
Revisions			
Symbol	Description	Date	Approved

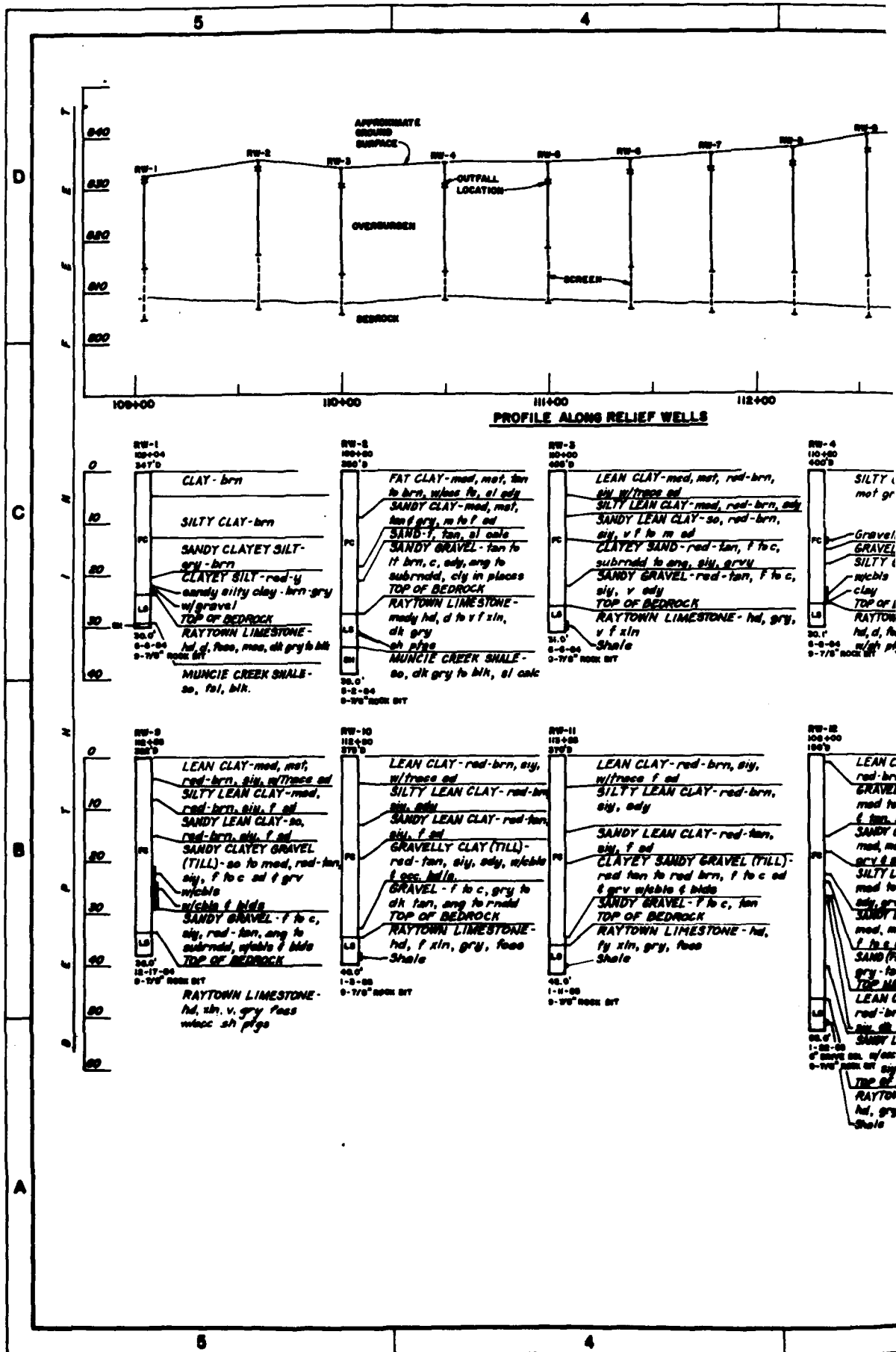
**U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI**

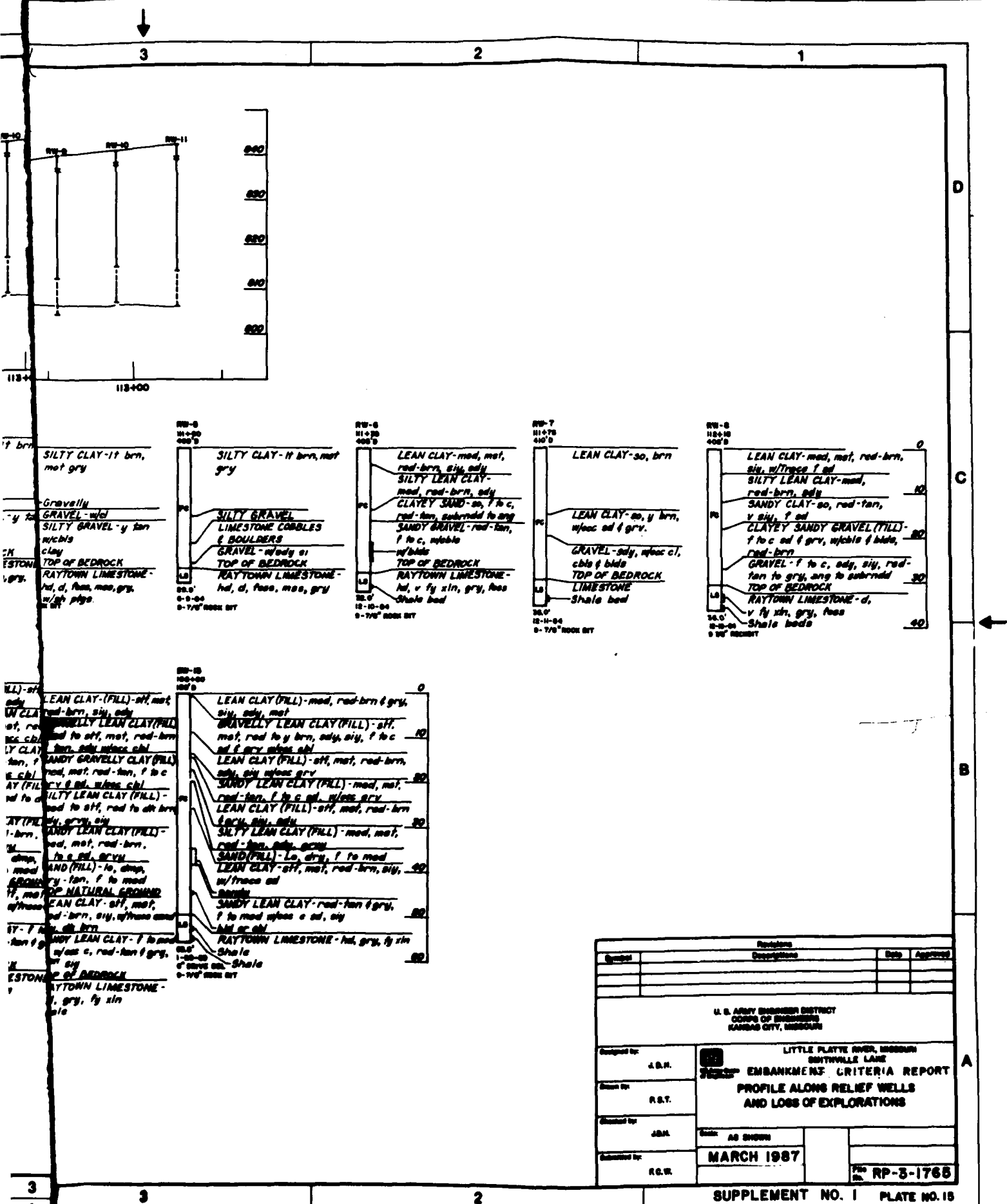
**LITTLE PLATTE RIVER, MISSOURI
SHITWILLIE LAKE
EMBANKMENT CRITERIA REPORT**

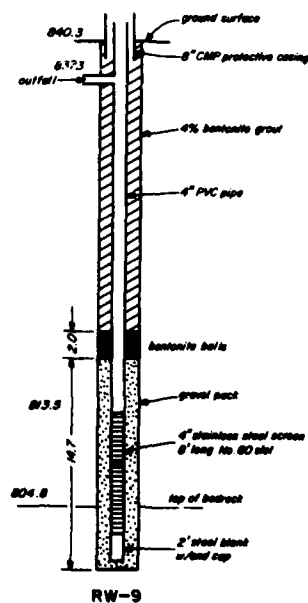
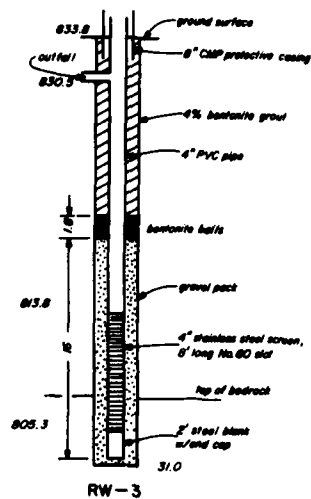
LOGS OF EXPLORATIONS

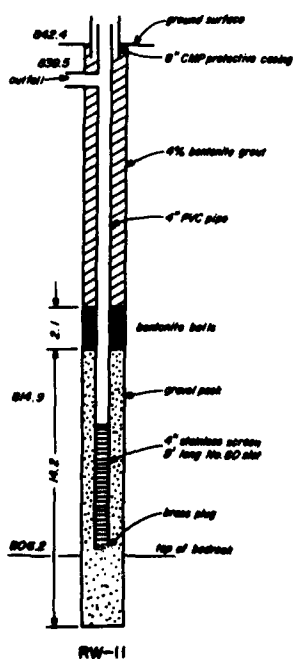
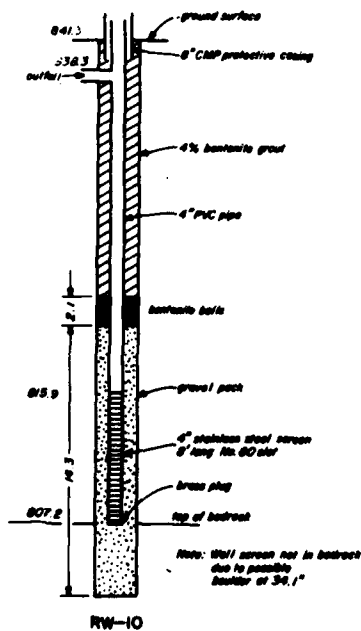
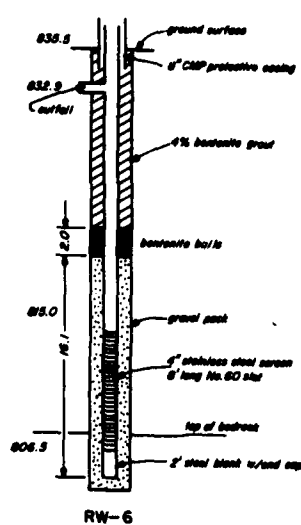
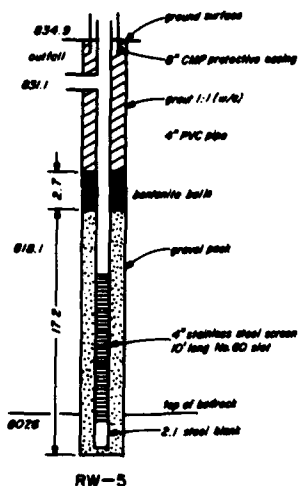
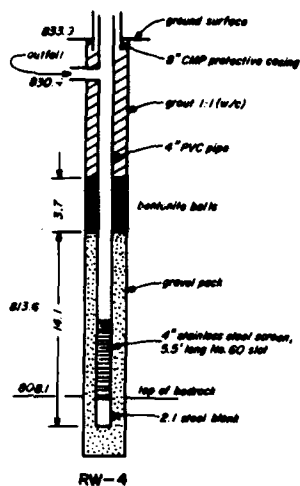
Designed by:	J. D. H.	Scale: AS SHOWN	MARCH 1967	File No. RP-3-1763
Drawn by:	R. B. T.			
Checked by:	J. D. H.			
Submitted by:	P. C. W.			











Revisions		Date	Approved
Number	Description		

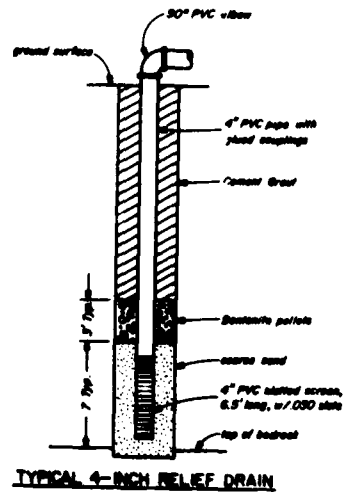
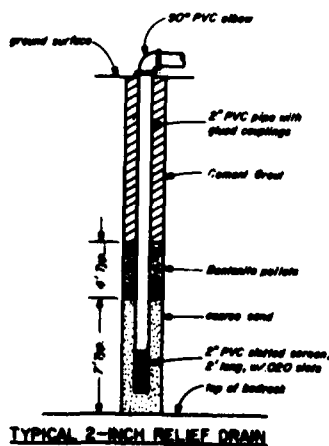
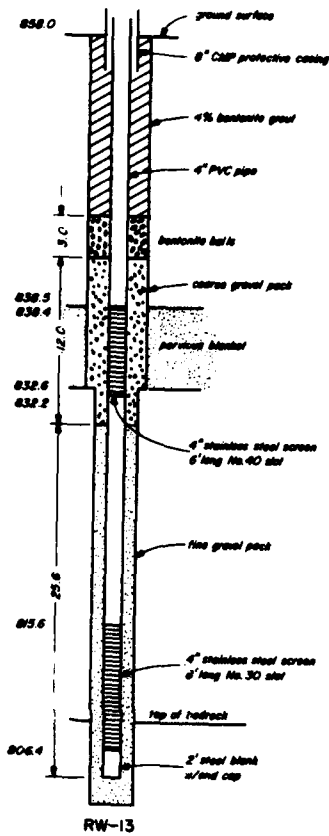
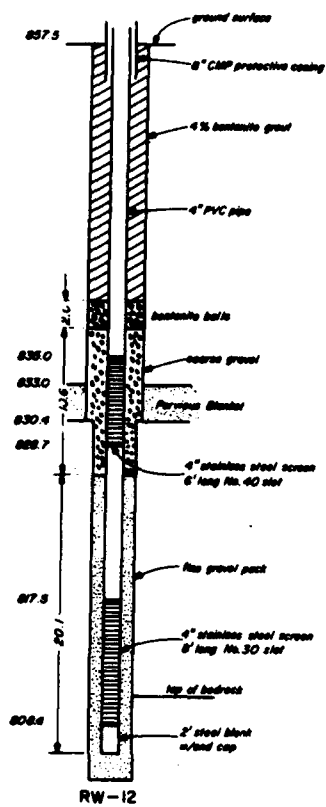
**U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI**

Designed by: J. B. H.	 LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT INSTALLATION DETAILS OF RW-1 THRU RW-11	Scale: AS SHOWN	
Drawn by: J. M. T.			
Checked by: J. G. M.			
Submitted by: F. C. W.			

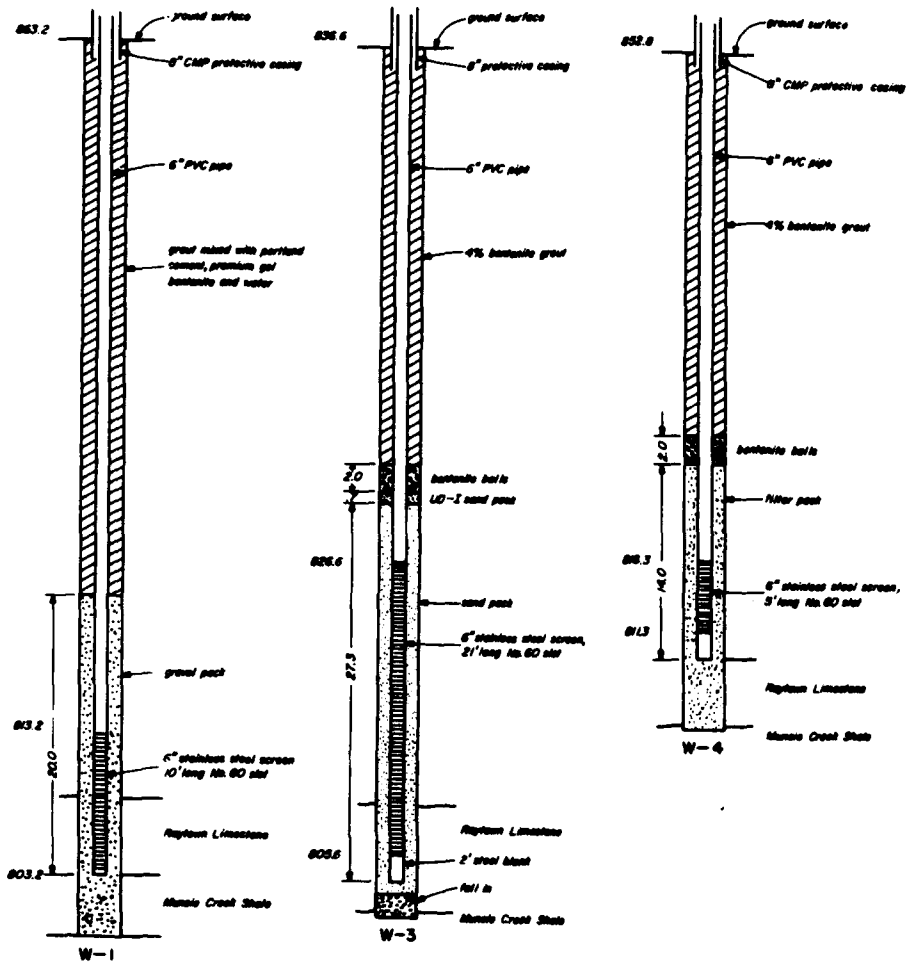
MARCH 1967

File No. **RP-3-1766**

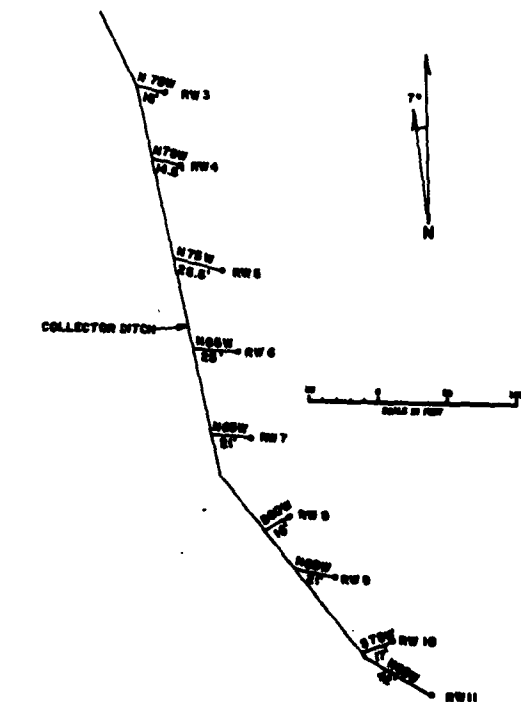
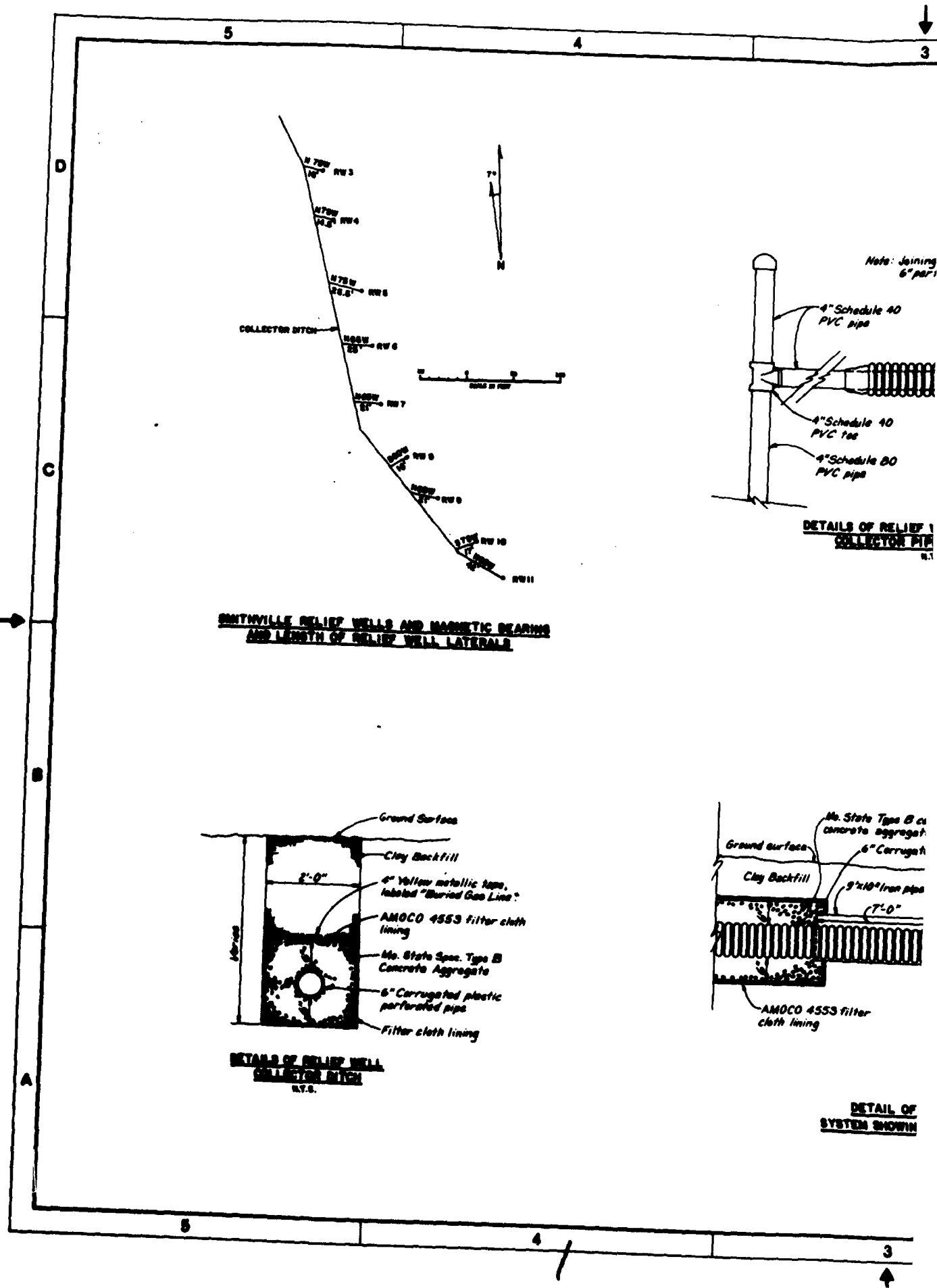
RW-12 AND RW-13
WELLS INTERCEPTING PERVIOUS BLANKET



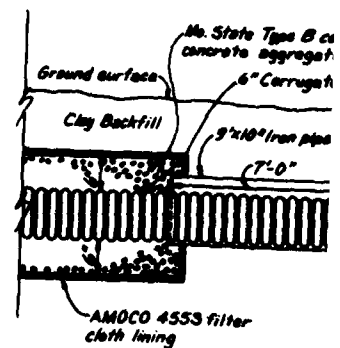
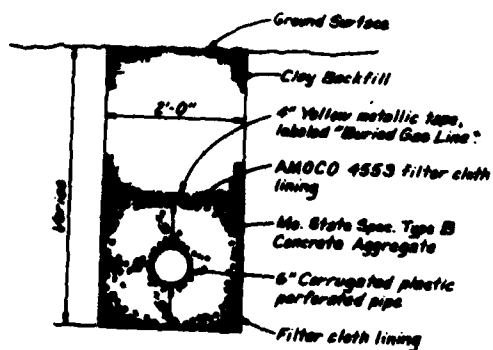
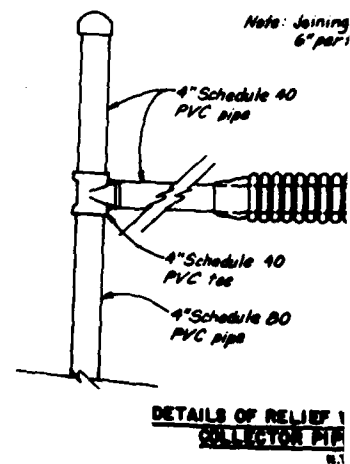
W-1, W-3, AND W-4
PUMPED WELLS

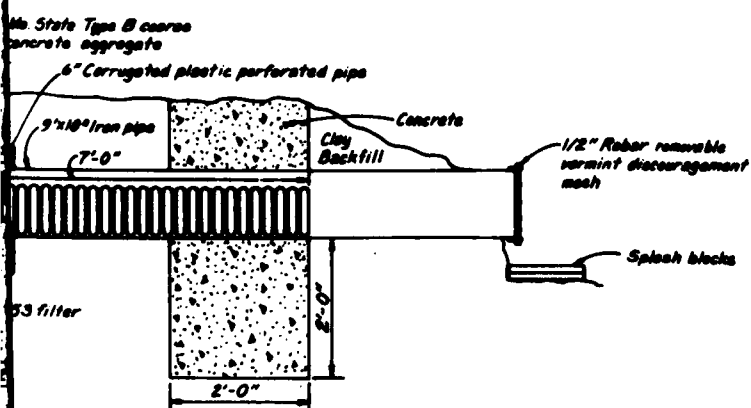
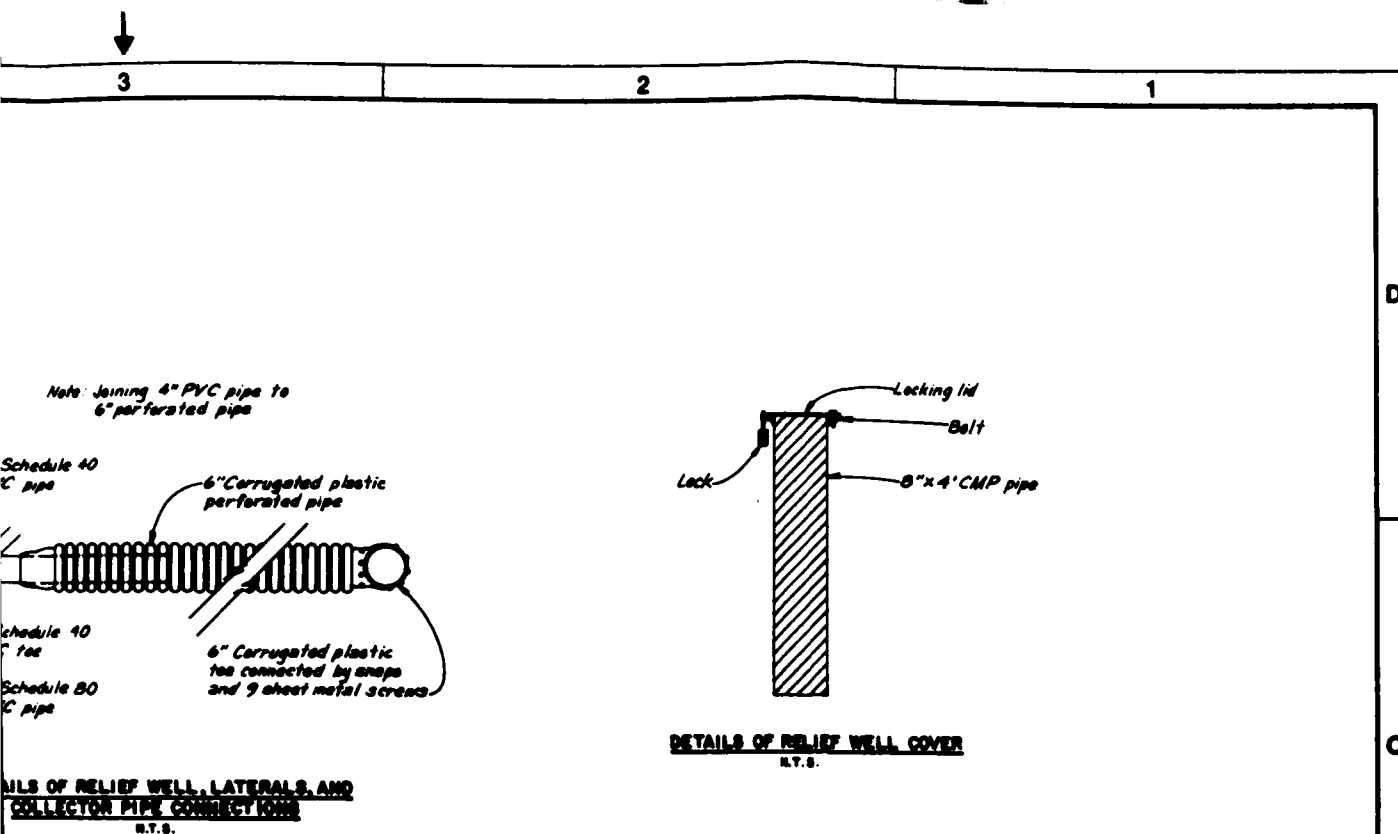


Author	Reviewer	Date	Approved
U.S. ARMY ENGINEER DISTRICT DISTRICT OF COLUMBIA WASHINGTON, D.C.			
Designed by:	LITTLE PLATE OVER, MISSOURI SOUTHVILLE LANE		
Drawn by:	ENGINEER EMBANKMENT CRITERIA REPORT		
Checked by:	INSTALLATION DETAILS OF RW-12 6 RW-12, PUMPED WELLS AND 2-1/2" AND 4-1/2" RELIEF DRAINS		
Reviewed by:	AS SHOWN		
Submitted by:	MARCH 1987		
	RP-3-1767		



**SMITHVILLE RELIEF WELLS AND MAGNETIC BEARINGS
AND LENGTH OF RELIEF WELL LATERALS**

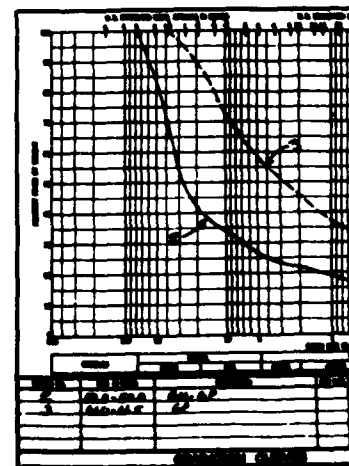
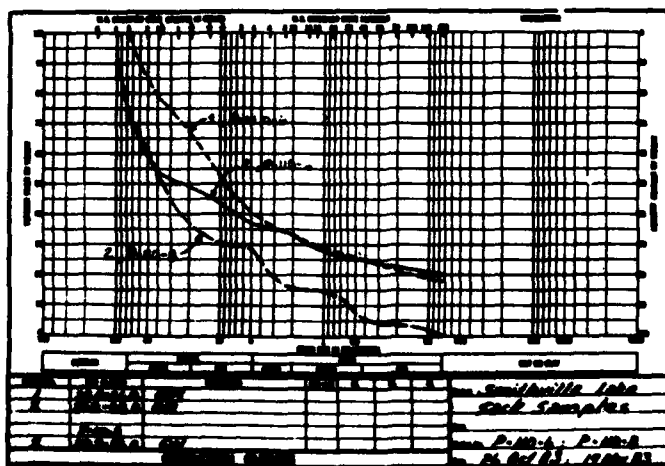
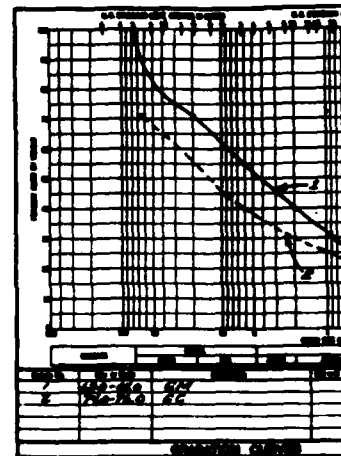
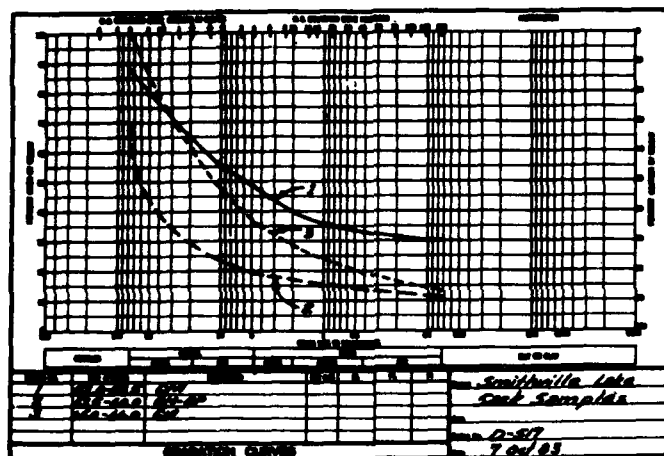
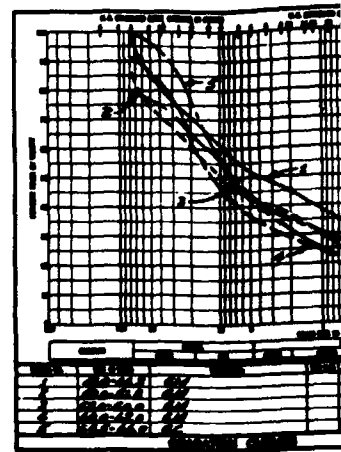
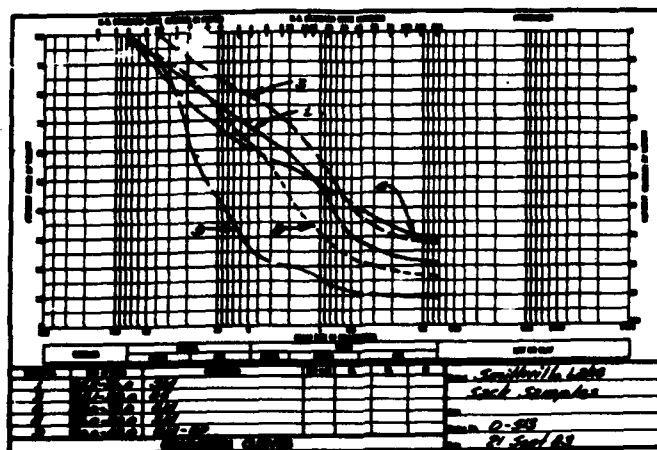


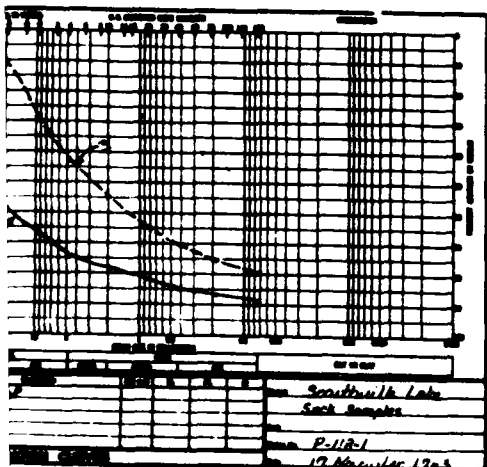
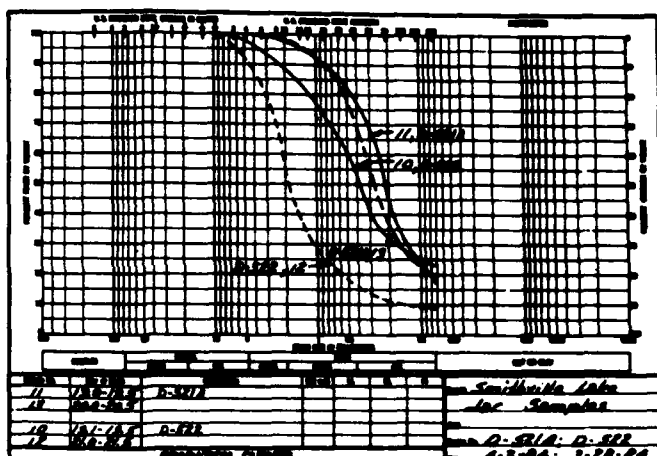
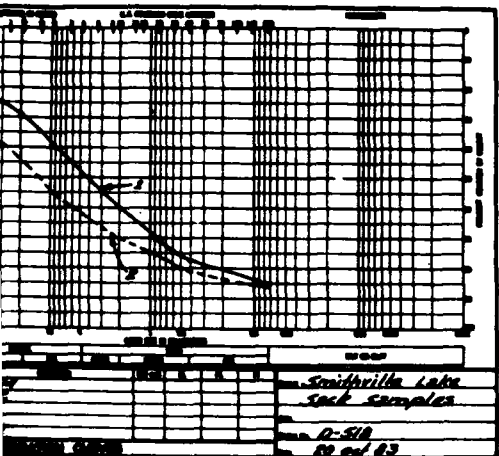
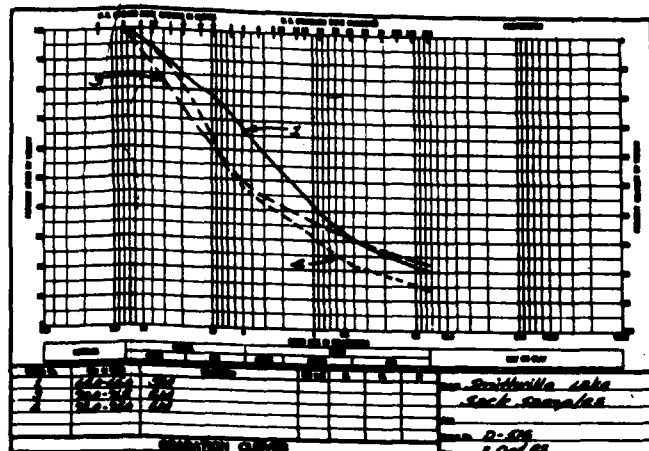
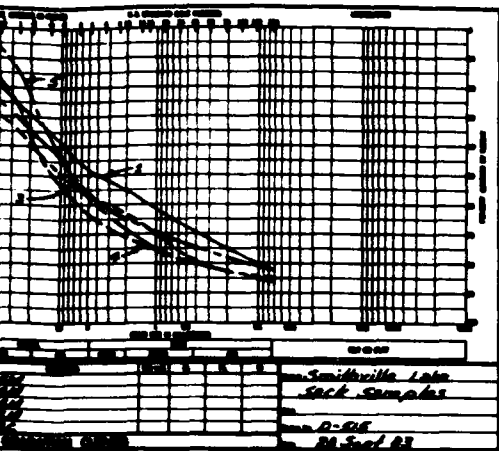


DETAIL OF TERMINATION OF COLLECTOR SYSTEM SHOWING 6" TO 10" PIPE CONNECTION

N.T.S.

Symbol	Definition	Notes	Amount
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	A.B.H.	LITTLE PLATE RIVER, MISSOURI BENTONVILLE LAKE EMBANKMENT CRITERIA REPORT	
Drawn by:	V.A.B.	DETAILS OF COLLECTOR DITCH	
Checked by:	A.B.H.	Scale:	AS SHOWN
Submitted by:	E.C.R.	MARCH 1967	
		RP-3-1768	





Grade	Location	Date	Amount

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
CHICAGO DIST., CHICAGO

DESIGNED BY: J.M.B.
CHECKED BY: P.B.T.
CALCULATED BY: J.B.H.
SUBMITTED BY: P.C.W.

LITTLE PLATTE RIVER, CHICAGO DISTRICT
BETHLEHEM LAKE
EMBANKMENT CRITERIA REPORT

GRADATION CURVES
LEFT ABUTMENT BORINGS

DATE: 28 SEPT 63
MARCH 1967

RP-3-1769

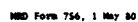


Figure 3



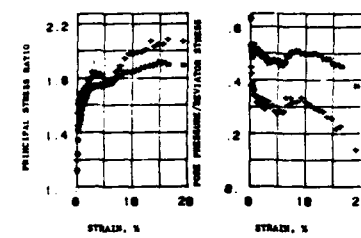
Figure 12

D

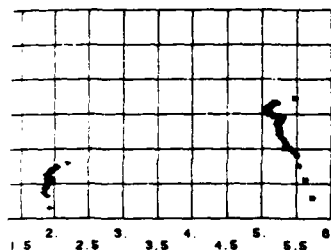
C

B

A



VE STRESS VECTOR CURVES ON SHEAR PLANE

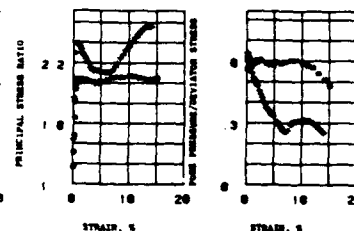


NORMAL STRESS, TSF

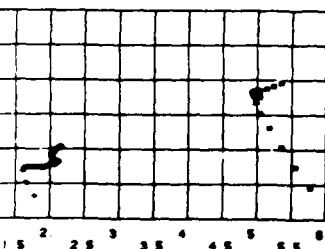
PROJECT: SHOTWELL
 BORING NO: UC 527
 DEPTH/RELEV: 04/205
 HSB LAB NO: 04/205
 DATE: 22 JUL 1984

TRIAXIAL COMPRESSION TEST REPORT

FIGURE 4



VE STRESS VECTOR CURVES ON SHEAR PLANE



NORMAL STRESS, TSF

PROJECT: SHOTWELL
 BORING NO: UC 527
 DEPTH/RELEV: 04/205
 HSB LAB NO: 04/205
 DATE: 22 JUL 1984

TRIAXIAL COMPRESSION TEST REPORT

FIGURE 5

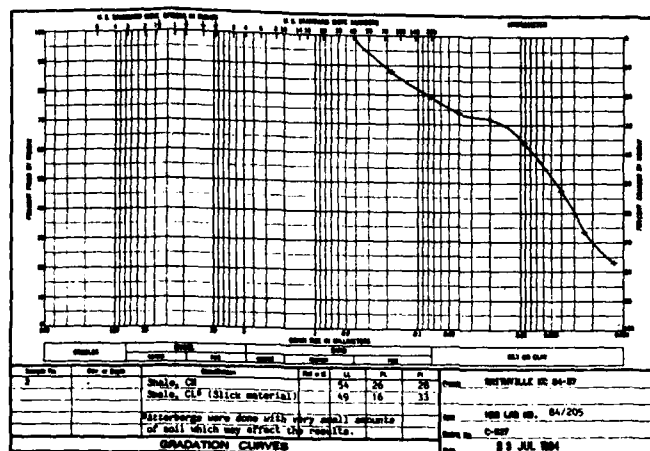
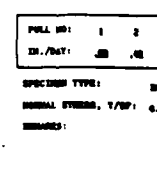
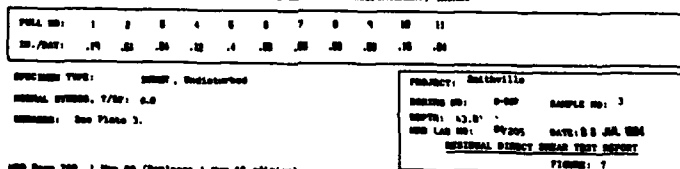
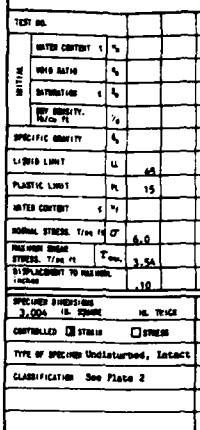
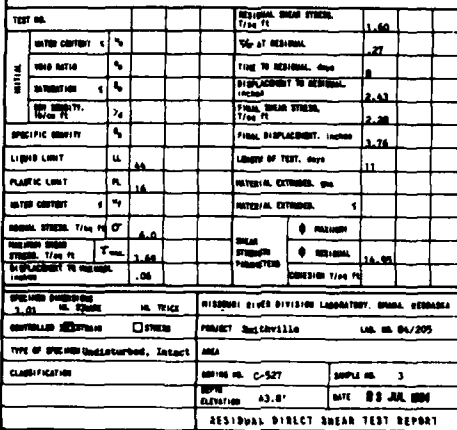
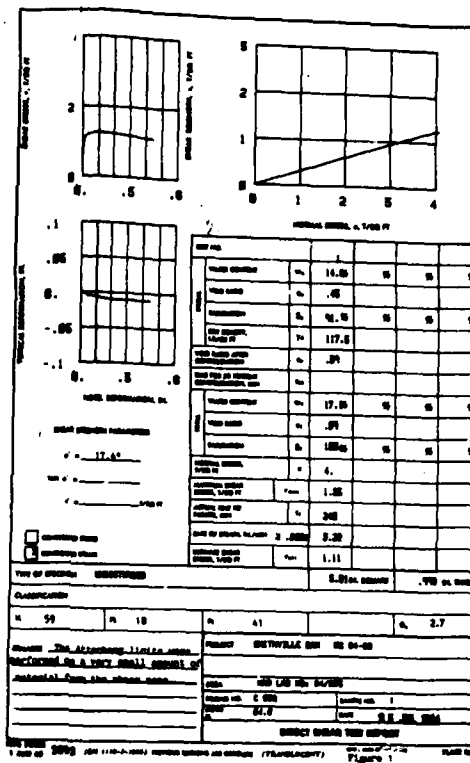
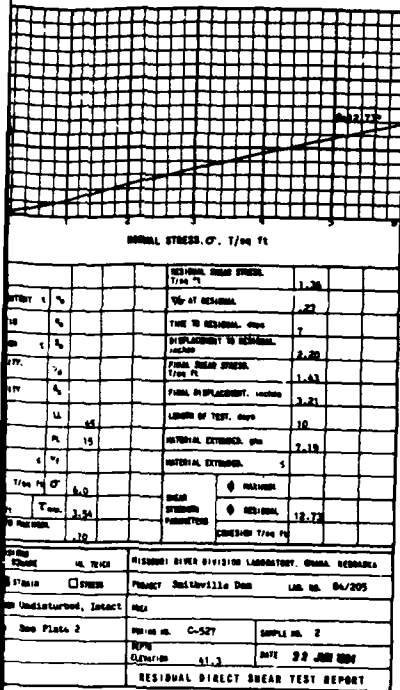


FIGURE 5

Revisions			
Symbol	Descriptions	Date	Approved
U. S. ARMY ENGINEER DISTRICT CCRPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	J.D.N.	LITTLE PLATTE RIVER, MISSOURI SHOTWELL LAKE	
Drawn by	J.M.P.	EMBANKMENT CRITERIA REPORT TEST DATA	
Checked by	J.D.N.	LEFT ABUTMENT, R TESTS BORINGS UC-527 (P-108-1, C-533 (P-12-2)	
Submitted by	F.C.R.	Scale	AS SHOWN
		MARCH 1987	
		RP-3-1770	

SUPPLEMENT NO. 1 PLATE NO. 20





RESIDUAL STRESS/NORMAL STRESS
VS.
ACCUMULATED RESIDUAL DISPLACEMENT, INCHES

30

25

20

15

10

5

0

0

0

0

0

0

0

0

0

0

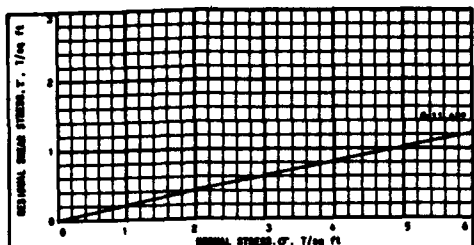
0

0

0

0

Revisions			
Symbol	Descriptions	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	J. D. H.	LITTLE PLATTE RIVER, MISSOURI BENTHVILLE LAKE	
Drawn by	J. W.	EMBANKMENT CRITERIA REPORT	
Checked by	J. B. H.	TEST DATA-LEFT ABUTMENT	
Submitted by	P. C. W.	RESIDUAL S TEST, BORING C-632 (P-80-10)	
Scale		AS SHOWN	
Date		MARCH 1987	
File No.		RP-3-1771	

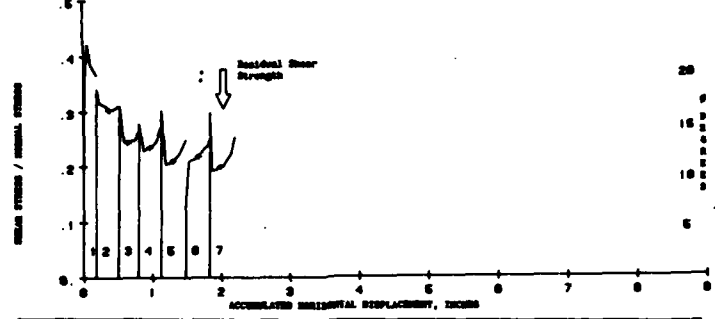


TEST NO.		SHEAR STRESS, T, T/100 PL		1.21	
MATERIAL	WATER CONTENT, %	%		20	
	WATER RATIO	%		7	
	LIQUID LIMIT	%		1.99	
	PLASTIC LIMIT	%		1.53	
SPECIFIC GRAVITY	G _s	2.50		2.50	
	LIQUID LIMIT	7		7	
PLASTIC LIMIT	PL	5		5	
	WATER CONTENT	5		5	
NORMAL STRESS, T/100 PL	T ₁	5.0		5.0	
	T ₂	2.54		2.54	
DISPLACEMENT TO RESIDUAL, INCHES	INCHES	.05		.05	
	DISPLACEMENT TO RESIDUAL, INCHES	.05		.05	
PROJECT: SHERIDAN DIVISION LABORATORY, SHERIDAN, MONTANA		PROJECT: SHERIDAN DIVISION LABORATORY, SHERIDAN, MONTANA		PROJECT: SHERIDAN DIVISION LABORATORY, SHERIDAN, MONTANA	
CONTROLLED: ☒ OTHER: ☐		CONTROLLED: ☒ OTHER: ☐		CONTROLLED: ☒ OTHER: ☐	
TYPE OF SPECIMEN: Undisturbed, Precast		TYPE OF SPECIMEN: Undisturbed, Precast		TYPE OF SPECIMEN: Undisturbed, Precast	
CLASSIFICATION		CLASSIFICATION		CLASSIFICATION	
SHEAR STRESS, T/100 PL		SHEAR STRESS, T/100 PL		SHEAR STRESS, T/100 PL	
DISPLACEMENT TO RESIDUAL, INCHES		DISPLACEMENT TO RESIDUAL, INCHES		DISPLACEMENT TO RESIDUAL, INCHES	
RESIDUAL DIRECT SHEAR TEST REPORT		RESIDUAL DIRECT SHEAR TEST REPORT		RESIDUAL DIRECT SHEAR TEST REPORT	

100 Form 700, 1 May 60 (Replaces 1 Nov 48 edition)

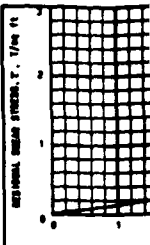
Figure 1

RESIDUAL STRESS/RESIDUAL STRESS
vs.
ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



PULL NO:	1	2	3	4	5	6	7
IN./DAY:	.10	.15	.14	.10	.10	.15	.10
SPECIMEN TYPE: UNDERSIZED, PRECAST							
NORMAL STRESS, T/100 PL: 5.0							
SHEAR STRESS: 1.21							
PROJECT: Sheridanville							
SHEAR STRESS, T/100 PL: 1.21							
DISPLACEMENT TO RESIDUAL, INCHES: 0.05							
DATE: 03 JUL 1960							
RESIDUAL DIRECT SHEAR TEST REPORT							
FIGURE: 2							

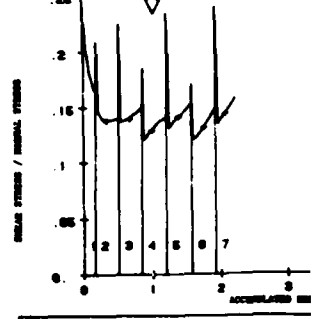
100 Form 700, 1 May 60 (Replaces 1 Nov 48 edition)



TEST NO.		SHEAR STRESS, T, T/100 PL		1.21	
MATERIAL	WATER CONTENT, %	%		20	
	WATER RATIO	%		7	
	LIQUID LIMIT	%		1.99	
	PLASTIC LIMIT	%		1.53	
SPECIFIC GRAVITY	G _s	2.50		2.50	
	LIQUID LIMIT	7		7	
PLASTIC LIMIT	PL	5		5	
	WATER CONTENT	5		5	
NORMAL STRESS, T/100 PL	T ₁	5.0		5.0	
	T ₂	2.54		2.54	
DISPLACEMENT TO RESIDUAL, INCHES	INCHES	.05		.05	
	DISPLACEMENT TO RESIDUAL, INCHES	.05		.05	
PROJECT: SHERIDAN DIVISION LABORATORY, SHERIDAN, MONTANA		PROJECT: SHERIDAN DIVISION LABORATORY, SHERIDAN, MONTANA		PROJECT: SHERIDAN DIVISION LABORATORY, SHERIDAN, MONTANA	
CONTROLLED: ☒ OTHER: ☐		CONTROLLED: ☒ OTHER: ☐		CONTROLLED: ☒ OTHER: ☐	
TYPE OF SPECIMEN: Undisturbed, Precast		TYPE OF SPECIMEN: Undisturbed, Precast		TYPE OF SPECIMEN: Undisturbed, Precast	
CLASSIFICATION		CLASSIFICATION		CLASSIFICATION	
SHEAR STRESS, T/100 PL		SHEAR STRESS, T/100 PL		SHEAR STRESS, T/100 PL	
DISPLACEMENT TO RESIDUAL, INCHES		DISPLACEMENT TO RESIDUAL, INCHES		DISPLACEMENT TO RESIDUAL, INCHES	
RESIDUAL DIRECT SHEAR TEST REPORT		RESIDUAL DIRECT SHEAR TEST REPORT		RESIDUAL DIRECT SHEAR TEST REPORT	

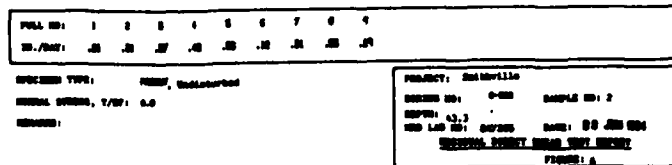
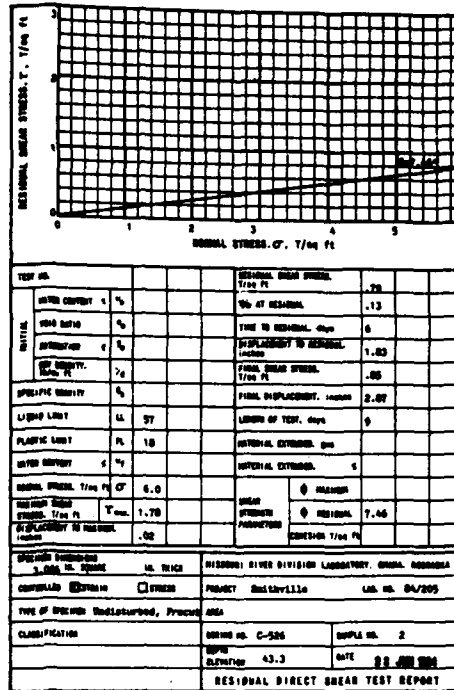
100 Form 700, 1 May 60 (Replaces 1 Nov 48 edition)

RESIDUAL STRESS/RESIDUAL STRESS
vs.
ACCUMULATED HORIZONTAL DISPLACEMENT, INCHES



PULL NO:	1	2	3	4	5	6	7
IN./DAY:	.10	.15	.14	.10	.10	.15	.10
SPECIMEN TYPE: UNDERSIZED, PRECAST							
NORMAL STRESS, T/100 PL: 5.0							
SHEAR STRESS: 1.21							
PROJECT: Sheridanville							
SHEAR STRESS, T/100 PL: 1.21							
DISPLACEMENT TO RESIDUAL, INCHES: 0.05							
DATE: 03 JUL 1960							
RESIDUAL DIRECT SHEAR TEST REPORT							
FIGURE: 2							

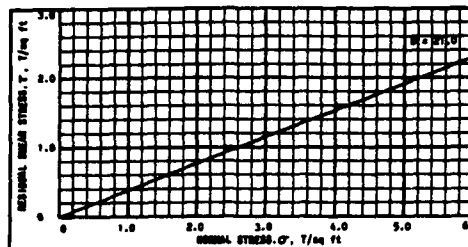
100 Form 700, 1 May 60 (Replaces 1 Nov 48 edition)



Revisions			
Symbol	Description	Date	Approved

**U S ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI**

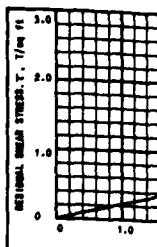
Designed by A.B.N.	 ENGINEERING CRITERIA REPORT TEST DATA LEFT ABUTMENT, RESIDUAL S TESTS BORING C-526 (P-110-6)	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE
Drawn by A.W.P.		
Checked by A.B.N.		
Submitted by P.C.W.		
Scale AS SHOWN		
MARCH 1967		
		NO. 3 1972



TEST NO.				RESIDUAL SHEAR STRESS, T/m ft	2.303
WATER CONTENT, %	-			σ _u AT RESIDUAL, lb/in	0.307
VOID RATIO	-			TIME TO RESIDUAL, days	0
EXTRUSION, %	-			DISPLACEMENT TO RESIDUAL, inches	2.900
WATER CONTENT, %	-			FINAL SHEAR STRESS, T/m ft	2.71
SPECIFIC GRAVITY	-			FINAL DISPLACEMENT, inches	3.120
LIQUID LIMIT	AL	31.0		AGE OF TEST, days	0
PLASTIC LIMIT	PL	19.0		RESIDUAL EXTENSION, gms	-
WATER CONTENT, %	-			RESIDUAL EXTENSION, %	-
RESIDUAL STRESS, T/m ft	σ'	6.0		RESIDUAL STRESS, T/m ft	28.6
RESIDUAL STRESS, T/m ft	T _{res}	3.637		RESIDUAL STRESS, T/m ft	21.2
DISPLACEMENT TO RESIDUAL, inches		0.114		RESIDUAL STRESS, T/m ft	-

SPECIMEN DESCRIPTION	3.0	AL STONE	-	AL STONE	MISSOURI RIVER DIVISION LABORATORY, DODGE, MISSOURI
CONDITION	☒ STONE	☐ FRESH		PROJECT	St. Louisville
TYPE OF SPECIMEN	Undisturbed, Intact			LAB. NO.	60/74
CLASSIFICATION	From very small			TEST NO.	505
Soil sample from residual surface				DATE	17 MAY 1964
Full 4 has partial data.					

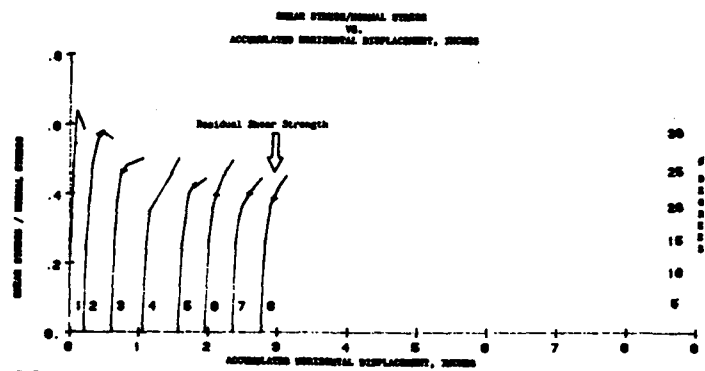
Figure 7



TEST NO.				RESIDUAL SHEAR STRESS, T/m ft	2.303
WATER CONTENT, %	-			σ _u AT RESIDUAL, lb/in	0.307
VOID RATIO	-			TIME TO RESIDUAL, days	0
EXTRUSION, %	-			DISPLACEMENT TO RESIDUAL, inches	2.900
WATER CONTENT, %	-			FINAL SHEAR STRESS, T/m ft	2.71
SPECIFIC GRAVITY	-			FINAL DISPLACEMENT, inches	3.120
LIQUID LIMIT	AL	31.0		AGE OF TEST, days	0
PLASTIC LIMIT	PL	19.0		RESIDUAL EXTENSION, gms	-
WATER CONTENT, %	-			RESIDUAL EXTENSION, %	-
RESIDUAL STRESS, T/m ft	σ'	6.0		RESIDUAL STRESS, T/m ft	28.6
RESIDUAL STRESS, T/m ft	T _{res}	3.637		RESIDUAL STRESS, T/m ft	21.2
DISPLACEMENT TO RESIDUAL, inches		0.114		RESIDUAL STRESS, T/m ft	-

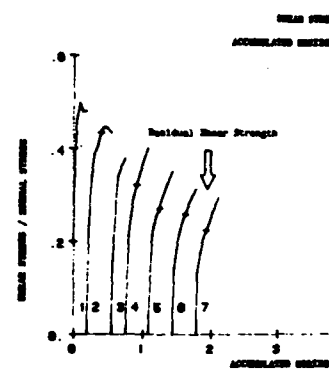
SPECIMEN DESCRIPTION	3.0	AL STONE	-	AL STONE	MISSOURI RIVER DIVISION LABORATORY, DODGE, MISSOURI
CONDITION	☒ STONE	☐ FRESH		PROJECT	St. Louisville
TYPE OF SPECIMEN	Undisturbed, Intact			LAB. NO.	60/74
CLASSIFICATION	From very small			TEST NO.	505
Soil sample from residual surface				DATE	17 MAY 1964
Full 4 has partial data.					

Figure 7



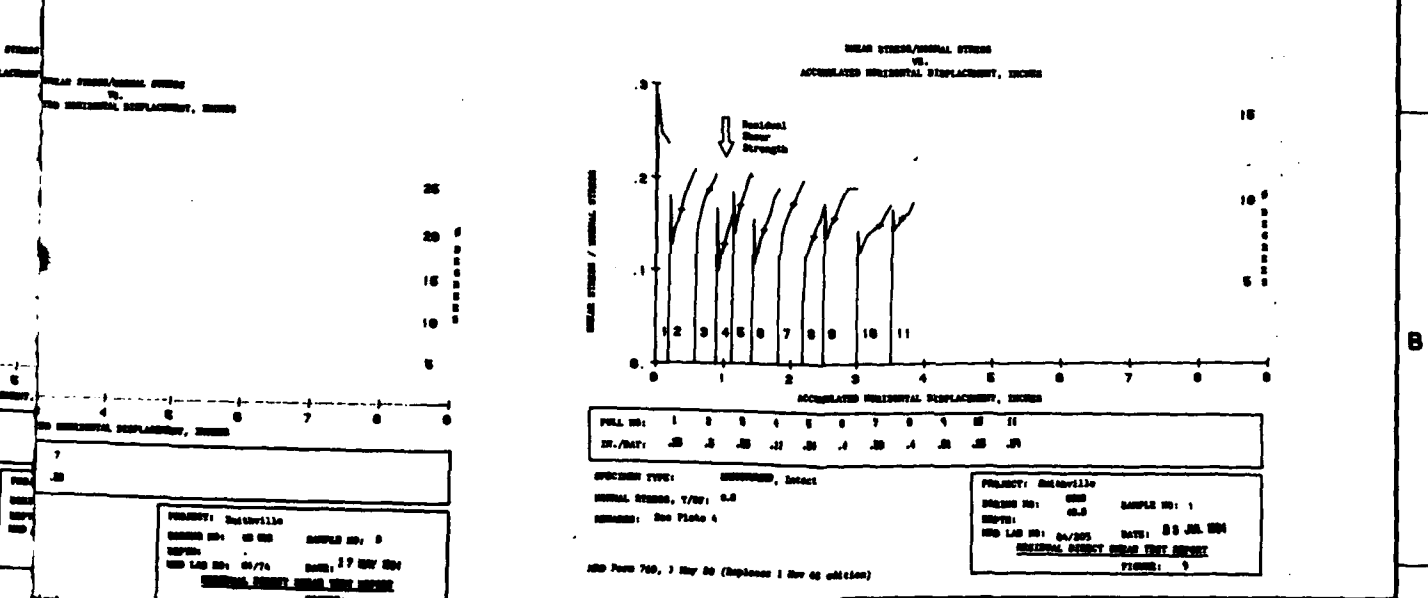
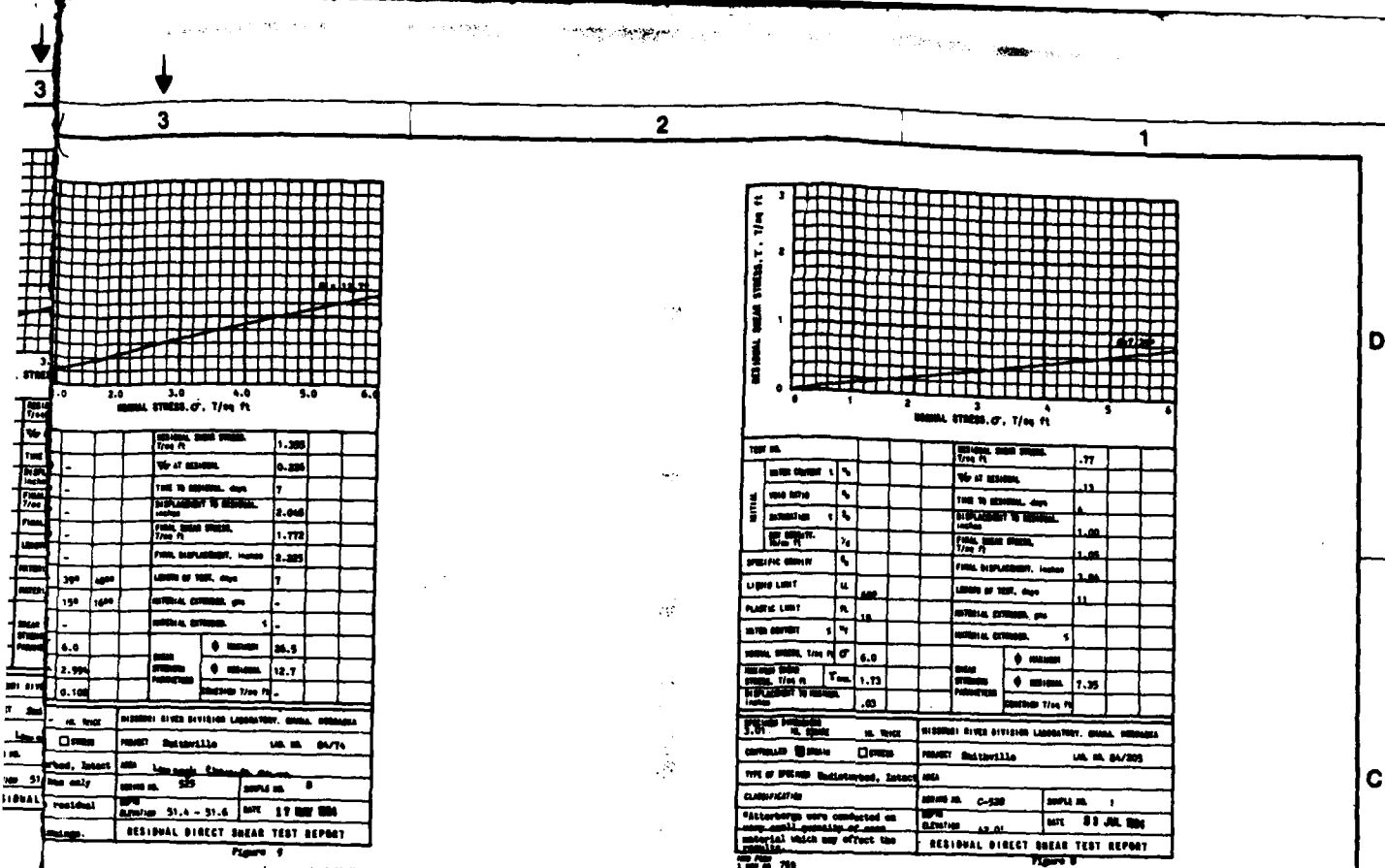
FULL NO:	1	2	3	4	5	6	7	8
DR./DAY:	.5	.5	.5	.5	.5	.5	.5	.5
SPECIMEN TYPE:	DRIFT							
NORMAL STRESS, T/m ft	0.0							
REMARKS:	See Plate - 3							
PROJECT:	St. Louisville							
TESTING NO:	NO 60							
SAMPLE NO:	2							
DATE:	17 MAY 1964							
TESTING STATION:	MISSOURI RIVER DIVISION LABORATORY, DODGE, MISSOURI							

MS Form 700, 1 May 60 (Replaces 1 Nov 58 edition)



FULL NO:	1	2	3	4	5	6	7
DR./DAY:	.5	.5	.5	.5	.5	.5	.5
SPECIMEN TYPE:	DRIFT						
NORMAL STRESS, T/m ft	0.0						
REMARKS:							
PROJECT:	St. Louisville						
TESTING NO:	NO 60						
SAMPLE NO:	2						
DATE:	17 MAY 1964						
TESTING STATION:	MISSOURI RIVER DIVISION LABORATORY, DODGE, MISSOURI						

MS Form 700, 1 May 60 (Replaces 1 Nov 58 edition)



U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

LITTLE PLATE RIVER, MISSOURI
SEATTLE LAKE
EMBANKMENT CRITERIA REPORT
TEST DATA
LEFT ABUTMENT, RESIDUAL S TEST
FORMS UC-888 (P-108-2, C-888 (P-108-2))

Designed by: J.B.M.
 Drawn by: J.B.M.
 Checked by: J.B.M.
 Submitted by: F.C.E.

DATE: 17 MAY 1964
 DATE: 17 MAY 1964
 DATE: 17 MAY 1964
 DATE: 17 MAY 1964

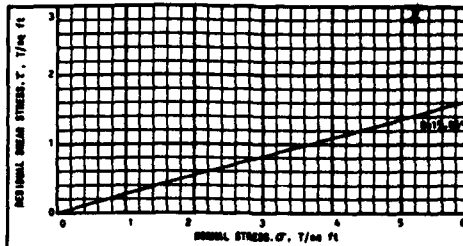
REVISIONS

Symbol	Description	Date	Approved

MARCH 1967

RP-5-1773

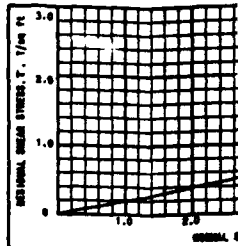
SUPPLEMENT NO 1 PLATE NO 25



TEST NO.				ORIGINAL SHEAR STRESS, T/100 FT	1.62
WATER CONTENT, %	14			% AT RESIDUAL	.87
VOID RATIO	1.4			TIME TO RESIDUAL, days	0
EXTENSION, %	14			DISPLACEMENT TO RESIDUAL, inches	3.15
WATER CONTENT, %	14			FINAL SHEAR STRESS, T/100 FT	1.17
SPECIFIC GRAVITY	2.6			FINAL DISPLACEMENT, inches	3.90
LIQUID LIMIT	44			LIMIT OF TEST, days	10
PLASTIC LIMIT	16			RESIDUAL EXTENSION, %	-
WATER CONTENT, %	14			RESIDUAL EXTENSION, %	-
WATER STRESS, T/100 FT	6.0			WATER STRESS, T/100 FT	6.0
WATER STRESS, T/100 FT	3.36			WATER STRESS, T/100 FT	3.36
DISPLACEMENT TO RESIDUAL, inches	.12			DISPLACEMENT TO RESIDUAL, inches	.12

SPECIES	CLAY	IN. TEST	RESIDUAL DIRECT SHEAR DIVISION LABORATORY, BUREAU OF MINES
CONTROLS	☒ SHEAR	☐ STRESS	PROJECT
TYPE OF SPECIMEN	Undisturbed, Intact		LAB. NO. 84/205
CLASSIFICATION	There was not enough material to determine the	TEST NO. 84/205	SAMPLE NO. 1, and's name
Atterberg Limits	See Plate 1.	TEST NO. 84/205	DATE 22 JUN 66

Figure 1



TEST NO.				ORIGINAL SHEAR STRESS, T/100 FT	1.62
WATER CONTENT, %	14			% AT RESIDUAL	.87
VOID RATIO	1.4			TIME TO RESIDUAL, days	0
EXTENSION, %	14			DISPLACEMENT TO RESIDUAL, inches	3.15
WATER CONTENT, %	14			FINAL SHEAR STRESS, T/100 FT	1.17
SPECIFIC GRAVITY	2.6			FINAL DISPLACEMENT, inches	3.90
LIQUID LIMIT	44			LIMIT OF TEST, days	10
PLASTIC LIMIT	16			RESIDUAL EXTENSION, %	-
WATER CONTENT, %	14			RESIDUAL EXTENSION, %	-
WATER STRESS, T/100 FT	6.0			WATER STRESS, T/100 FT	6.0
WATER STRESS, T/100 FT	3.36			WATER STRESS, T/100 FT	3.36
DISPLACEMENT TO RESIDUAL, inches	.12			DISPLACEMENT TO RESIDUAL, inches	.12

SPECIES	CLAY	IN. TEST	RESIDUAL DIRECT SHEAR DIVISION LABORATORY, BUREAU OF MINES
CONTROLS	☒ SHEAR	☐ STRESS	PROJECT
TYPE OF SPECIMEN	Undisturbed, Intact		LAB. NO. 84/205
CLASSIFICATION	Specimen was observed on existing concave shaped soil. Considerable material was extruded during the test. From only small amount of soil from	TEST NO. 84/205	SAMPLE NO. 1, and's name
Atterberg Limits	See Plate 1.	TEST NO. 84/205	DATE 22 JUN 66

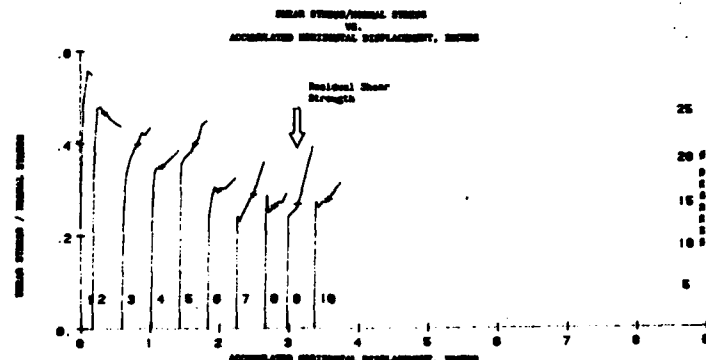
Figure 1

D

C

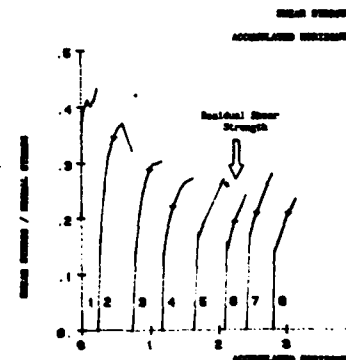
B

A



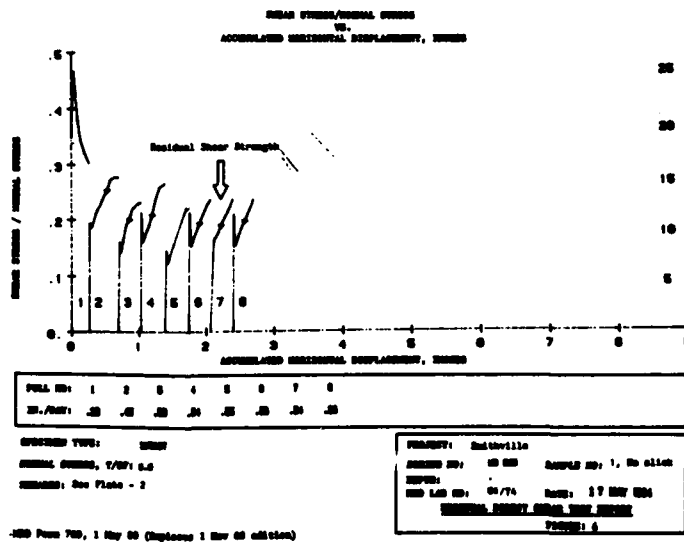
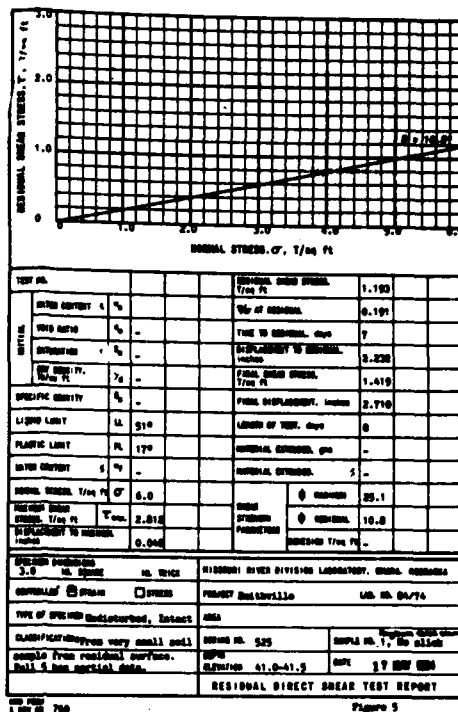
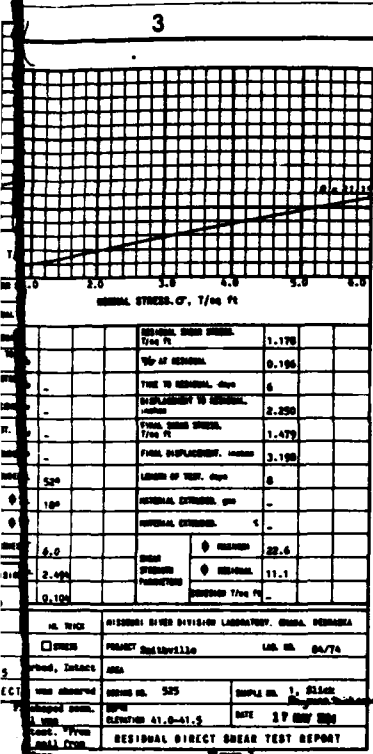
TEST NO.	1	2	3	4	5	6	7	8	9	10
WATER CONTENT, %	.28	.27	.24	.24	.24	.24	.24	.24	.24	.24
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0

AND Page 700, 1 May 60 (Replaces 1 May 60 edition)

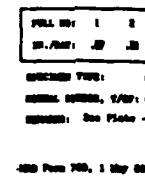
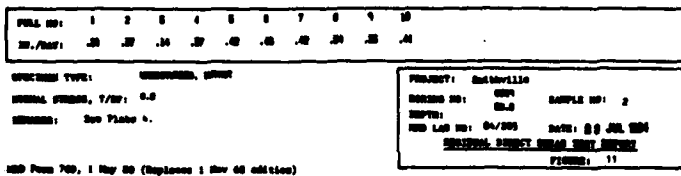
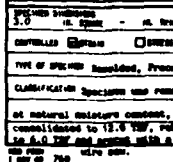
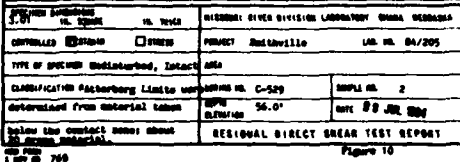


TEST NO.	1	2	3	4	5	6	7	8	9	10
WATER CONTENT, %	.28	.27	.24	.24	.24	.24	.24	.24	.24	.24
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
WATER STRESS, T/100 FT	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0

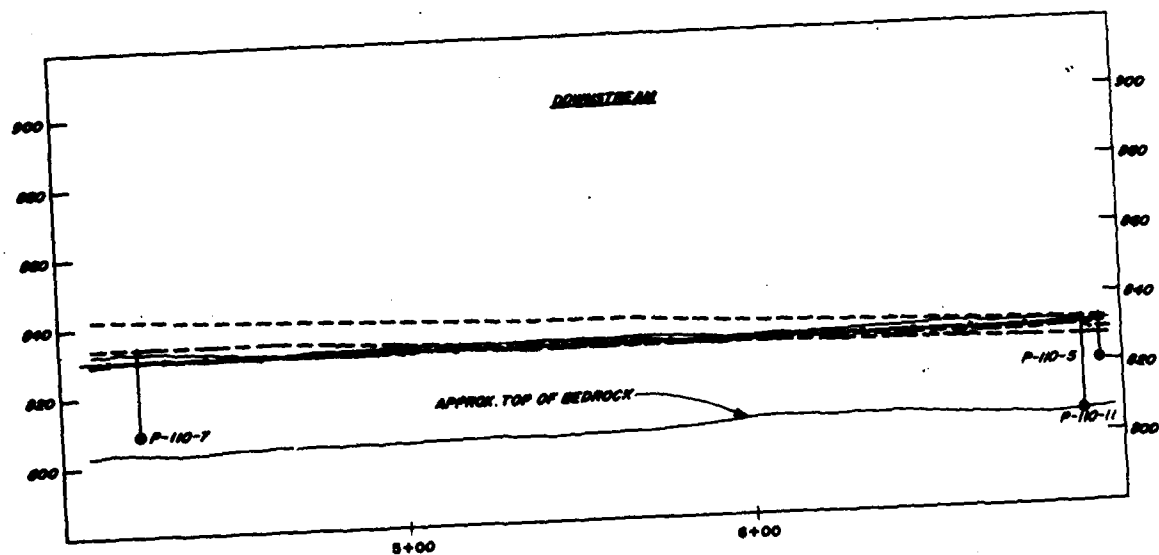
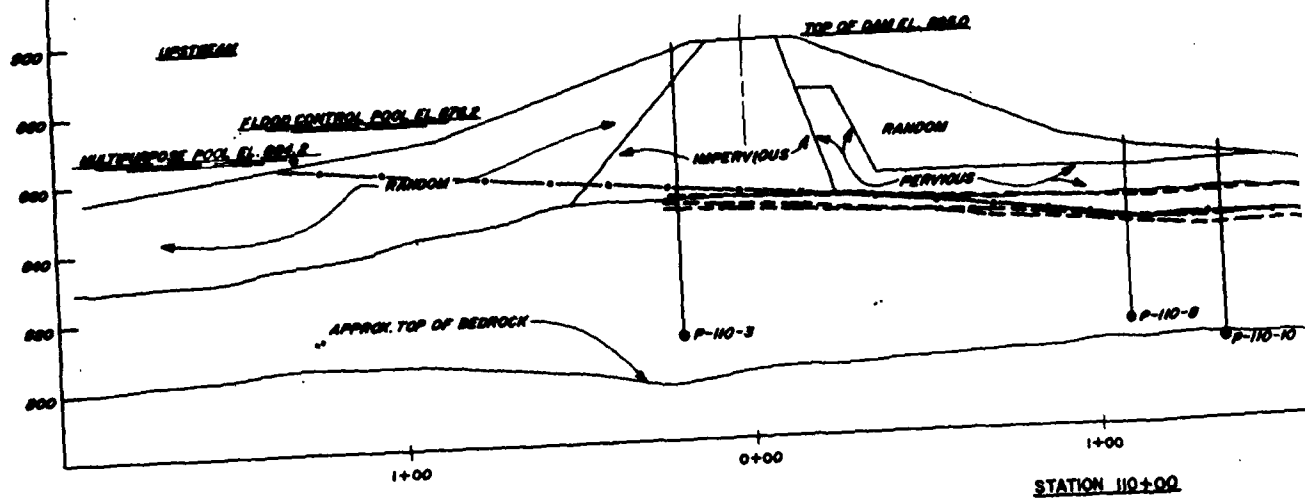
AND Page 700, 1 May 60 (Replaces 1 May 60 edition)

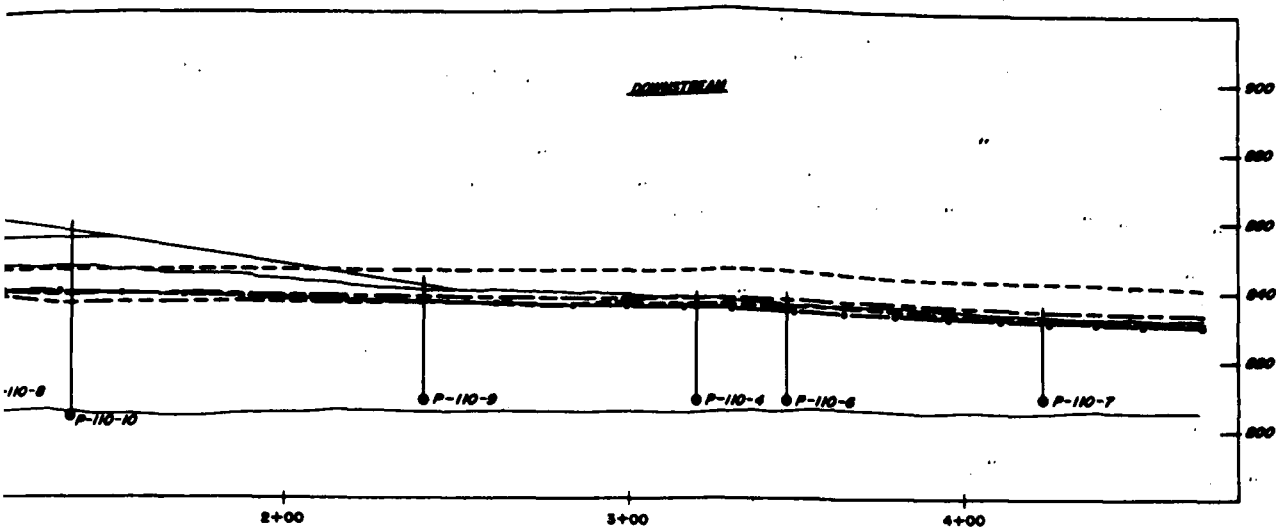


Revisions			
Symbol	Descriptions	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	J. B. H.	LITTLE PLATE OVER, MISSOURI EMBANKMENT CRITERIA REPORT TEST DATA LEFT ABUTMENT, RESIDUAL 8 TESTS BORING UC-885 (P-100-0)	
Drawn by	J. W. R.		
Checked by	J. B. H.		
Submitted by	F. C. W.		
Date		MARCH 1967	
File No.		RP-3-1774	



ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929





NOTES:

- 13 FEB. 1964 - POOL EL. 884.0 (BEFORE WELLS INSTALLED)
- 3 JUN 1966 - POOL EL. 880.0 (AFTER WELLS INSTALLED)
- 17 OCT. 1966 - POOL EL. 873.17 (RECORD POOL)
- PROJECTED SPILLWAY CREST POOL EL. 880.2

Revisions			
Number	Description	Date	Approved

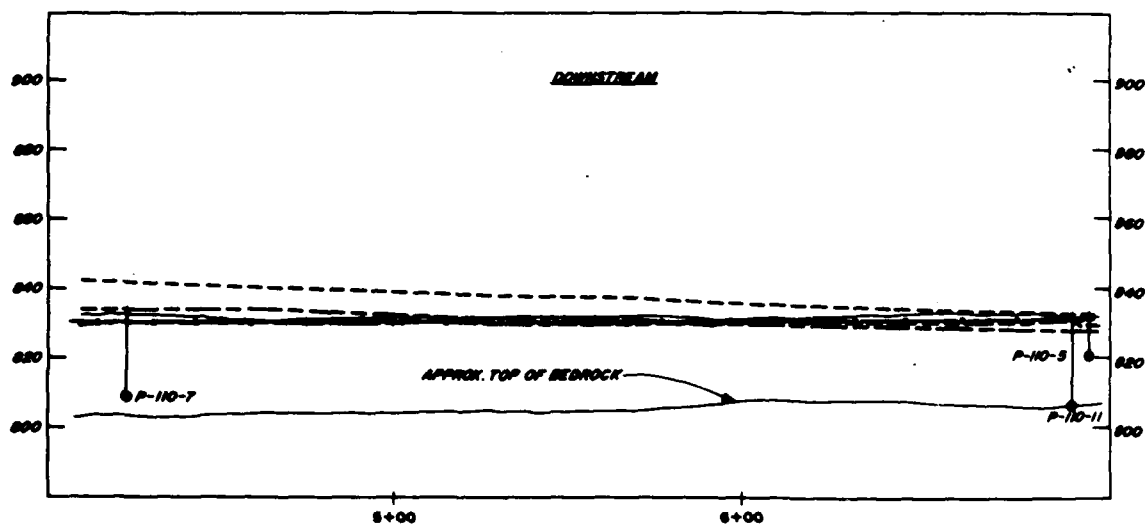
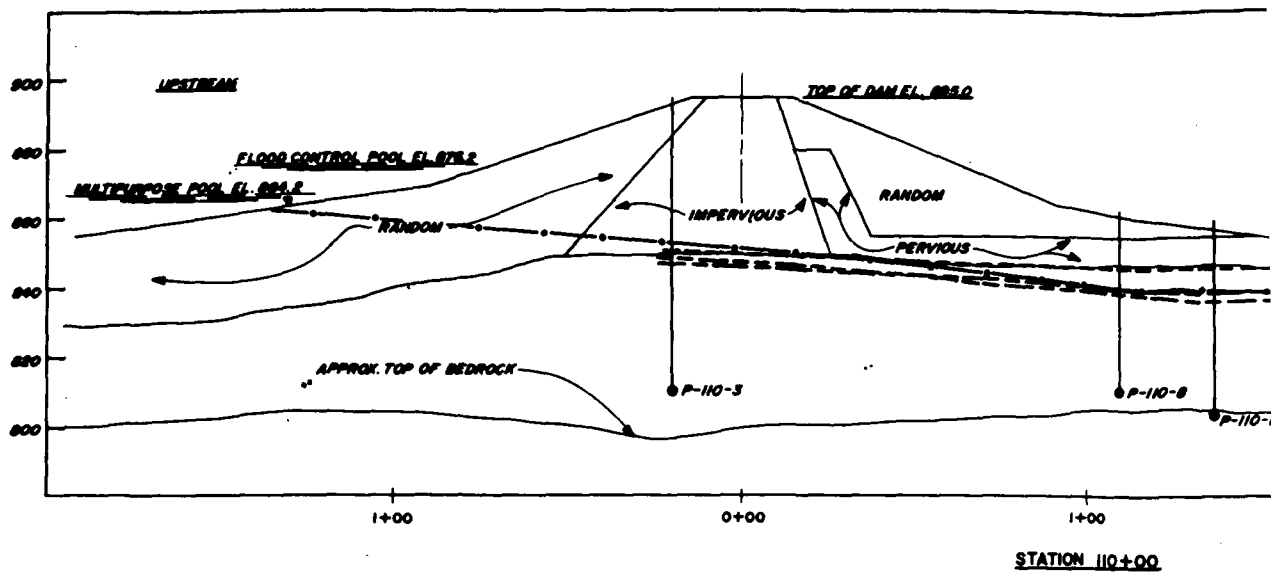
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

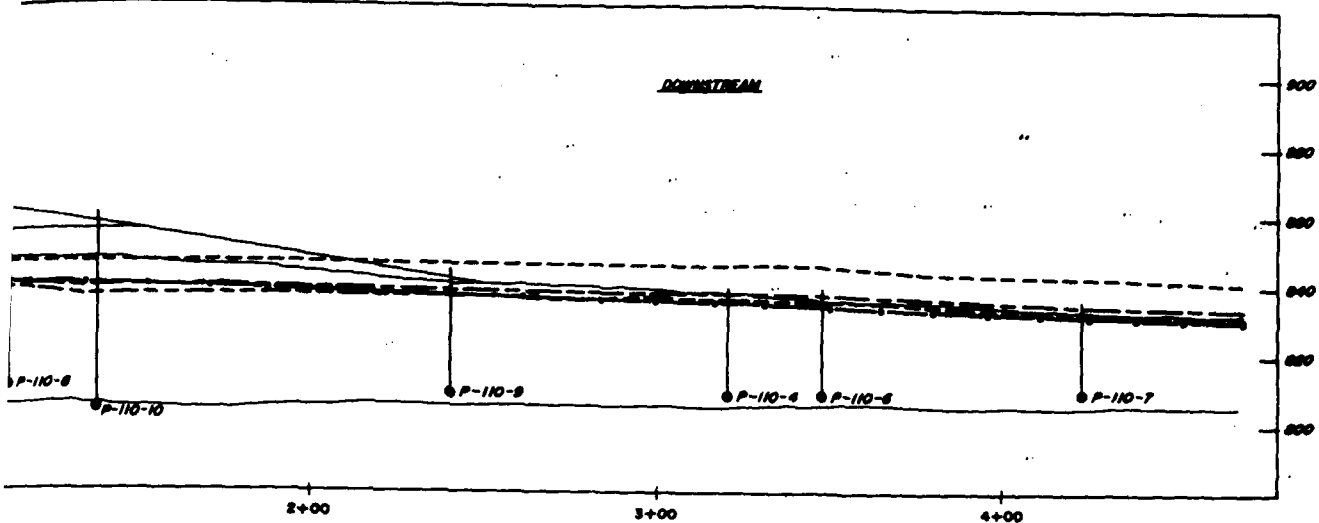
Designed by: J. B. H.	LITTLE PLATE RIVER, MISSOURI BETHVILLE LAKE EMBANKMENT CRITERIA REPORT PIEZOMETRIC LEVEL AT STA. 10+00 BEFORE & AFTER WELLS INSTALLED AT RECORD POOL, AND AT PROJECTED SPILLWAY CREST	Scale: AS SHOWN	
Drawn by: J. W. P.			
Checked by: J. B. H.			
Submitted by: P. C. W.			

MARCH 1987

RP-3-1776

ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929





- NOTES
- 15 FEB. 1984 - POOL EL. 884.0 (BEFORE WELLS INSTALLED)
 - 3 JUN 1985 - POOL EL. 886.0 (AFTER WELLS INSTALLED)
 - 17 OCT. 1985 - POOL EL. 873.17 (RECORD POOL)
 - PROJECTED SPILLWAY CREST POOL EL. 880.2

Revisions			
Number	Description	Date	Approved

**U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI**

Designed by: J. B. H.	LITTLE PLATTE RIVER, MISSOURI BENTLEY LAKE EMBAKMENT CRITERIA REPORT PIEZOMETRIC LEVEL AT STA. NO +00 BEFORE & AFTER WELLS INSTALLED AT RECORD POOL, AND AT PROJECTED SPILLWAY CREST	Scale: AS SHOWN	
Drawn by: J. W. P.			
Checked by: J. B. H.			
Submitted by: F. C. W.			

MARCH 1987

RP-3-1776

AD-A182 106

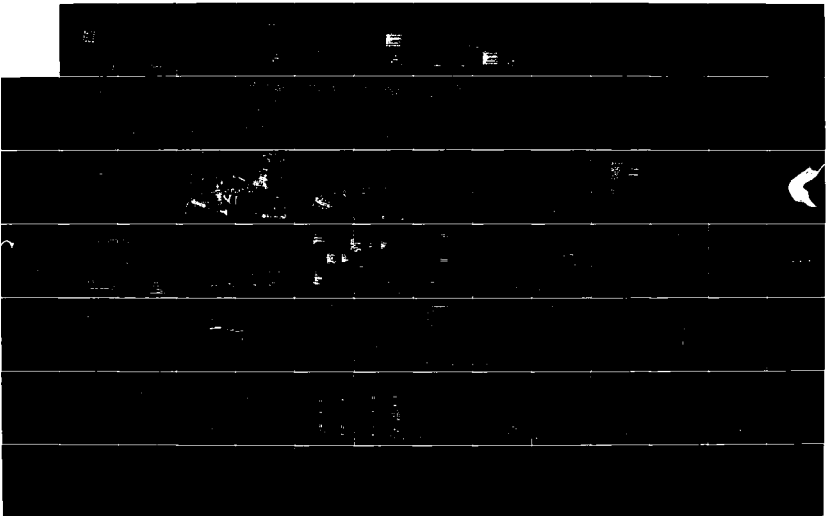
MULTIPLE-PURPOSE PROJECT: PLATTE RIVER BASIN LITTLE
PLATTE RIVER MISSOURI (U) CORPS OF ENGINEERS KANSAS
CITY MO KANSAS CITY DISTRICT F C WALBERG ET AL. MAR 87

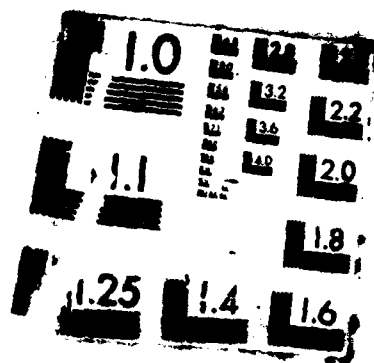
2/3

UNCLASSIFIED

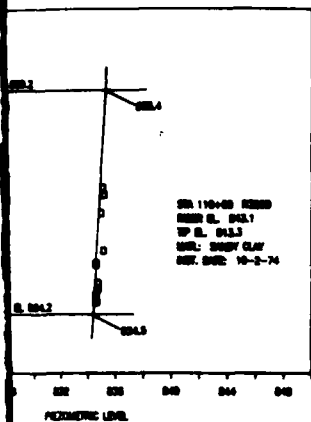
F/G 13/2

NL

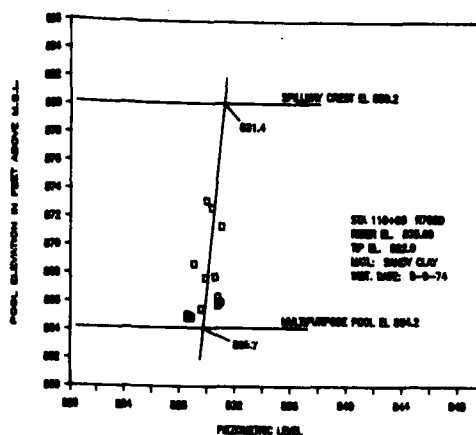




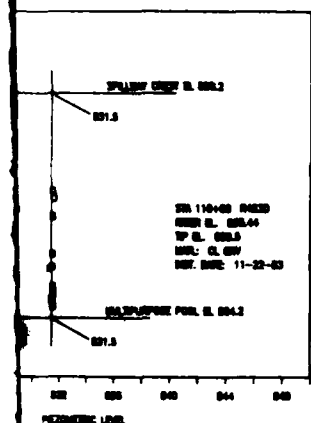
P-110-4



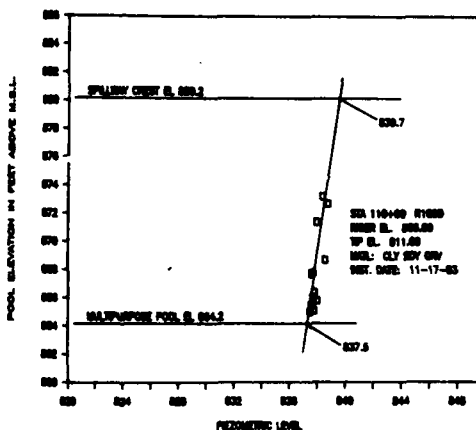
P-110-5



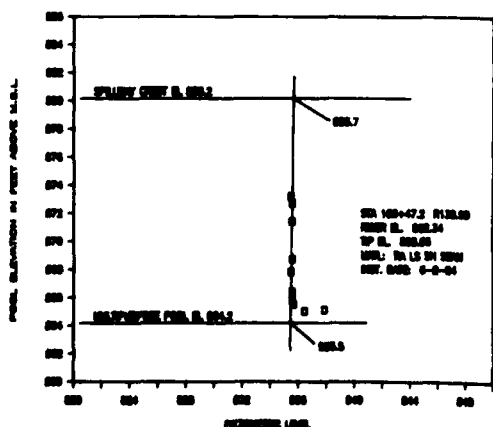
P-110-7



P-110-8



P-110-10



NOTE: ALL PIEZOMETER READINGS TAKEN AFTER INSTALLATION OF RELIEF WELLS.

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
 CORPS OF ENGINEERS
 KANSAS CITY, MISSOURI

Designed by: J.S.H.

Drawn by: J.W.P.

Checked by: J.S.H.

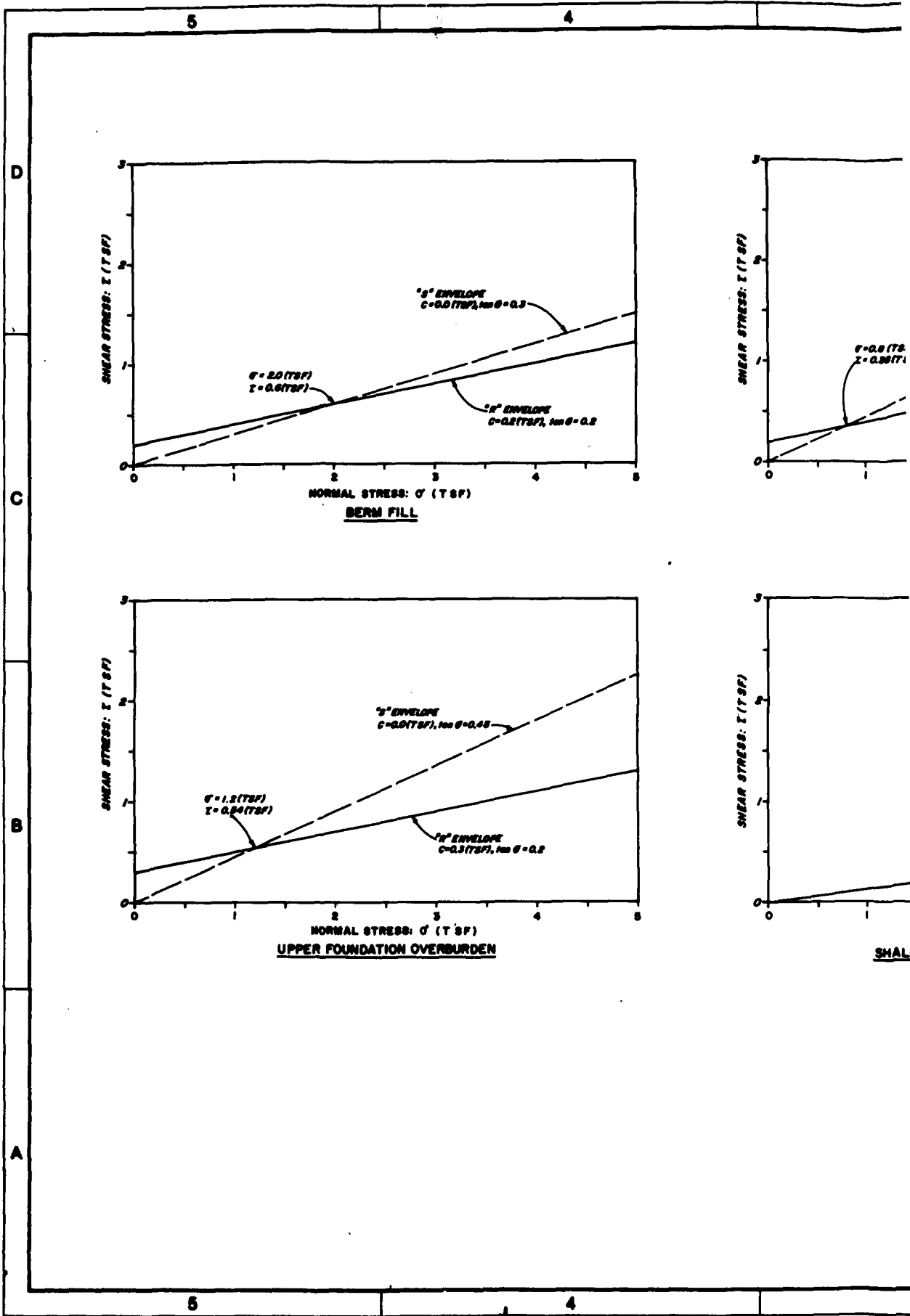
Submitted by: F.C.W.

LITTLE PLATTE RIVER, MISSOURI
 SMITHVILLE LAKE
EMBANKMENT CRITERIA REPORT
PROJECTED PIEZOMETRIC LEVELS FROM
PIEZOMETER DATA OBTAINED AFTER
INSTALLATION OF RELIEF WELLS-STATION 110+00

Scale: AS SHOWN

MARCH 1987

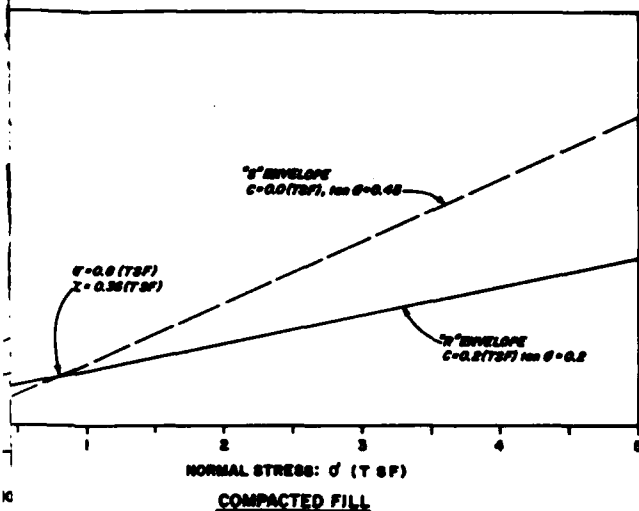
File No. **RP-3-1777**



3

2

1

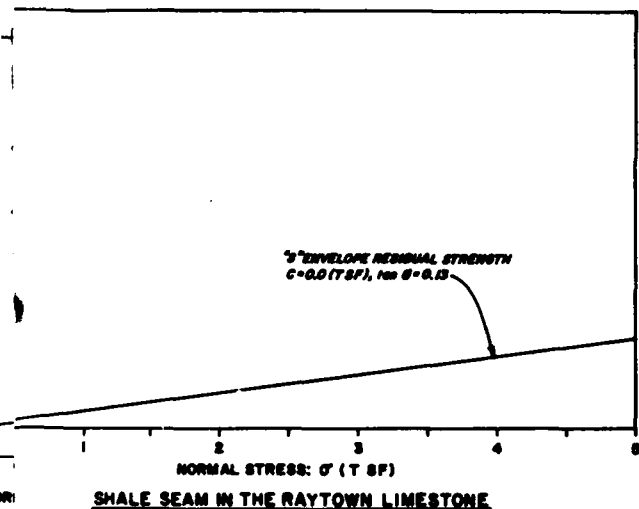


PHYSICAL SOIL CONSTANTS(1)						
MATERIAL	UNIT WEIGHT (LB/FT ³)		SHEAR STRENGTH PARAMETERS			
	MOIST	SATURATED	c		phi	
			(TSF)	(deg)	(TSF)	(deg)
BERM FILL	110	118	0.2	0.2	0.0	0.3
COMPACTED FILL	120	125	0.2	0.2	0.0	0.48
UPPER FOUNDATION OVERBURDEN	115	120	0.3	0.2	0.0	0.48

PHYSICAL SOIL CONSTANTS(2)			
MATERIAL	UNIT WEIGHT (SATURATED) (LB/FT ³)	SHEAR STRENGTH PARAMETERS	
		c(TSF)	tan phi
LOWER (COARSER) FOUNDATION OVERBURDEN	120	0	0.58
RAYTOWN LIMESTONE(3)	140	0	0
SHALE SEAM IN THE RAYTOWN LIMESTONE(4)	140	0	0.13

NOTES:

- (1) SOIL CONSTANTS REFERENCED IN DESIGN MEMORANDUM #10, "SOIL DATA AND EMBANKMENT DESIGN", APPENDIX B PLATE NO. 2.
- (2) SOIL CONSTANTS REFERENCED IN THE "LEFT ABUTMENT STABILITY REPORT", JULY 1984, (PLATE NO. A-27).
- (3) ZERO SHEAR STRENGTH IS ASSIGNED FOR THE RAYTOWN LIMESTONE MEMBER. THE THIN AND VERTICALLY JOINTED MEMBER IS ASSUMED TO BE INCAPABLE OF RESISTING SHEARING FORCES.
- (4) RESIDUAL STRENGTH IS LISTED FOR THE SHALE SEAM IN THE RAYTOWN LIMESTONE. (PEAK STRENGTHS ARE LISTED FOR THE REMAINDER OF MATERIALS).

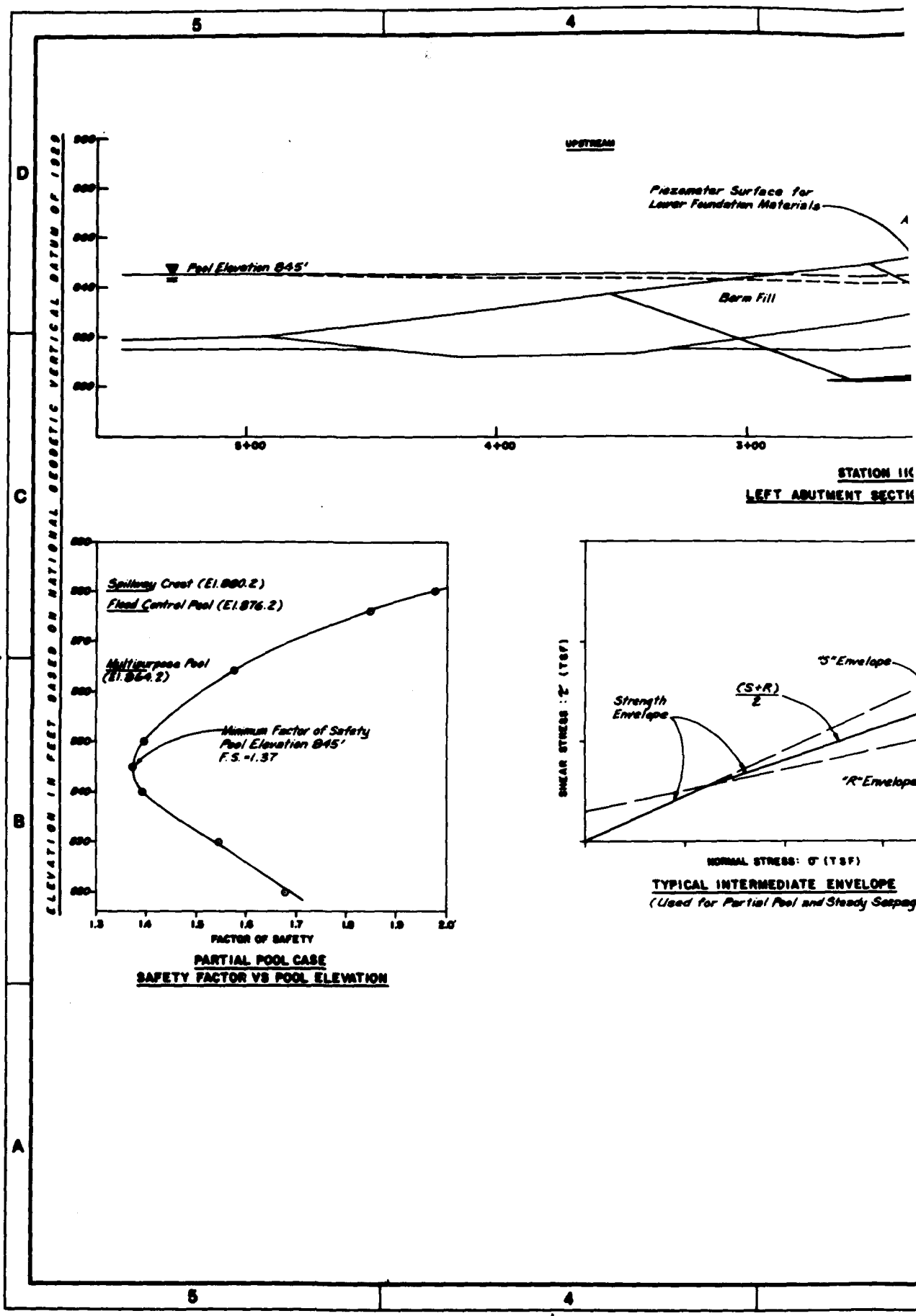


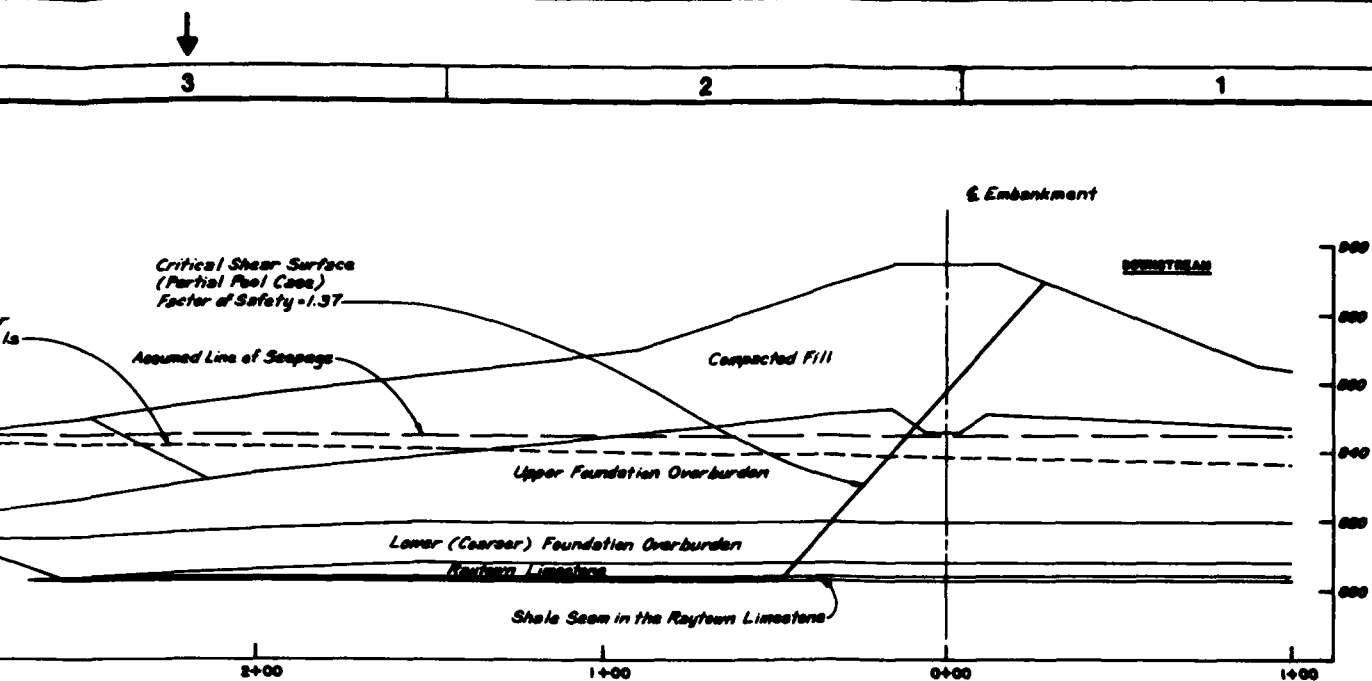
Revisions			
Symbol	Description	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	U.S.G.	LITTLE PLATE RIVER, MISSOURI BENTLEYVILLE LAKE EMBRANKMENT CRITERIA REPORT SHEAR STRENGTH PARAMETERS (LEFT ABUTMENT)	
Drawn by:	P.B.T.		
Checked by:	J.B.M.		
Submitted by:	P.C.W.		
Scale: AS SHOWN		MARCH 1987	
		RP-3-1778	

3

2

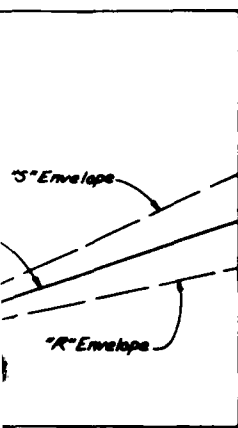
SUPPLEMENT NO 1 PLATE NO. 28





STATION 110+00

EMBANKMENT SECTION - PARTIAL POOL CASE



F (TSF)

ENVELOPE

and Steady Seepage Cases)

RESULTS OF THE PARTIAL POOL STABILITY ANALYSIS

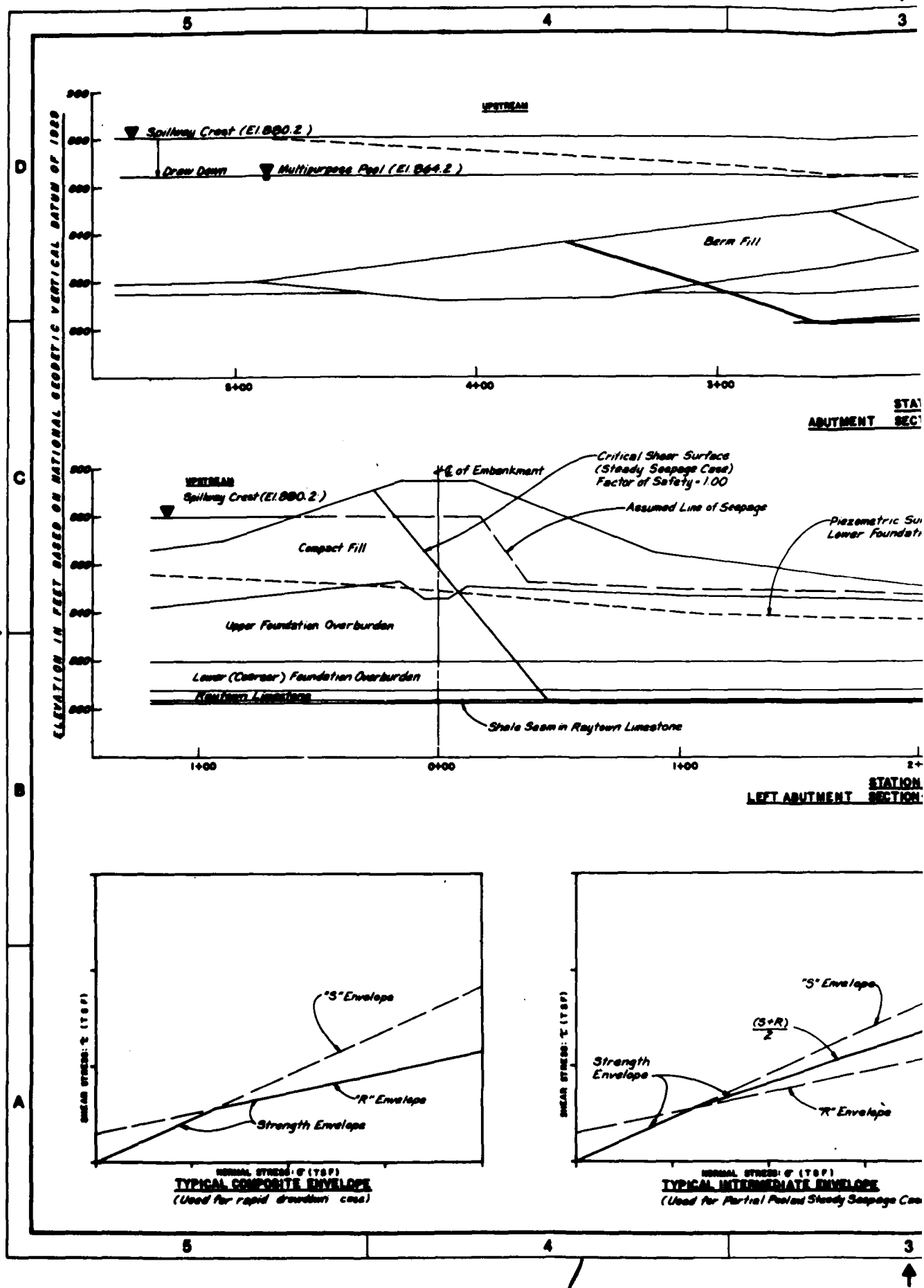
POOL ELEVATION	FACTOR OF SAFETY
NO POOL	1.00
830	1.56
840	1.30
845	1.37
850	1.40
(MULTIPURPOSE)	
854.2	1.37
(FLOOD CONTROL)	
876.2	1.56
(SPILLWAY CREST)	
880.2	1.37

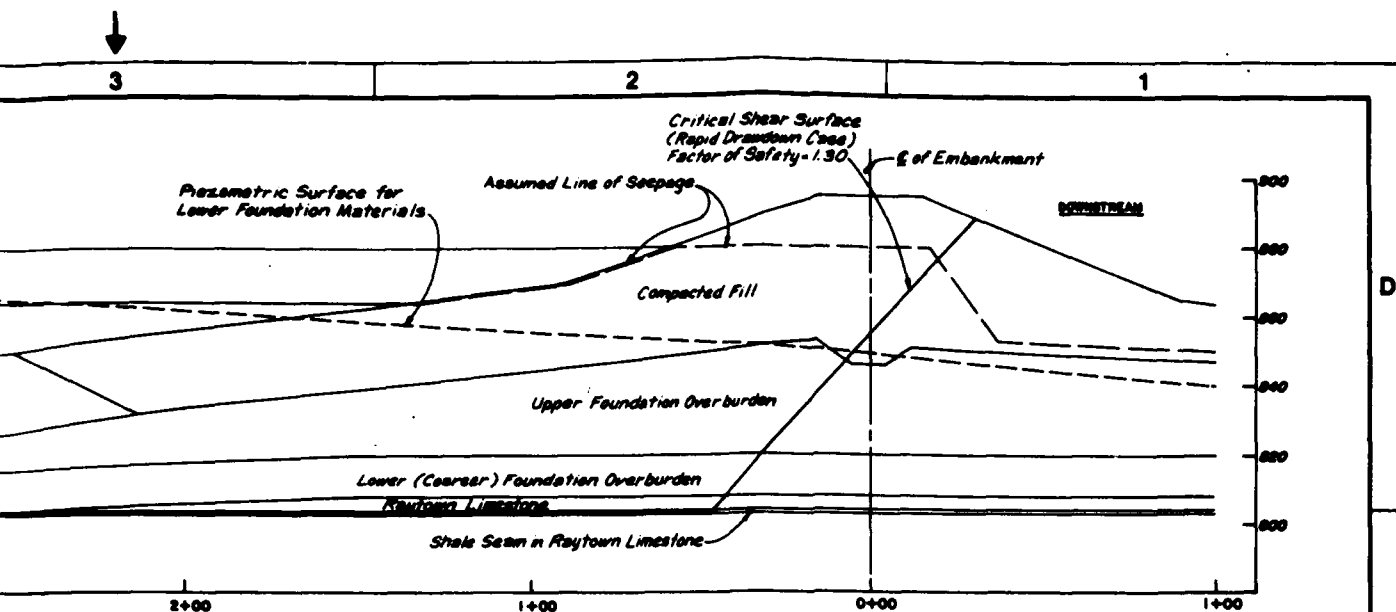
NOTES

1. SHEAR STRENGTH PARAMETERS FOR SOILS COMPRISING THE LEFT ABUTMENT ARE REFERENCED ON PLATE NO. 26.
2. SHEAR STRENGTHS FOR MOST OF THE MATERIALS WERE DERIVED FROM THE STRENGTH ENVELOPES SHOWN ON THIS PLATE. THE S STRENGTH WAS ASSIGNED TO THE LOWER COARSE FOUNDATION OVERBURDEN MATERIAL AND THE RESIDUAL STRENGTH WAS ASSIGNED TO THE SHALE SEAM IN THE RAYTOWN LIMESTONE.
3. THE ASSUMED LINE OF SEEPAGE IS USED TO DETERMINE WATER PRESSURES ACTING ON THE COMPACTED FILL AND BEING FILL MATERIALS.
4. THE PIEZOMETRIC SURFACE IS USED TO DETERMINE WATER PRESSURES ACTING ON LOWER COARSE FOUNDATION OVERBURDEN AND RESIDUAL MATERIALS. THE PIEZOMETRIC SURFACE IS DERIVED FROM WATER LEVELS WHICH WERE READ IN A LINE OF PIEZOMETERS AT STATION 110+00 AFTER THE INSTALLATION OF RESIDUAL MEASURES AND PROJECTED TO THE REQUIRED POOL ELEVATION.
5. WATER PRESSURES FOR THE UPPER FOUNDATION OVERBURDEN MATERIAL ARE INTERPOLATED VALUES FROM THE ASSUMED LINE OF SEEPAGE AND PIEZOMETRIC SURFACE.
6. THE RAYTOWN LIMESTONE BEDROCK MEMBER IS SHOWN TO EXTEND 267 FEET UPSTREAM. THIS UPSTREAM LIMIT WAS DETERMINED FROM THE TOP OF BEDROCK CONTOURS AND SUBCROP MAP (SEE PLATE NO. A-3) AND FROM THE GULL LOG DETAILING THE INSTALLATION OF PIEZOMETER P-110-2 (SEE PLATE NO. 12).
7. A COMPUTER PROGRAM ENTITLED UTENAS2 WAS USED TO PERFORM THE STABILITY ANALYSIS. OPTIONS WERE SELECTED WHICH ALLOWED THE PROGRAM TO SEARCH FOR THE CRITICAL SURFACE CORRESPONDING TO A MINIMUM FACTOR OF SAFETY AND TO SOLVE FOR THE FACTOR OF SAFETY UTILIZING SPENCER'S PROCEDURE.

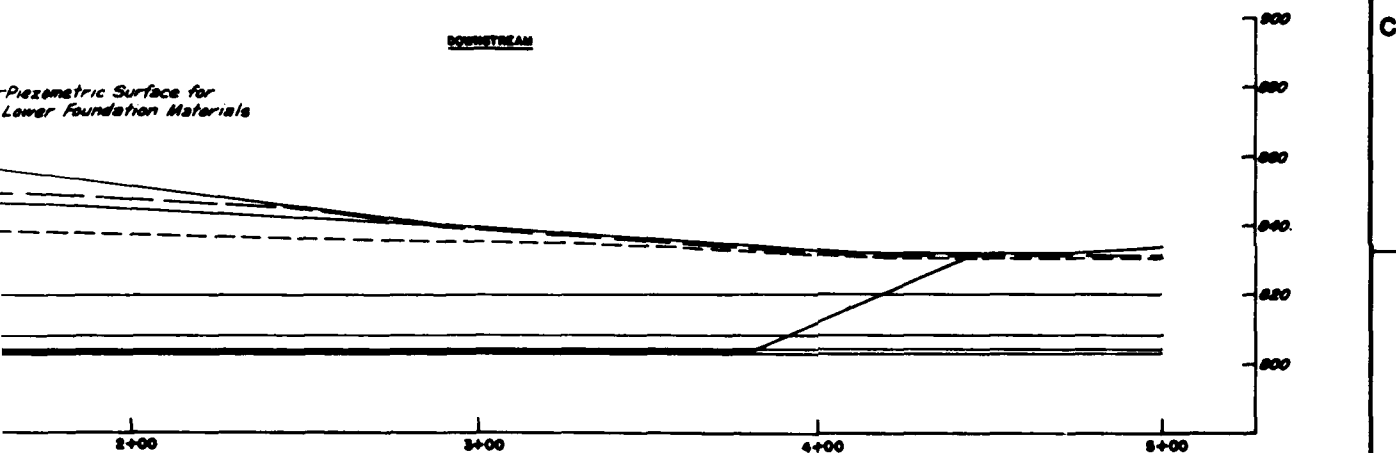
Revisions			
Symbol	Description	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	M.S.R.	LITTLE PLATE OVER, MISSOURI BENTVILLE LAKE EMBANKMENT CRITERIA REPORT	
Drawn by:	V.A.S.	PARTIAL POOL STABILITY ANALYSIS (LEFT ABUTMENT)	
Checked by:	J.B.H.	Scale:	AS SHOWN
Submitted by:	F.C.H.	MARCH 1987	
		File No.	RP-3-1779

SUPPLEMENT NO 1 PLATE NO. 29





STATION 110+00
SECTION - RAPID DRAWDOWN CASE



STATION 110+00
SECTION - STEADY SEEPAGE CASE

NOTES

1. SHEAR STRENGTH PARAMETERS FOR SOILS COMPRISING THE LEFT ABUTMENT ARE REFERENCED ON PLATE NO. 30.
2. SHEAR STRENGTHS FOR MOST OF THE MATERIALS WERE DERIVED FROM THE STRENGTH ENVELOPES SHOWN ON THIS PLATE. THE S STRENGTH WAS ASSIGNED TO THE LOWER COARSE FOUNDATION OVERBURDEN MATERIAL AND THE RESIDUAL STRENGTH WAS ASSIGNED TO THE SHALE SEAM IN THE RAYTOWN LIMESTONE.
3. THE ASSUMED LINE OF SEEPAGE IS USED TO DETERMINE WATER PRESSURES ACTING ON THE COMPACTED FILL AND BERM FILL MATERIALS.
4. THE PIEZOMETRIC SURFACE IS USED TO DETERMINE WATER PRESSURES ACTING ON LOWER COARSE FOUNDATION OVERBURDEN AND BEDROCK MATERIALS. THE PIEZOMETRIC SURFACE IS DERIVED FROM WATER LEVELS WHICH SHALL BE IN A LINE OF PIEZOMETERS AT STATION 110+00 AFTER THE INSTALLATION OF PERMANENT MEASURES AND PROJECTED TO THE REQUIRED POOL ELEVATIONS.
5. WATER PRESSURES FOR THE UPPER FOUNDATION OVERBURDEN MATERIAL ARE INTERPOLATED VALUES FROM THE ASSUMED LINE OF SEEPAGE AND PIEZOMETRIC SURFACE.
6. THE RAYTOWN LIMESTONE BEDROCK MEMBER IS SHOWN TO EXTEND 80 FT UPSTREAM, THUS UPSTREAM LEFT WAS DETERMINED FROM THE TOP OF SPRING CONTAINING AND SLURRY AND ONE PLATE NO. 4-20 AND FROM THE SHELL LOG DETAILING THE INSTALLATION OF PIEZOMETER P-110-2 ONE PLATE NO. 12.
7. A COMPUTER PROGRAM ENTITLED LITEXAS WAS USED TO PERFORM THE STABILITY ANALYSES. OPTIONS WERE SELECTED WHICH ALLOWED THE PROGRAM TO SEARCH FOR THE SHEAR SURFACE CORRESPONDING TO A GIVEN FACTOR OF SAFETY AND TO SOLVE FOR THE FACTOR OF SAFETY UTILIZING SPENCER'S PROCEDURE.



Envelope
by Seepage Cases

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by: U.S.G.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE EMBANKMENT CRITERIA REPORT		
Drawn by: V.A.S.	RAPID DRAWDOWN AND STEADY SEEPAGE STABILITY ANALYSES (LEFT ABUTMENT)		
Checked by: J.B.H.	Scale: AS SHOWN		
Submitted by: R.C.W.	MARCH 1987	File No. RP-3-1780	

SUPPLEMENT NO 1 PLATE NO. 30

APPENDIX A

APPENDIX A

OPERATION AND MAINTENANCE MANUAL

SMITHVILLE LAKE
LITTLE PLATTE RIVER, MISSOURI

APPENDIX V

EMBANKMENT CRITERIA AND
PERFORMANCE REPORT

SUPPLEMENT NO. 1

APPENDIX A

SMITHVILLE DAM
LEFT ABUTMENT STABILITY REPORT

July 1984

DEPARTMENT OF THE ARMY
Kansas City District, Corps of Engineers
Kansas City, Missouri

OPERATION AND MAINTENANCE MANUAL

SMITHVILLE LAKE
APPENDIX V
EMBANKMENT CRITERIA AND
PERFORMANCE REPORT

SUPPLEMENT NO. 1

LEFT ABUTMENT REMEDIAL MEASURES

APPENDIX A

CONTENTS

<u>Section</u>	<u>Title</u>	<u>Page</u>
1.	Introduction	V-1-A-1
2.	Geology	V-1-A-2
	a. Glacial History	V-1-A-2
	b. Overburden	V-1-A-3
	c. Bedrock	V-1-A-3
3.	Design Criteria	V-1-A-3
	a. Design Measures, Left Abutment	V-1-A-3
	b. DM Stability	V-1-A-4
4.	Left Abutment Embankment-Stability Analysis	V-1-A-4
	a. Method of Analysis	V-1-A-5
	b. Preliminary Analysis	V-1-A-5
5.	Field Investigation	V-1-A-6
	a. Additional Sampling Rationale	V-1-A-6
	b. Exploratory Drilling and Instrumentation Installation	V-1-A-6
	c. Results of Exploratory Drilling Program	V-1-A-7
	d. Instrumentation Monitoring	V-1-A-8
6.	Test Wells	V-1-A-9
	a. Pumped Test Wells	V-1-A-9
	1. Installation and Development	V-1-A-9
	2. Pump Testing	V-1-A-9
	b. Test Relief Wells	V-1-A-10
	1. Installation and Development	V-1-A-10
	2. Well Effectiveness	V-1-A-11

<u>Section</u>	<u>Title</u>	<u>Page</u>
7.	Shear Strength Testing	V-1-A-11
a.	Test Procedure	V-1-A-11
	1. Direct Shear and Residual Shear	V-1-A-11
	2. Triaxial Compression Consolidated-Undrained (with pore pressure measurements) - "R"	V-1-A-11
b.	Initial Tests	V-1-A-11
c.	Definitive Testing	V-1-A-12
8.	Stability Analysis Criteria	V-1-A-12
a.	Shear Strength Considerations	V-1-A-12
b.	Required Safety Factor	V-1-A-13
9.	Left Abutment Embankment - Stability Analyses	V-1-A-13
a.	Interim Analysis	V-1-A-14
b.	Final Analysis	V-1-A-14
10.	Conclusions	V-1-A-15
11.	Recommendations	V-1-A-15

Drawings

<u>Plate No.</u>	<u>Description</u>	<u>File #</u>
A-1	Embankment Plan	RP-3-1671
A-2	General Plan	RP-3-1672
A-3	Plan of Explorations - Left Abutment	RP-3-1673
A-3A	Boring Legend; Geologic Column - Left Abutment	RP-3-1674
A-4	Geologic Profile - Left Abutment; Section at 108+00 and 110+00	RP-3-1675
A-5	Top of Bedrock Contours and Subcrop Map - Left Abutment	RP-3-1676
A-6	Left Abutment Grout Curtain Profile	RP-3-1677
A-7	Logs of Explorations	RP-3-1678
A-8	Logs of Explorations	RP-3-1679
A-9	Piezometric Levels - P-110-2, P-110-3	RP-3-1680
A-9A	Piezometric Levels - P-110-3; 1984 Plot	RP-3-1681
A-10	Piezometric Levels - P-110-4, P-110-5	RP-3-1682
A-10A	Piezometric Levels - P-110-4; 1984 Plot	RP-3-1683
A-11	Piezometric Levels - P-114-1, P-119-1	RP-3-1684
A-11A	Piezometric Levels - P-114-1; 1984 Plot	RP-3-1685
A-12	PZ vs Pool Plot; P-110-3	RP-3-1686
A-13	PZ vs Pool Plot; P-110-4	RP-3-1687
A-14	PZ vs Pool Plot; P-114-1	RP-3-1688
A-15	Inclinometer Plot - I-110-1; Typical Reading	RP-3-1689
A-16	Inclinometer Plot - I-110-1; Movement vs Time at Raytown Shale Seam	RP-3-1690
A-17	Alignment Line "C"; Typical Data Sheet	RP-3-1691

Drawings (cont.)

<u>Plate No.</u>	<u>Description</u>	<u>File #</u>
A-18	Alignment Line "D"; Typical Data Sheet	RP-3-1692
A-19	Well Construction Details and Piezometer Schedule	RP-3-1693
A-20	Pump Test Drawdown Contours	RP-3-1694
A-21	Piezometric Contours - Interim Analysis	RP-3-1695
A-22	Piezometric Contours - Final Analysis	RP-3-1696
A-23	Summary of Shear Strengths Results	RP-3-1697
A-24	Plot of Shear Strength Results	RP-3-1698
A-25	Plot of Shear Strength Results - Raytown Shale Seam	RP-3-1699
A-26	Stability Section - Station 110+00	RP-3-1700
A-27	Design Parameters	RP-3-1701
A-28	Steady Seepage Stability Analysis Summary	RP-3-1702
A-29	Safety Factor vs Pool Elevation - Interim Analysis	RP-3-1703
A-30	Proposed Pressure Relief Wells - Plan	RP-3-1704
A-31	Proposed Pressure Relief Wells - Profile	RP-3-1705
A-32	Test Data Summary - Compacted Embankment Materials	RP-3-1706
A-33	Test Data Summary - Compacted Embankment Materials	RP-3-1707
A-34	Test Data Summary - Foundation Overburden	RP-3-1708
A-35	Test Data Summary - Foundation Overburden	RP-3-1709
A-36	Test Data Summary - Foundation Shales	RP-3-1710
A-37	Embankment Stability Analysis Summary	RP-3-1711

1. Introduction.--Smithville Dam is located in northwest Missouri on the Little Platte River, one mile northeast of Smithville, Missouri. The project was authorized by the Flood Control Act of 1965 with construction starting in February 1974. Impoundment was begun in October 1979 but lake filling was delayed because of real estate acquisition problems. Multipurpose pool, elevation 864.2, was first reached in June 1982. The record high pool of elevation 869.4 was reached in April 1983 and again in May 1984. Flood control pool (full pool) is elevation 876.2. A spillway crest pool is elevation 880.2.

At the 2nd periodic inspection in the fall of 1982, which was the first inspection after the pool reached multipurpose level, high piezometric response was noted in several of the piezometers downstream of centerline. Comparisons between recorded levels and those anticipated during design were made. In general, it was noted that piezometric levels in the foundation under the downstream slope were somewhat above the horizontal pervious drain under multipurpose pool conditions. It was believed from reviewing the embankment design memorandum that piezometric levels for stability studies were assumed to be at the base of the pervious drain downstream of centerline. At that time it was believed the safety factor might be somewhat lower than the 1.6 shown in the embankment design memorandum for the steady seepage case. It was recommended that close monitoring of piezometric levels be continued, particularly at Station 110+00 where the level was about 3 feet above ground surface at the toe.

In April 1983, a site visit was made to inspect the dam with a record high pool at elevation 869.37. A seep area had developed at the toe at Station 110+00, however, seepage was not enough to observe flowing quantities. Shortly after this, work was begun on the embankment criteria report and review of stability analysis for the embankment DM was started. In August 1983 complaints from an adjacent landowner about a wet area 3,000 feet below the main dike were reported, and an investigation of seepage in the left abutment was initiated. As a result of this investigation, which included installation of additional piezometers through the downstream slope and a review of the original design stability analysis, it was found that:

a. The piezometric level assumed for design in the left abutment was actually some 20 feet below the base of the horizontal pervious blanket in this area.

b. Projected piezometric levels for a spillway crest pool based on recorded data to date were about 25 to 35 feet above what was assumed for design.

c. The higher than anticipated piezometric levels meant that available shear strength of the foundation shales became more critical.

d. The uppermost bedrock units in the left abutment have been exposed to erosional unloading and glacial loading. These factors, coupled with a nearly horizontal bedrock surface and essentially flat lying sedimentary rock units, suggested the possibility of the existence of a shear zone or zones near the bedrock surface.

e. Preliminary stability analysis conducted with the projected piezometric levels and the DM design strengths showed the safety factor well below the desired 1.6 of the D.M.

Accordingly, a more extensive stability investigation was initiated, and a revised plan of lake releases was quickly put into effect. The objective of the revised plan was to reduce the chances of subjecting the dam to high pool levels. The frequency of monitoring the dam was increased for all pool levels above multipurpose. Further, two inclinometer devices were installed in the most critical areas at Stations 108+00 and 110+00. The investigation included additional drilling and sampling, and installation of additional piezometers in the left abutment area. Since there were immediate concerns for the stability of the dam at high pool levels, an interim solution to improve stability was developed. It consisted of reducing piezometric levels at high pools by pumping test wells installed through the downstream slope. This action allowed time to complete the investigation and to develop a permanent solution or design measure which would assure the long term safety of the dam. Once the test wells were installed and operational, the project was returned to a normal operating plan. Following installation of the pumped test wells, four flowing test wells were installed at the toe of the embankment. The effect of these test wells were analyzed to determine if pressure relief wells at the toe would provide a satisfactory permanent solution.

2. Geology.--Smithville Lake is located near the southern limit of the Dissected Till Plains Section of the Central Lowlands Physiographic Province. Major topographic features are the maturely to submaturely developed valleys of Little Platte River, Crows Creek, and Camp Branch. Drainage patterns typical of northern Missouri are developed on thick glacial deposits resulting in gently rolling topography. Bedrock exposures are not common but can occasionally be found along the bases of valley walls of major streams. Maximum relief in the area is about 160 feet.

a. Glacial history.--Pleistocene glaciers extended into this region of Missouri approximately 750,000 years ago during the Kansan glacial episode and persisted for approximately 100,000 years. Glaciers may have also advanced into the area during the earlier Nebraskan episode. Both the Nebraskan and Kansan advances were from the north-northwest and are attributed to the Iowa ice lobe from the Keewatin ice center in Canada. Since the same general regions were traversed during both episodes, the content of resultant drift materials is similar and difficult to distinguish. The southern limit of glaciation is generally recognized as being slightly south of and approximately parallel to the present course of the Missouri River.

Pleistocene ice sheets have been compared in size and extent to those of the Antarctic which have an average central thickness of about 6,500 feet. Estimated thicknesses of marginal masses are of the order of 1,600 feet. Glacial erosion was primarily by abrasion and quarrying whereby slabs of frozen ground were sheared from and dragged forward over nonfrozen ground. Magnitudes of erosion were dependent upon the thickness and velocity of the ice mass, the nature of materials incorporated into the basal ice, and the character of surfaces overridden. Glacial sediments include nonstratified till and, less frequently, fluvio-glacial deposits of stratified silts, sands, and gravels. Drift of variable thickness has been deposited upon essentially

flat lying Pennsylvanian bedrock and is the thickest in pre-Pleistocene topographic lows.

b. Overburden.--Overburden in the vicinity of the dam is of two principal types; alluvium and glacial drift. Alluvium occupies the valleys of the Little Platte River and its tributaries and generally consists of lean and fat clays overlying clayey sands and sandy clays with minor amounts of basalt gravel. Thicknesses range from 25 to 50 feet. Upland areas are mantled with deposits of glacial drift. In the left abutment area, the drift ranges in thickness up to 85 feet and generally consists of 20 to 60 feet of till overlying 5 to 25 feet of coarser outwash sediments. Till, in general, is composed of unsorted, unconsolidated, nonstratified sediments deposited directly by and underneath glacial ice masses and consists of heterogeneous, random mixtures of clay, silt, sand, gravel, cobbles, and boulders. Till in the left abutment is predominantly fine-grained, relatively impervious silty clay and clayey silt with scattered gravel and cobbles and isolated sand lenses. Coarser materials underlying the till are meltwater sediments deposited beyond advancing or retreating ice sheets which form a permeable zone of saturated dirty sand, gravel, and cobbles. This zone is exposed near the base of the valley walls in the reservoir area just upstream of the dam and therefore, is subjected to reservoir hydrostatic pressures. The overlying less permeable fine-grained silty clays and clayey silts create a confined flow system. Seepage can potentially occur where the piezometric surface in the confined system is above the ground surface. Plate A-3 shows where these conditions exist.

c. Bedrock.--Near surface bedrock strata are of the Pennsylvanian System, Lansing and Kansas City Groups and consist of alternating beds of shale and limestone. The uppermost units within the area of this investigation are, in descending order, Raytown Limestone, Muncie Creek Shale, Paola Limestone, Chanute Shale, and Cement City Limestone. A geologic column for the left abutment is shown on plate A-3A. The top of bedrock surface in the left abutment landward of approximately dam Station 107+50 is formed by the Raytown Limestone. Top of bedrock contours and subcrop map are shown on plate A-5.

The configuration of the left abutment bedrock surface is the result of a pre-Pleistocene stream channel trending generally east-west through the abutment. It is one of two major channels mapped in the reservoir area which are associated with the ancestral Missouri River drainage system prior to the advance of Pleistocene glaciers. The other is located several miles upstream of the dam in the reservoir area. As ice masses traversed the area, existing sediments were scoured away and near-surface bedrock strata subjected to shear forces induced by ice thrusts.

3. Design criteria.

a. Design measures, left abutment.--In Design Memorandum No. 10, Smithville, "Soil Data and Embankment Design," high piezometric levels resulting from underseepage were not expected to be a problem "due to the thickness of the impervious foundation materials, the large amount of fines in the sands and gravels and the scarcity of continuous pervious layers beneath the embankment." It was thought the horizontal pervious drain would "intercept groundwater seepage from the foundation." However, precautionary

measures were taken. Random fill was placed upstream of the impervious core from Station 110+00 to the end of the dam and covered an existing draw on the upstream slope. This draw was believed to possibly intercept zones of sand and gravel which might be continuous through the abutment. A 5-foot thick clay blanket was placed on the left abutment starting at Station 117+00 and extending to elevation 880 (see plate A-1). It completed cover over the draw on the upstream slope. A centerline cutoff trench to rock was not excavated because of relatively thick overburden. However, curtain grouting was conducted through the overburden from about Station 105+00 to 109+00. Grouting extended through the Raytown, Muncie Creek, and Paola units into the Chanute Shale (see plate A-6). As part of the embankment construction, a similar draw downstream of the embankment starting high up on the abutment and trending back towards centerline was backfilled with random material.

b. DM stability.--Water pressures during the design analysis were computed based on the assumption of hydrostatic pressure below the saturation line. For the left abutment section, the saturation line under the downstream slope was in the foundation at elevation 825.

For the original embankment design, a minimum safety factor of 1.6 was sought when both the partial pool and steady seepage cases used a peak strength approach and when the shale-overburden contact formed part of the failure surface. When a residual shale shear strength approach was used, a safety factor in excess of 1.0 was sought. Bedrock was assumed to be shale in all cases of the DM stability analysis. The higher strengths for overburden or limestone, where present, were not used since "(1) there may be remnants of weathered shale above the limestone; and (2) it would require thin, partially weathered, jointed limestone to carry very large forces."

The stability analyses were conducted using a computer program entitled "Stability Analysis-Wedge Method" (File No. 41-R3-C102), commonly referred to as KC SLOPE. KC SLOPE is capable of searching for a minimum safety factor for a wedge-shaped failure surface. The program accounts for water forces assuming hydrostatic pressure beneath the assigned saturation line. Analyses were conducted in accordance with EM 1110-2-1902 (1968 draft) with the slope of $E_A = .08$, and $E_p = 0.0$. Slope geometry is shown on plate A-26. DM design strengths and soil properties are shown on plate A-27.

Results of the steady seepage case with the failure surface occurring at the top of bedrock are as follows. A safety factor of 1.64 was obtained using peak design strengths for the left abutment section (approx sta 110+00). When residual design shale shear strengths were used, the safety factor dropped to 1.08.

4. Left abutment embankment-stability analysis.--Reanalysis of the steady seepage case of embankment stability was conducted for the left abutment section as a result of actual piezometric levels higher than those assumed during design. In general, these piezometric conditions are characterized by 1) above ground uplift levels in the foundation at the downstream toe near the left abutment; 2) unusually high centerline piezometric levels; and 3) piezometric levels in the foundation that respond from 40 to 60 percent to pool changes with little time lag (see plates A-9 to A-14).

a. Method of stability analysis.---The computer program used in the DM analysis, KC SLOPE, was not written to account for uplift forces in the foundation. For this reason, all subsequent analyses were conducted with the hand wedge method in accordance with EM 1110-2-1902 (April 1970) with the slope of $E_a = E_p = 0.0$ (since $ab > 7/8H$) and with a computer program, SLOPE8R, developed at the University of California at Berkeley. This computer program was used because it can analyze a non-circular failure surface and boundary water pressures can be assigned for the slide surface. SLOPE8R uses Spencer's procedure to calculate the safety factor for specified non-circular slip surfaces. It is a special solution of the Morgenstern and Price method in which all the interslice side forces are assumed to have the same inclination. The program satisfies both force and moment equilibrium conditions for each slice. The two unknown parameters, F (the safety factor) and θ , (the side force inclination) are varied simultaneously by iteration until a convergent solution is found with the net force and moment imbalance less than specified values. The method will not compute the same safety factor nor locate the same minimum failure surface as the hand wedge method. In satisfying moment equilibrium it usually results in solving for a different side force inclination, θ , than is assumed in the hand wedge method. In all cases, the safety factor obtained through hand wedge studies were lower than those obtained by SLOPE8R, because side force inclination was greater than that assumed in the hand wedge analysis. When the same side force is used in the hand wedge analysis, computed safety factors agree quite closely. This should be expected because both methods would then be solving for F by force equilibrium equations.

b. Preliminary analysis.---Preliminary analysis of the steady seepage case of left abutment embankment stability was conducted at Station 110+00, because recorded piezometric levels in the foundation were some 25 feet higher than were assumed in the design analysis. Stability studies used full pool phreatic conditions in the embankment portion of the slide surface and piezometric uplift levels projected for full pool conditions in the foundation portion of the slide. Projected levels were based on the recorded responses of P-110-3 and P-110-4 to changes in pool elevation. Projected uplift levels were elevation 859 at 20 feet upstream of centerline, with a straight line gradient to elevation 849.5 at 320 feet downstream of centerline. The foundation was assumed to be saturated to the base of the horizontal pervious drain.

Safety factors calculated using the projected uplift pressures on the slide surface and peak DM design strengths ($c = 0.0$, $\tan \phi = .30$ for shale) were 1.42 using SLOPE8R with $\theta = 8.0$ degrees and 1.23 using the hand wedge method. When the shale shear strength was reduced to residual DM design condition ($c = 0.0$, $\tan \phi = .16$), the safety factors decreased to 1.0 using SLOPE8R with $\theta = 6.9$ degrees and 0.92 and by the hand wedge method. DM design strengths are shown on plate A-27. A summary of stability analyses is shown on plate A-28.

5. Field investigations.

a. Additional sampling rationale.--Shale design strengths in the DM, both peak and residual, were based on laboratory tests from samples obtained from the shale units in the right abutment, outlet works area, and valley. Difficulty was encountered obtaining shale samples from the left abutment because of the thick overburden, the presence of gravels and cobbles above rock, the thinness of the shale seams, and the weathered, broken nature of the limestones overlying the shales. However, the strength of these same shale units in the outlet works did not control the selection of the design strength envelopes. Since the safety factor of the stability analysis for the left abutment was adequate, additional sampling and testing for the left abutment was not warranted for the original design.

The design shale shear strengths became more critical when higher than anticipated piezometric levels were recorded in the left abutment. The two abutments have different geologic histories. The upper units in the rock foundation of the left abutment have been subjected to both erosional unloading and glaciation, whereas the same units in the outlet works area and right abutment were protected by overlying bedrock units. It was believed either weathering and movement of the ice mass over the left abutment or valley stress relief could have caused one or more shear zones near the bedrock surface. The strength of a shear zone could be less than the design strength.

Thus, an investigation was initiated to determine if a weak zone or zones did exist. Sampling efforts were directed towards obtaining 6-inch core samples of soft shale seams in the Raytown Limestone, the contact of the Raytown and underlying Muncie Creek shale, and suspected soft seams in the upper Muncie Creek where persistent core losses had occurred in previous borings. It was also desired to determine if soft zones, shear planes or slickensides were present in the remainder of the Muncie Creek Shale or in the underlying Chanute Shale. Drilling and sampling was also conducted near Station 105+50 where it was suspected soft weathered Muncie Creek might be present because of the absence of the overlying Raytown Limestone in this area. Soft, highly weathered Muncie Creek had been excavated in the right abutment cutoff trench during construction.

b. Exploratory drilling and instrument installation.--The exploration program included a series of core borings to obtain shale samples for shear testing and installation of inclinometers to monitor embankment movements and piezometers to define pressure gradients and monitor piezometric responses to pool fluctuations. Subsurface information was obtained from explorations for this stability study and previous seepage investigation, and from preconstruction borings. A plan of explorations for the left abutment is shown on plate A-3. The plan shows instrumentation currently being monitored in the area of concern as well as locations of core borings done for this study.

Sixteen exploratory borings were completed as part of this study. Nine were 6-inch diameter core borings into bedrock which were completed as either piezometers in the overburden or as pore pressure devices in the foundation shales. Six borings were drilled just through the overburden and completed as piezometers. Overburden piezometers were constructed with the

screened interval set in the more permeable basal dirty sand, gravel and cobbles. Inclinator casing was installed in two borings. Seven of the 6-inch core borings were advanced through the Chanute Shale and terminated in the Cement City Limestone, one was advanced through the Paola Limestone and terminated at the top of the Chanute Shale, and one was advanced 5 feet into and terminated in the Raytown Limestone. Selected samples of the shale seam in the Raytown Limestone, the Muncie Creek Shale, and the Chanute Shale were preserved for shear testing. The first three core borings completed contained several thin zones of core loss and spins in the Muncie Creek and Chanute Shales. In order to determine if these resulted from the drilling process or represented other weak zones, a method was developed whereby an NX size core hole was drilled through the Chanute Shale, filled with cement grout, and the grout allowed to cure for approximately 60 hours. This hole was then overcored with a 6-inch core barrel. If thin weak zones had been present, the rigid column of grout would have provided sufficient torsional shear resistance to prevent spins and core losses. This procedure was used in C-528 which is located only 10 feet south of previously drilled C-527, in order to be able to recover and inspect a complete section of core. Core from the NX hole, from the 6-inch overcore and from C-527 were physically compared to determine if spins and core losses represented shear zones. In fact, unshattered shale was recovered in the 6-inch overcore of C-528 at the horizons where spins and core losses occurred in C-527 and NX portion of G-528.

Overburden drilling was accomplished with 9-7/8-inch rockbit or 9-1/2-inch fishtail bit except in UC-530 and UC-531 which were drilled in an area suspected to be underlain by weathered Muncie Creek Shale. In these two holes, the lowermost 12 feet of overburden was sampled with a 5-inch, fixed piston Shelby tube sampler in order to accurately locate the top of bedrock and to insure a quality sample of the contact.

Inclinator borings were drilled 5 feet into the Cement City Limestone with a 5-7/8-inch rock bit and completed with 3-inch grooved aluminum inclinator casing grouted the full depth. The lowermost 5 feet of each hole was grouted with 1:1 water:cement grout to anchor the casing firmly into the Cement City Limestone. The remainder of the annulus between the casing and drill hole was filled with grout consisting of 50 percent bentonite by weight of Portland Cement. A basic mixture consisted of a 94 lb. sack of Portland cement, 47 lb. of powdered bentonite and 38 gallons of water. This mixture forms a semi-rigid material capable of proportionally transmitting any deformation to the casing should movement occur.

Piezometers and pore pressure devices are the open tube type and are constructed of 3/4-inch PVC pipe with 1-1/2-inch diameter, 1-1/2 ft. long 0.020-inch slotted PVC well screen tips. Overburden piezometer tips are isolated and sealed in the lower, more permeable materials. Five open tube pore pressure devices were installed in the foundation shales; two in the thick shale seam in the Raytown Limestone, two in the Chanute Shale and one in the Muncie Creek Shale.

c. Results of exploratory drilling program.--At the start of the investigation the purpose of the drilling program was primarily to sample and preserve for testing, the Raytown-Muncie Creek contact. It seemed then that the most likely plane to have been disturbed by ice thrust deformation would

have been the Raytown-Muncie Creek contact since the Raytown forms the top of bedrock and consists of moderately hard limestone directly overlying a relatively soft shale. The first core hole, however, revealed slickensided planes in a very soft shale seam in the lower part of the Raytown Limestone. The seam is about 0.4 to 0.5 feet thick, approximately 1-1/2 feet above the Muncie Creek and is persistent throughout the abutment. Apparent shear planes were observed in every sample of the shale seam recovered. Several thin shale partings are present above this seam but they are not continuous. The contact of the Raytown with the Muncie Creek as well as the remainder of the Muncie Creek and the Chanute were determined to be inconsequential with respect to embankment stability.

d. Instrumentation monitoring.--Prior to the initiation of the seepage investigation, the basal layer the left abutment overburden was monitored by piezometers located upstream of centerline and at the toe at Station 110+00, and at the toe at Station 114+00 and Station 119+00. Monitoring of these piezometers was done on a quarterly basis. During the seepage investigation, additional devices were installed at the top of the dam at Station 106+00, downstream mid-slope at Station 110+00, downstream of toe at Station 110+00 and at the toe at Station 118+00. The devices confirmed the high pressures measured by the existing devices and enabled better definition of the piezometric surface. Monitoring was increased to monthly.

Recorded data showed piezometers in the left abutment foundation overburden respond rapidly to changes in pool level. Responses ranged from 40-60 percent with maximum time lag of a few days. All new installations responded similarly to existing devices (see plates A-9 - A-11A).

Beginning in April 1984 and as a result of the revised operation plan for the project, instrumentation was monitored according to the following schedule:

<u>Pool Elevation</u>	<u>Frequency</u>
864-868	weekly
868-869	biveekly
869 and above	daily

Monitoring included all piezometers, inclinometers, and alignment monuments in the left abutment area. This increased monitoring also resulted in better definition of projections of piezometric levels for a spillway crest pool condition (see plates A-12 - A-14).

Two inclinometers installed in April 1984 at Station 108+00 and Station 110+00 have shown no movement. They have been read at least weekly since installation with biveekly and daily readings at higher pool levels. Typical movement versus depth and movement along the Raytown shale seam versus time plots are shown on plates A-15 - A-16. Alignment monuments similarly have shown no indications of movement. Typical plots are shown on plates A-17 - A-18.

6. Test wells.--Seven test wells were installed in the left abutment area and tested to determine their effect of lowering the piezometric surface. Locations are shown on plate A-3. Three of the wells; W-1, W-3, and W-4; are

located in areas within which the piezometric surface is below existing ground surface and therefore, must be pumped. The other four; W-2, W-5, W-6, and W-7; are in the area within which the piezometric surface is above the ground surface and consequently, are free flowing.

a. Pumped test wells.

(1) Installation and development.--Pumped test wells were drilled with nominal 13-inch diameter rockbits and self-destroying organic polymer drilling fluid through the Raytown Limestone. Installation details are shown on plate A-19. Screen is 6-inch diameter type 304 stainless steel, continuous slot design. Casing is 6-inch diameter schedule 80 PVC pipe. Screen slot opening is 0.060-inch. Gravel pack gradation is as follows:

<u>Sieve Size</u>	<u>% Retained</u>
No. 4	12
No. 8	66
No. 16	90
No. 30	96
No. 50	99
No. 100	99.7
No. 200	100

Well development consisted of dispersing clays with a polyphosphate solution, surging with surge plunger, and high velocity water jetting through the screened section.

(2) Pump testing.--W-1, W-3 and W-4 were pump tested individually as they were completed. In addition, a long term pump test was done simultaneously pumping W-1, W-3, and W-4. W-1 was pump tested individually at 10 GPM while monitoring P-110-3, P-110-8, P-110-4, P-110-6, P-110-7, P-106-4, P-118-1 and P-119-1. No measurable responses were recorded in P-106-4, P-118-1 or P-119-1. Computed values for transmissivity (T) and storativity (S) from pumping W-1 are as follows:

P-110-3
 T = 1000 GPD/ft.
 S = 1.4×10^{-4}

P-110-4
 T = 1300 GPD/ft.
 S = 0.9×10^{-5}

P-110-8
 T = 300 GPD/ft.
 S = 0.8×10^{-4}

P-110-6
 T = 1800 GPD/ft.
 S = 0.8×10^{-4}

W-3 was pump tested at 25 GPM while monitoring P-114-1, P-113-1, P-112-1, P-110-3, P-110-8, P-110-4, P-110-6, P-110-7, P-118-1 and P-119-1. Measurable responses were recorded only in P-114-1 and P-113-1. P-112-1

was not functioning during the test. Computed values for T and S from pumping W-3 are as follows:

P-113-1

T = 3200 GPD/ft.

S = 0.5×10^{-2}

P-114-1

T = 4300 GPD/ft.

S = 0.5×10^{-2}

A pump test was started in W-4 at 10 GPM while monitoring P-110-9, P-109-1, P-110-3, P-108-2 and W-1. However, after 2 hours pumping, drawdown in the well was near the top of the screen and no responses were observed in any of the piezometers. The test was terminated and the well was redeveloped. The well was not pump tested individually again.

The rather wide range of T & S values obtained from the pump tests and the somewhat unpredictable effects of pumping each of the three wells indicate significant lateral variation in permeability of and/or thickness of the basal pervious material. The transmissivity of the pervious appears to be greatest upstation of approximately Station 112+00 and is quite small downstation of approximately Station 108+00. The higher transmissivity in the vicinity of W-3 may be caused by either a thickening of the pervious zone or the presence of higher permeability material or a combination of both. The lower values near W-4 appear to be caused by the presence of less permeable material in this area. As shown on plate A-20, the greatest drawdown over the largest area results from pumping W-1.

On 31 May 1984, a long term pump test was started with all 3 wells pumping simultaneously. W-1 was operated at an average discharge of 7.5 GPM, W-3 at an average discharge of 17 GPM and W-4 at an average discharge of 5 GPM. Again, drawdown in W-4 was rapid with no corresponding response in nearby piezometers. The pump was shutoff, removed and the well again redeveloped. No improvement in well performance was realized. The test was run for 27 hours. All piezometers installed in the basal pervious overburden were monitored during the test. Distance-drawdown plots were constructed for W-1 and W-3 from pump test data at 24 hours and these contoured as cones of heads. Drawdown contours are shown on plate A-20. These were subtracted from contours of the piezometric surface projected to the spillway crest pool for the interim analysis and the resulting contours of the piezometric surface with pumping are shown on plate A-21. These piezometric projections were based on early data and were subsequently revised based upon additional data, spring 1984.

b. Test relief wells.

(1) Installation and development.--Flowing test wells were drilled with nominal 10-inch diameter rockbits. Because piezometer levels were above ground, weighted bentonite slurry drilling fluids were used. Installation details are shown on plate A-19. Screen is 4-inch diameter type 304 stainless steel continuous slot design. Casing is 4-inch schedule 80 PVC pipe. Screen slot opening is 0.060-inch except W-5 in which slot opening is 0.030-inch. Gravel pack gradation is the same as for the pumped wells. Development was done by jetting a polyphosphate solution through the screened section and by surging with a surge plunger.

(2) Well effectiveness.--Installation of 4 flowing test wells, W-2, W-5, W-6 and W-7 was completed on 9 June 84. After all the test wells were completed and developed, they were all plugged, the piezometric surface allowed to stabilize and then the wells unplugged. Piezometers were monitored daily for the first full week after unplugging the test wells and weekly thereafter. The pool was at elevation 868.4 during this time. Total drawdown in each piezometer in response to the flowing wells was plotted, contours drawn, and these values subtracted from refined piezometric contours projected to a pool at spillway crest for the final analysis. These are shown on plate A-22.

7. Shear strength testing.--To provide design strength parameters for the interim and final analyses, consolidated-drained direct shear "S" tests, residual shear tests, and triaxial compression consolidated-undrained (with pore pressure measurement) "R" tests were conducted.

a. Test procedures.

(1) Direct shear and residual shear.--The direct shear and residual shear tests were conducted with a 3-inch square shear box in which a usually 0.5-inch thick sample is made to shear horizontally. For intact shale specimens the peak strength is obtained within .3 inches of horizontal displacement. The residual shear condition is attained by repeatedly reversing the direction of shear on the induced shear plane. Intact, precut and remolded specimens were tested. Specimens were consolidated in most cases to 6.0 tons per square foot (TSF) prior to shearing. Deformation rates were on the order of 0.2 to 0.4 inches/day. Soft shale specimens were trimmed using a band saw; a diamond rock saw was used when harder rock formed part of the specimen.

(2) Triaxial compression consolidated-undrained (with pore pressure measurement) R tests.--The R tests were conducted to determine total and effective stress strength parameters on slickensided surfaces in the soft Raytown shale seam. The slickenside was oriented at 55 degrees to 60 degrees from horizontal to assure failures developed on the slick. Specimen size was 1.4-inch diameter by 3-inch long so that 2 specimens could be trimmed from a single 6-inch core containing the near horizontal slick. Specimens were trimmed using a band saw and a supporting jig to prevent disturbance or shearing on the slick. All specimens were back-pressure saturated to obtain 100 percent saturation (Skempton's B parameter greater than .95 was required) prior to consolidation and shear. The shearing rate was selected to allow pore pressure measurements and was based on t_{50} from the consolidation data.

b. Initial tests.--A 4-inch core sample (hole I-108-1) recovered from the Raytown Limestone contained soft shale seam about 1-1/2 foot above the Muncie Creek contact. The seam had a continuous near horizontal slickensided surface but the sample was too small to test intact. There was, however, sufficient material to run a remolded specimen. The specimen was consolidated to 12 TSF, rebounded to 6 TSF and precut prior to shearing. The specimen developed a residual shear strength of $\tan \phi = 0.147$. At first this was believed to represent a lower bound residual strength. However, subsequently it was discovered that the shear surface contained "gritty" particles and the test results were considered unrepresentative.

The first 6-inch core hole (C-525) recovered the same thick shale seam in the Raytown, but as in the earlier exploratory drilling efforts, core loss and spins occurred in the upper Muncie Creek. The slick was present in the thick Raytown seam (sample 1) and direct shear and residual shear tests were conducted. A $\tan \phi = .416$ and $\tan \phi = .196$ were obtained. The Raytown/Muncie Creek contact as well as other soft seams in the Raytown and a low angle fracture in the Chanute shale were tested. These results are presented on plates A-23 - A-24. Based on these encouraging results, it was felt the suspected soft seams in the Muncie Creek which had not been successfully recovered and possibly other areas of core loss would probably dictate the design shear strength. Sampling efforts were adjusted accordingly. However, it was considered desirable to obtain at least one other sample of the Raytown slick to better define the effective stress envelope, especially any cohesion intercept, and to develop a total stress strength envelope in case this was needed for the stability analysis.

c. Definitive testing.--An R test was conducted on the Raytown slick (C-527). Although problems were encountered with the test, an interim design strength of $c' = 160$ pounds per square foot (psf), $\tan \phi' = .265$ was selected for the Raytown seam see plate A-27. Since this was considerably lower than the previous direct shear result from C-525, another direct shear test was run C-528. It did not confirm the earlier direct shear test results. This time a $\tan \phi = .289$ and $\tan \phi = .129$ was obtained. The inconsistency is best explained by the concave shape of the slick surface in the earlier test as opposed to a near planar surface in the latter. Because of the wide difference in results, it was considered prudent to run an additional R test and residual shear tests to increase the confidence level in the strength results. In the meantime, by using an overcoring technique, it was determined that suspected soft seams in the upper Muncie Creek were not present. Thus the design strength would be dictated by the slickensided surface which had been encountered in all the samples recovered from the Raytown shale seam.

Residual strengths obtained on precut specimens from the Raytown seam were nearly identical to the result from the intact specimen on the planar slick, $\tan \phi = .13$. Of some interest is the fact that the peak strengths on the precut surfaces were also quite similar to the peak strengths on the planar intact specimen. The second R test result from C-532, was $c' = 500$ psf, $\tan \phi' .31$. A final direct shear test was run at a consolidation pressure of 4 TSF. From these additional tests, the final design strength was adjusted to $c' = 250$ psf, $\tan \phi' = .268$ (see plate A-27). Thus the selected peak design strength was not significantly lower than the original design peak envelope.

8. Stability analysis criteria.

a. Shear strength considerations.--The DM and preliminary stability analyses were conducted using two different strength approaches: 1) peak design strength along the entire failure slide surface and 2) peak design strength along the failure surface in the active and passive wedges and residual shale shear strength in the central block portion of the slide. These strength approaches were discussed with MRD and OCE personnel at a site inspection in late April. It was agreed that use of the residual strength in a stability analysis may be overly conservative. However, use of peak strengths in the analysis was probably unconservative, because of strain

incompatibility between a shear zone in the shale, the foundation overburden and the embankment. With small strains in the shale, the peak strength could be developed before peak strengths are attained in the embankment and overburden.

Accordingly, a third approach was considered for the interim and final analyses which would allow use of peak strength in the active wedge portion of the failure surface and in the shale seam, but not in the passive wedge portion since relatively large displacements would be required to develop full passive resistance. It was considered reasonable to use strengths in the passive wedge (foundation overburden) which correspond to 0.5 percent strain development. Strains somewhat larger than this were required to develop the peak strength in tests on the slicksided shale surface. Test results on overburden clay samples from the left abutment and for the embankment were available from the earlier design investigation. Although record control tests from the embankment indicate higher strengths than those used for the design analysis, design strengths were not changed. (The DM strengths were determined for minimum required placement conditions, 95 percent maximum density and +3 percent optimum water content.) As a result of the extensive exploratory boring through the foundation overburden, it was concluded that a coarse-grained pervious layer was consistently present immediately above rock. Since DM tests on overburden samples were on the weaker lean and fat clays it was believed reasonable to assign a strength to the pervious layer of $c=0$, $\phi = -30$ degrees in the active wedge portion of the failure surface. In the final analysis, the Raytown Limestone above the shale seam in the active wedge was assumed to have a vertical joint and was given no shear strength. See plate A-27 for a summary of the design strengths used for the various analyses.

b. Required safety factor.--In many cases during a DM stability analysis, comparatively little is known about specific foundation conditions at a given location. The design safety factor of 1.5 required by EM 1110-2-1902 for the steady seepage case is in part, intended to provide an added degree of safety, in case possible locally weak areas do exist. Since considerable exploratory work, sampling, and testing was conducted for this investigation, a safety factor of 1.3 for the left abutment embankment is appropriate for the detailed stability analyses conducted.

The minimum safety factor of 1.3 must be obtained using the more conservative hand wedge method (with horizontal E_1 and E_2 forces) instead of SLOPESR. The hand wedge method has been used extensively by the District in past stability analyses and thus is an integral part of our slope design experience. SLOPESR was used to expedite the analyses and to find the most critical combination of slope and piezometric levels. The results were checked by the hand wedge method.

9. Left abutment embankment stability analysis.--The re-analysis of the left abutment section for embankment stability was conducted in three phases. A preliminary stability analysis used DM strengths and projected piezometric uplift levels based on recorded data. The results of the preliminary analysis led to concerns for the stability of the dam at high pool levels. Subsequent analyses included an interim analysis for an interim solution and final analysis for a permanent solution.

a. Interim analysis.--An early objective of the investigation was to determine whether an interim solution could be effected which would not require the construction of an expensive stability berm. Further, emergency construction of a berm would have in all likelihood been rockfill and made additional and badly needed exploratory work very difficult, if not impossible.

The interim solution which was discussed with MRD in June 1984, consisted of pumped test wells installed through the downstream slope. The interim analysis was conducted at Station 110+00 with pool levels at multipurpose pool and at spillway crest pool. The third strength approach was used for the analysis, using peak strengths in the active wedge and central block and strength corresponding to 0.5 percent strain in the passive wedge. The strengths used are shown on plate A-27.

Piezometric levels used corresponded to recorded values for multipurpose pool and projected values for a spillway crest pool condition. Drawdown obtained from the pumped test well data at multipurpose pool was subtracted from the projected piezometric levels at spillway crest. The piezometric contours used in the interim analysis are shown on plate A-21. Results of these studies are summarized below and on plate A-28.

Safety Factor

	Hand Wedge	SLOPE8R	
Multipurpose Pool, El. 864.2	1.40	1.66	($\theta=8.5^\circ$)
Spillway Crest Pool, El. 880.2	1.20	1.41	($\theta=8.0^\circ$)
Spillway Crest Pool with pumped test wells	1.32	1.55	($\theta=8.9^\circ$)

From the plot of safety factor versus pool level it was determined that pumping would begin whenever a pool of elevation 872 or higher is forecast to assure a safety factor of greater than 1.3 (see plate A-29).

While this interim solution provides an adequate safety factor, it is not desirable as a permanent solution since it requires a specific human response for the life of the project whenever high pools are experienced. There are concerns that at some time in the future, when current personnel are no longer around, that the necessity of maintaining and of pumping the test wells each time the pool approaches elevation 872 may be forgotten. The solution is also dependent on complete mechanical and electrical reliability of pumps and generators; any significant downtime during periods of high pool would be potentially dangerous.

b. Final analysis.--The final stability analysis consisted of locating the most critical slope in the abutment area for the steady seepage case at spillway crest pool, both with and without pressure relief wells at the toe. Piezometric levels were projected for this pool condition, using the most recent recorded piezometric levels. This analysis of the more recent piezometric data indicated some reduction in anticipated pressure levels was justified. Drawdown data obtained from the installation of 4 test relief wells near Station 110+00 was used to evaluate the effect on stability. The drawdown obtained with the pool at elevation 868 was applied directly to refined projected spillway crest piezometric levels (see plate A-22).

With the completion of laboratory shear strength testing, the design shear strength for the shale foundation was further refined as discussed previously. This final design shear strength was used with the same strength approach used in the interim analysis, that is, peak strengths in the active wedge and central block, strength at .5 percent strain in passive wedge and a required safety factor of 1.3 (see plate A-27). Results at Station 110+00 are as follows:

	Safety Factor	
	Hand Wedge	SLOPESR
Spillway Crest Pool, without relief wells	1.25	1.47 (0-8.7°)
Spillway Crest Pool, with relief wells	1.30	1.53 (0-8.7°)

The safety factors above are conservative in that piezometric levels assumed for the "with wells" condition are conservative. The effect of the wells at a spillway crest pool should be greater than with the pool at elevation 868.

Further the safety factor will be increased by lowering the outfall of the wells with the installation of a buried collector pipe at a depth of 3-4 ft. and installing more pressure relief wells at the toe and through the embankment with the "outfall" in the pervious drain. (See recommendations.)

10. Conclusions.--The following conclusions are made.

- a. The embankment on the left abutment as constructed has an inadequate safety factor with respect to slope stability for a spillway crest pool, steady seepage case, because of higher than anticipated piezometric levels in the foundation.
- b. An interim solution of pumped test wells through the downstream slope will provide an adequate safety factor until a permanent solution can be implemented. The pumping will commence whenever the pool is predicted to rise to elevation 872 or higher.
- c. Construction of a stability berm is not warranted. In addition to the cost (\$500,000 +) and environmental impacts, it cannot be implemented as quickly or as efficiently as pressure relief wells. Installation of pressure relief wells at the toe area will provide an adequate safety factor for spillway crest pool, steady seepage conditions.

11. Recommendations.--The following recommendations are made.

- a. Pumped test wells should continue to be implemented as the interim solution until the permanent solution is in place.
- b. The following schedule for monitoring of the existing instrumentation in the left abutment area is recommended.

<u>Pool Elevation</u>	<u>Frequency</u>
865.5 and below	monthly
865.4 to 869	weekly
869 - 872	biweekly
872 and above	daily

c. Additional pressure relief wells should be installed in the toe area as shown on the plate A-30. The well outfalls should be connected to a buried collector pipe. Additional wells should be installed through the downstream slope with a riser section containing a well screen at the pervious horizontal sand blanket to provide pressure relief. Estimated cost for the above remedial measures are as follows:

12 additional pressure relief wells @\$6,000 ea	\$72,000
Buried collector system	8,000
	<u>\$80,000</u>

d. It is also recommended that the wells be installed with the District's own hired labor drill crews for the following reasons:

(1) With the existing piezometric surfaces well above the ground surface in the area where the wells for a final solution would be installed, the risk of an inexperienced and/or inept contract drill crew losing a hole or otherwise compromising the integrity of the dam is too great.

(2) The Kansas City District drill crews have successfully and efficiently installed four test wells in this area without any compromise of the safety of the dam.

(3) Due to the heterogeneity of the subsurface conditions in the left abutment area, some adjustments in well location, screen locations, screen size, etc., may be required based on the conditions encountered. District drill crews can more readily adapt to such required changes.

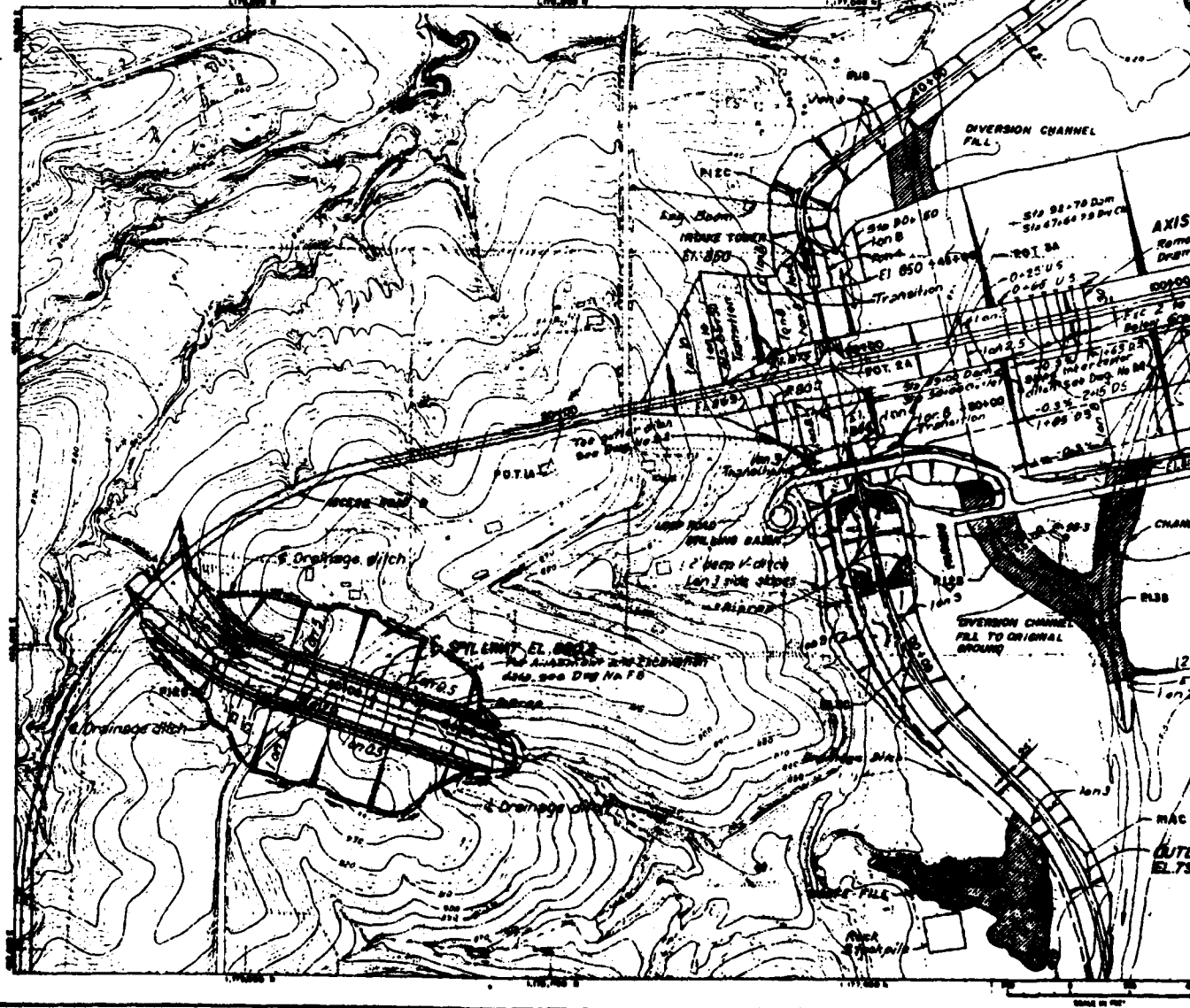
(4) If funds were made available for implementation of the proposed permanent well solution, District drill crews could have all of the work completed by time it would be put out for bids under normal procurement procedures.

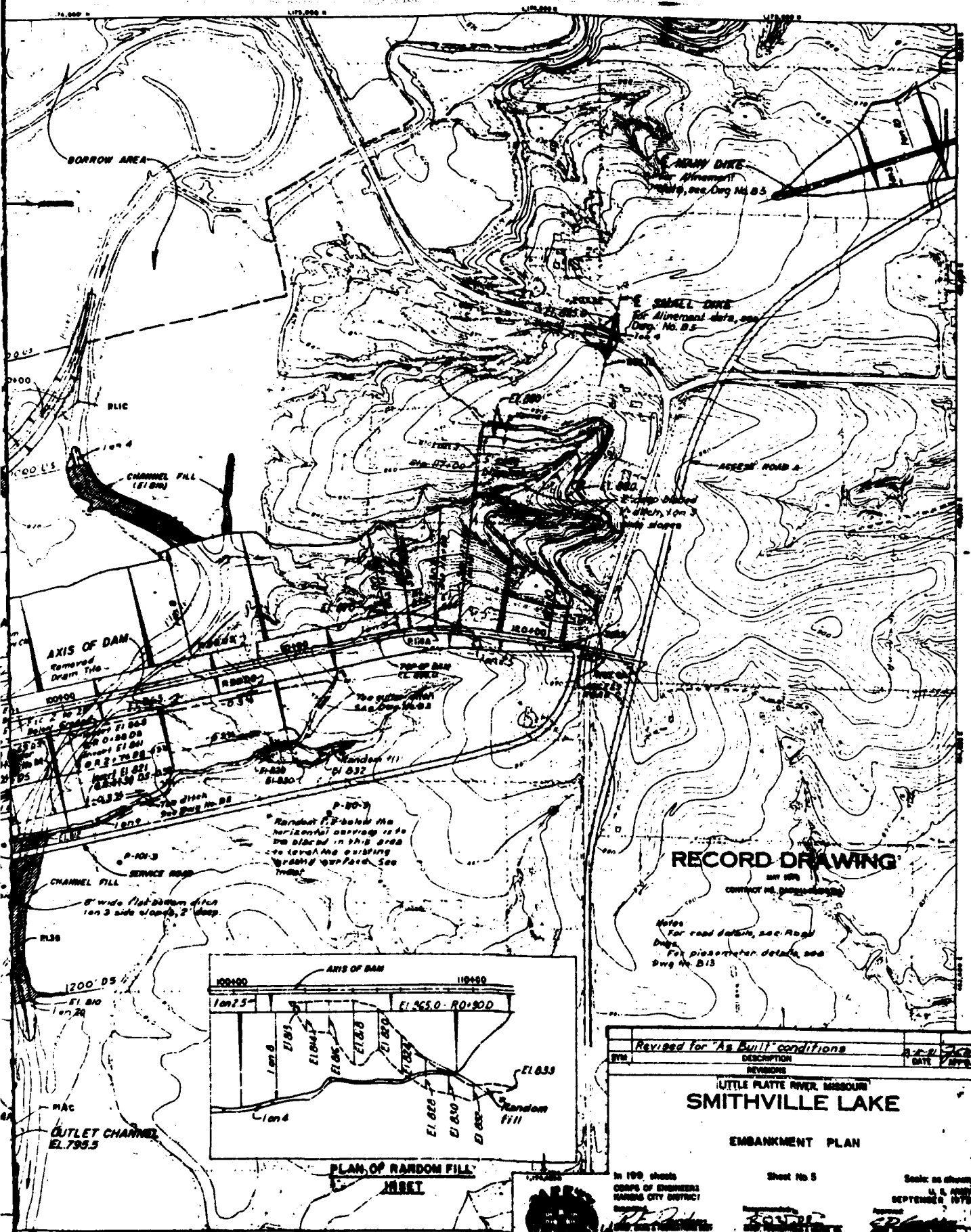
(5) For a viable dam safety program it is essential that in-house drill crew forces be able to respond quickly and effectively to dam emergencies. In order to provide this response it is necessary for in-house drill crew forces to have drilling experience in conditions and of the type that would be encountered in a dam emergency. Although the present conditions at Smithville are not an emergency, the opportunity to install relief wells and a collection system under high piezometric head conditions is an excellent means of further honing and refining the expertise that presently exists in the Kansas City District.

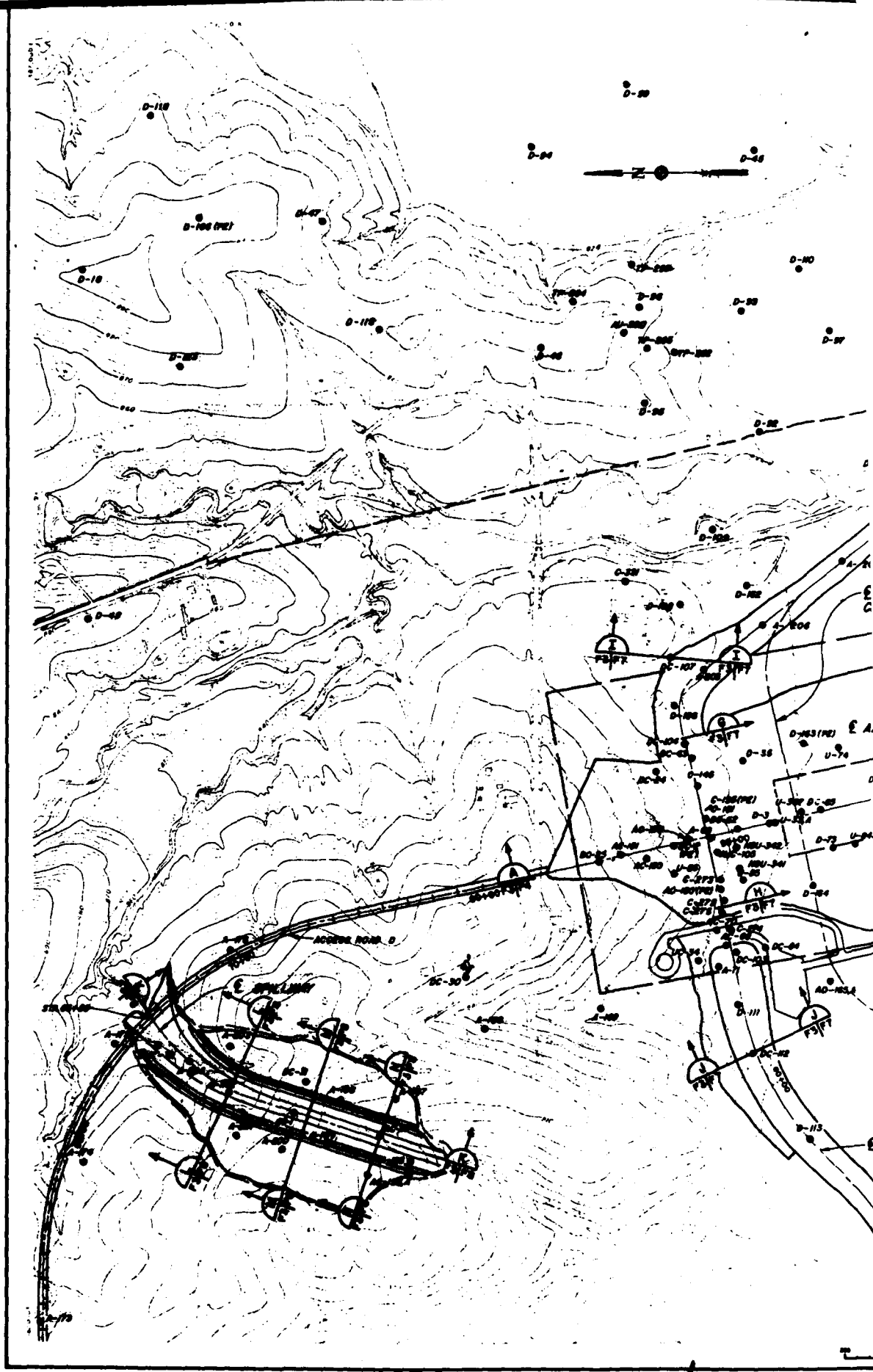
DRAWINGS

DRAWINGS

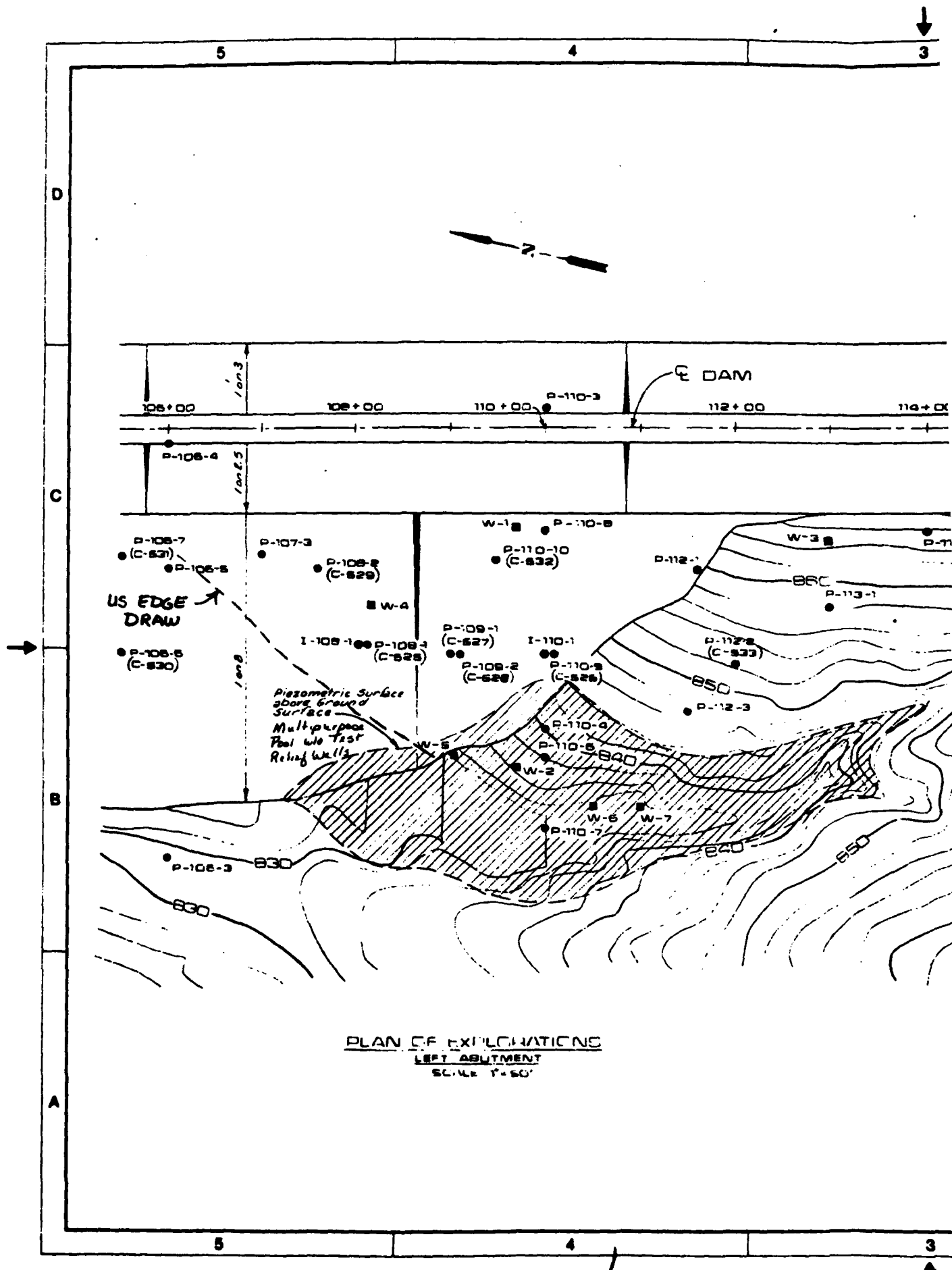
ALIGNMENT DATA												
P.I.	POB	LATITUDE	DEPARTURE	AZIMUTH	DISTANCE	STATION	P.C. STA	P.T. STA	A	D	T	R
AXIS DAM												
1A	1172.972	27° 43' 20" N		347° 30'	900.00'	60+00						
2A	1172.994	00° 48' 35" S		347° 30'	370.00'	89+00	Station 30+00	Outlet Works				
3A	1176.702	77° 39' 29" S		347° 30'	2281.63'	92+70	Station 47+66	29 Divergence Channel				
4A	1174.505	23° 48' 48" S		7° 30'	630.00'	14+00	09+17	00 09	20° 41'	6° 40'	151.54'	859.41' 300.00'
5A	1173.870	70° 48' 40" S		11° 30'	300.00'	119+48	47+12	3+00	4° 41'	1°	200.00	3729.55' 400.00'
6A	1173.576	77° 39' 29" S				124+00						
DIVERSION CHANNEL												
7C	1175.942	23° 48' 48" S		142° 30'	1801.75'	24+45	48	58+05	44° 30' R	5°	468.81'	1145.92' 890.00'
8B	1176.836	64° 48' 42" S		77° 30'	752.34'	59+46	75	41+39	42° 53' L	30°	121.67'	190.98' 216.67'
9A	1176.732	77° 39' 29" S		77° 30'	700.00'	47+64	29	Station 92+70	Dam Axis			
10B	1176.581	26° 48' 39" S		31° 00'	410.00'	53+00	18	56+10	18° 46' 30" L	15°	164.11'	381.97' 310.00'
11B	1176.229	64° 48' 42" S		77° 30'		56+10	56	60+19	46° 30' R	15°	164.11'	381.97' 310.00'
OUTLET WORKS												
12C	1173.912	23° 48' 48" S		142° 30'	1610.00'	24+45	48	58+05	44° 30' R	5°	468.81'	1145.92' 890.00'
13C	1177.815	53° 48' 42" S		77° 30'	580.00'	49+25	00	51+41	63° 11'	30°	121.67'	190.98' 216.67'
14A	1177.091	00° 48' 35" S		77° 30'	1020.73'	50+00	Station 89+00	Dam Axis				
15C	1176.873	07° 48' 29" S		43° 30'	1010.21'	57+50		62+70	32° 11'	6° 09'	244.97'	93.03' 530.00'
16C	1176.145	64° 48' 42" S		56° 30'		63+54		74+04	51° 41'	8°	465.48'	354.33' 630.00'

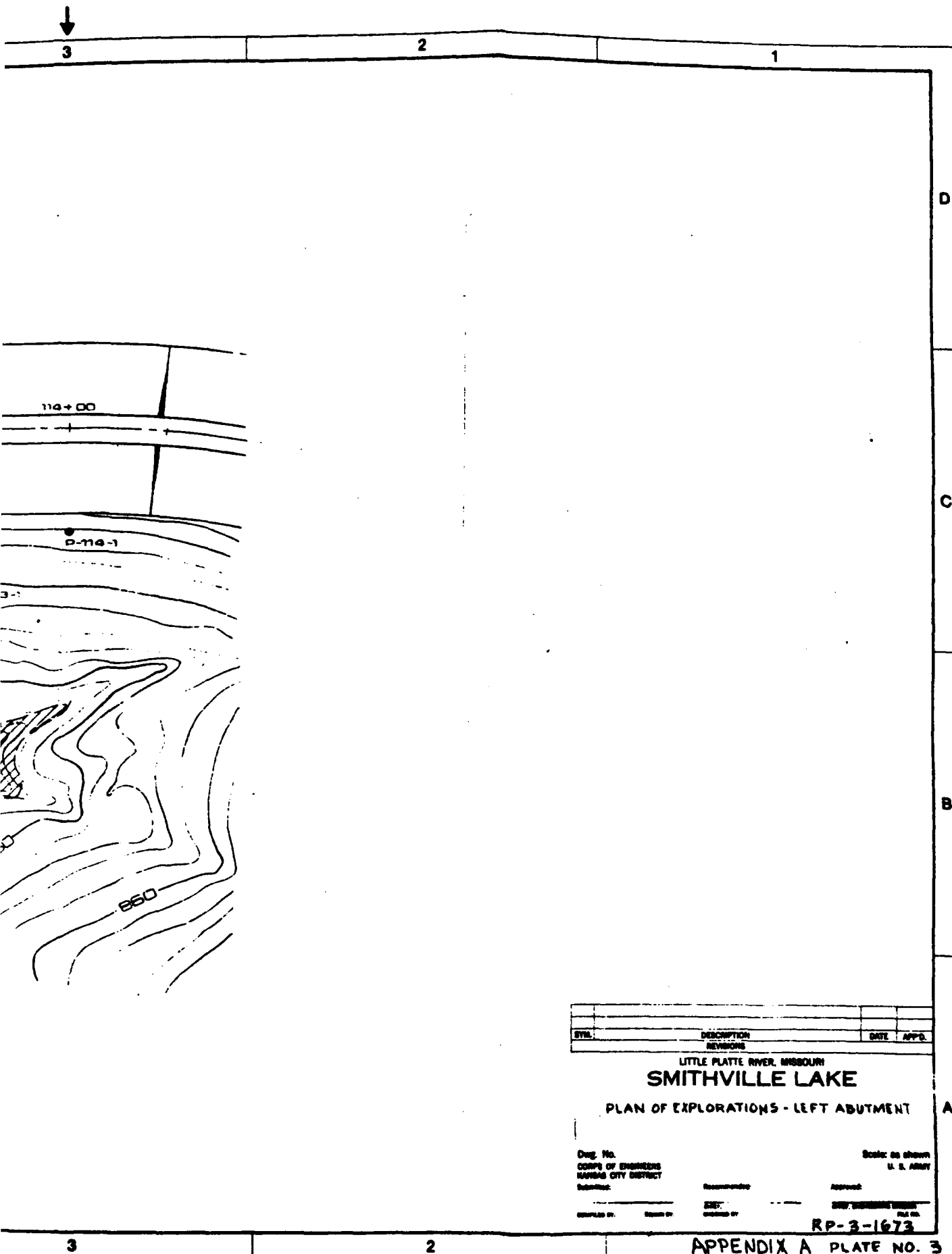






15-1-7





GEOLOGIC COLUMN - LEFT ABUTMENT, SMITHVILLE DAM

OVERBURDEN variable thickness
Silty clay w/occasional lenses of fine sand & occasional fine to coarse gravel, underlain by clayey silt & fine to medium sand w/clay & occasional gravel. Clayey, sandy gravel w/limestone chert & quartzite cobbles & boulders.

LAME FN - SHALE average thickness 7'
Soft, fissile, slightly calcareous, dark gray.

DELA FN - BAYVIEW LIMESTONE 6'
Moderately hard to hard, crystalline to dense fossiliferous & massive in upper part. Thin-bedded w/interbedded soft, calcareous shale w/pervasive soft to v soft clayey shale (approximately 1 to 1.5' above base of Bayview) overlying moderately hard, dense to finely crystalline fossiliferous limestone w/shaly partings in lower part.

DELA FN - MINNIE CREEK SHALE 7'
Soft to moderately hard w/occasional v soft thin laminae, platy, dk gray to black, calcareous, and to black carbonaceous at part base.

DELA FN - PEARL LIMESTONE 15'
Moderately hard to hard, finely crystalline, massive w/occasional argillaceous partings, gray, fossiliferous.

CHERRYVALE FN - SHALE 10'
Soft, occasional v soft or moderately hard fissile to platy, dk gray to black, calcareous, wavy clay partings in lower part, soft & clayey w/occasional oolitic nodules near base.

DELA FN - CHERRY CITY LIMESTONE 9'
Moderately hard to hard, crystalline to dense, thin to med-bedded; lt gray & argillaceous, w/white upper part, wavy discontinuous sh partings in lower.

CHERRYVALE FN - QUINCY SHALE 11'
v soft, med fissile, dk green-gray, argillaceous.

CHERRYVALE FN - MOUNTAIN LIMESTONE 3'
Hard, med-bedded, & crystalline w/wavy sh partings.

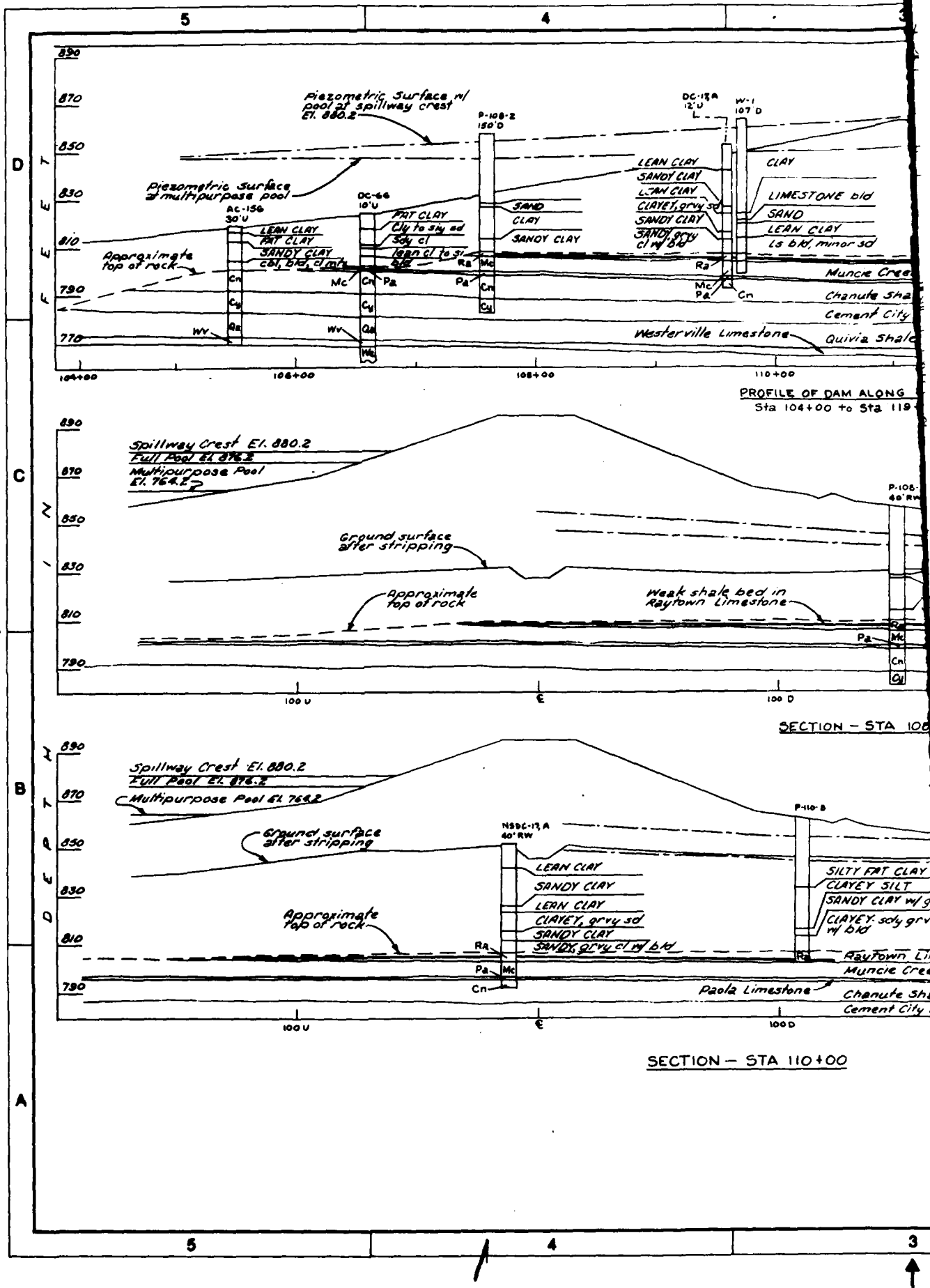
CHERRYVALE FN - WEA SHALE 20'
Soft, silty, massive, varicolored.

SMITHVILLE DAM
LEFT ABUTMENT

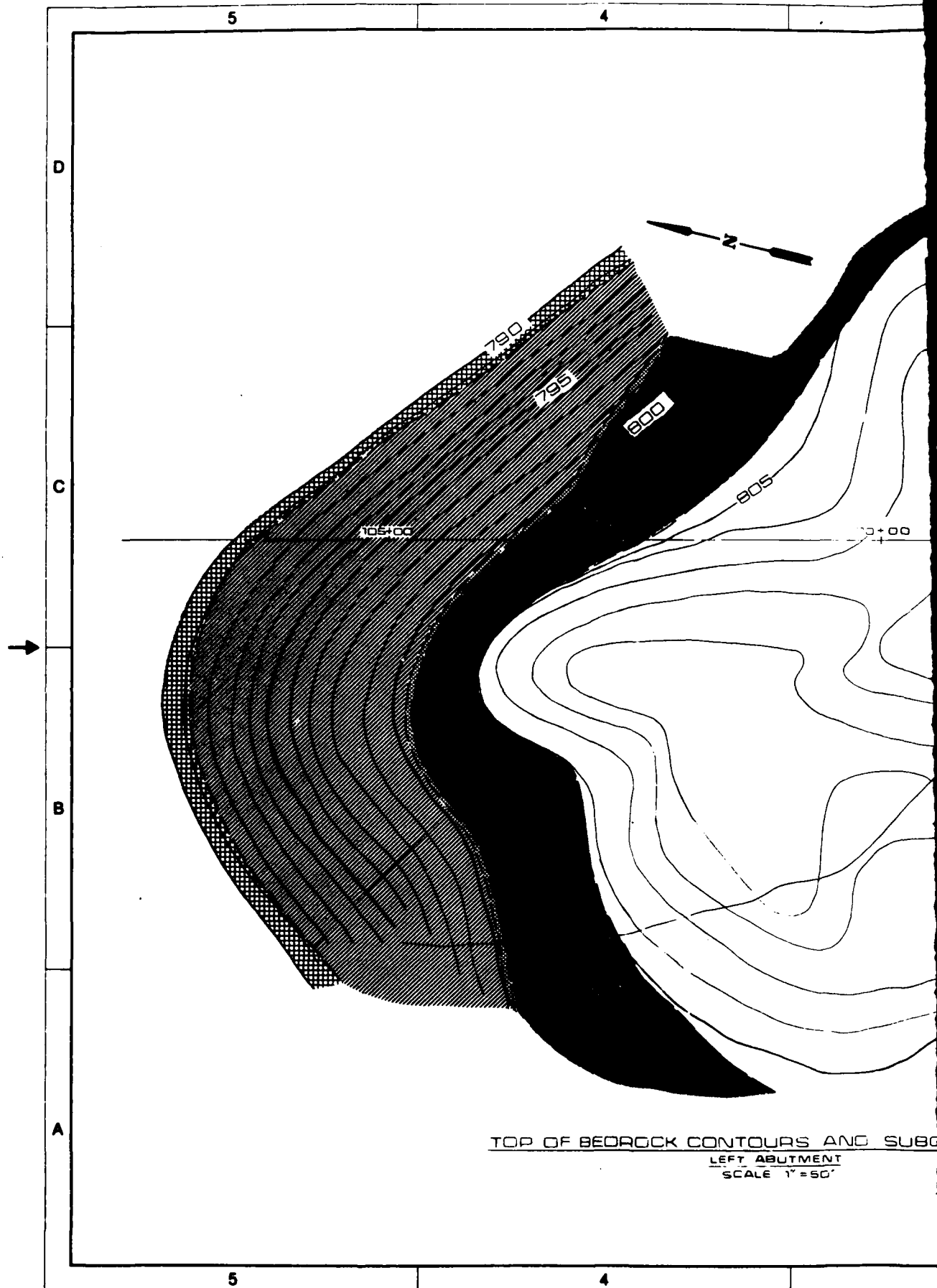
BORING LEGEND
GEOLOGIC Column -
LEFT ABUTMENT

APPENDIX A
PLATE 3A

RP-3-1674
JULY 1984



-1-7



15-1-7

↓

3

2

1

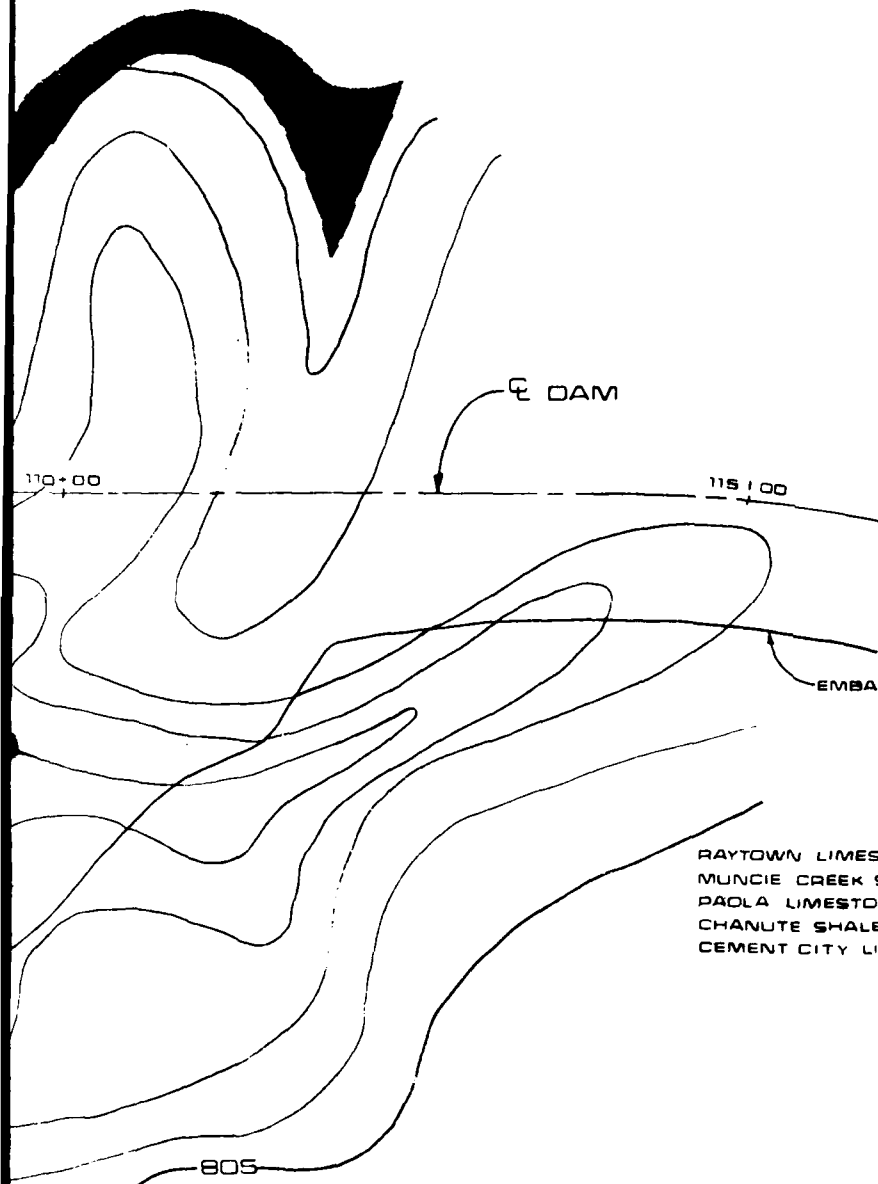
D

C

B

A

←



☒ DAM

110+00

115+00

EMBANKMENT TOE TRACE

805

- RAYTOWN LIMESTONE
- MUNCIE CREEK SHALE
- PADLA LIMESTONE
- CHANUTE SHALE
- CEMENT CITY LIMESTONE

NO SUBCROP MAP

SYBL.	DESCRIPTION	DATE	APPD.
REVISIONS			

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
 TOP OF BEDROCK CONTOURS
 AND SUBCROP MAP
 LEFT ABUTMENT

Desg. No. _____ Scale: as shown
 CORPS OF ENGINEERS U. S. ARMY
 KANSAS CITY DISTRICT
 Submitted: _____ Recommended: _____ Approved: _____
 Checked by: _____ Date: _____
 Drawn by: _____
 FILE NO. **RP-3-1676**

APPENDIX A PLATE NO. 5

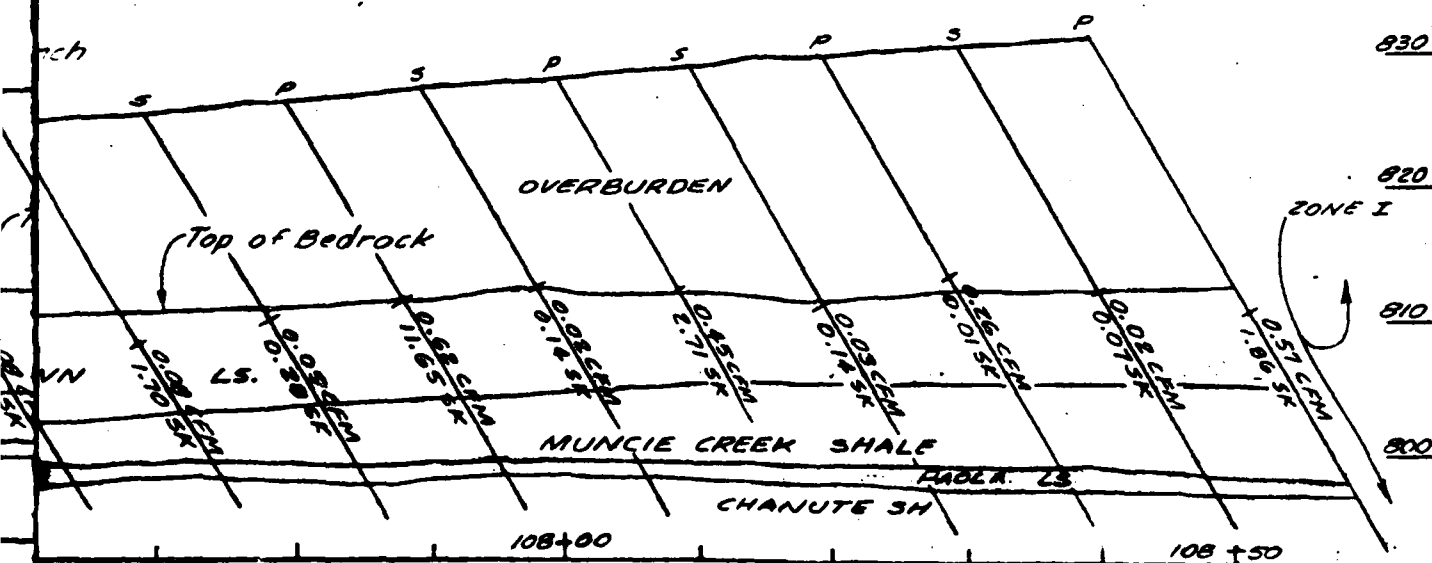
3

2

↑

1

2



STA 105+10 TO STA 108+95

LOOKING UPSTREAM

LEGEND

P Primary hole
S Secondary hole
N.G. Hole not pressure grouted

Diagram illustrating the orientation and measurement of a grout hole:

- Rockier depth
- Pressure Test
- Water take in gallons per minute
- Grout take in sacks of cement
- Grout holes inclined 30° landward and parallel to dam axis

0 10
SCALE IN FEET

**LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE**

LEFT ABUTMENT
GROUT CURTAIN PROFILE

in 1 street

Sheet No. 1

Book: as shown

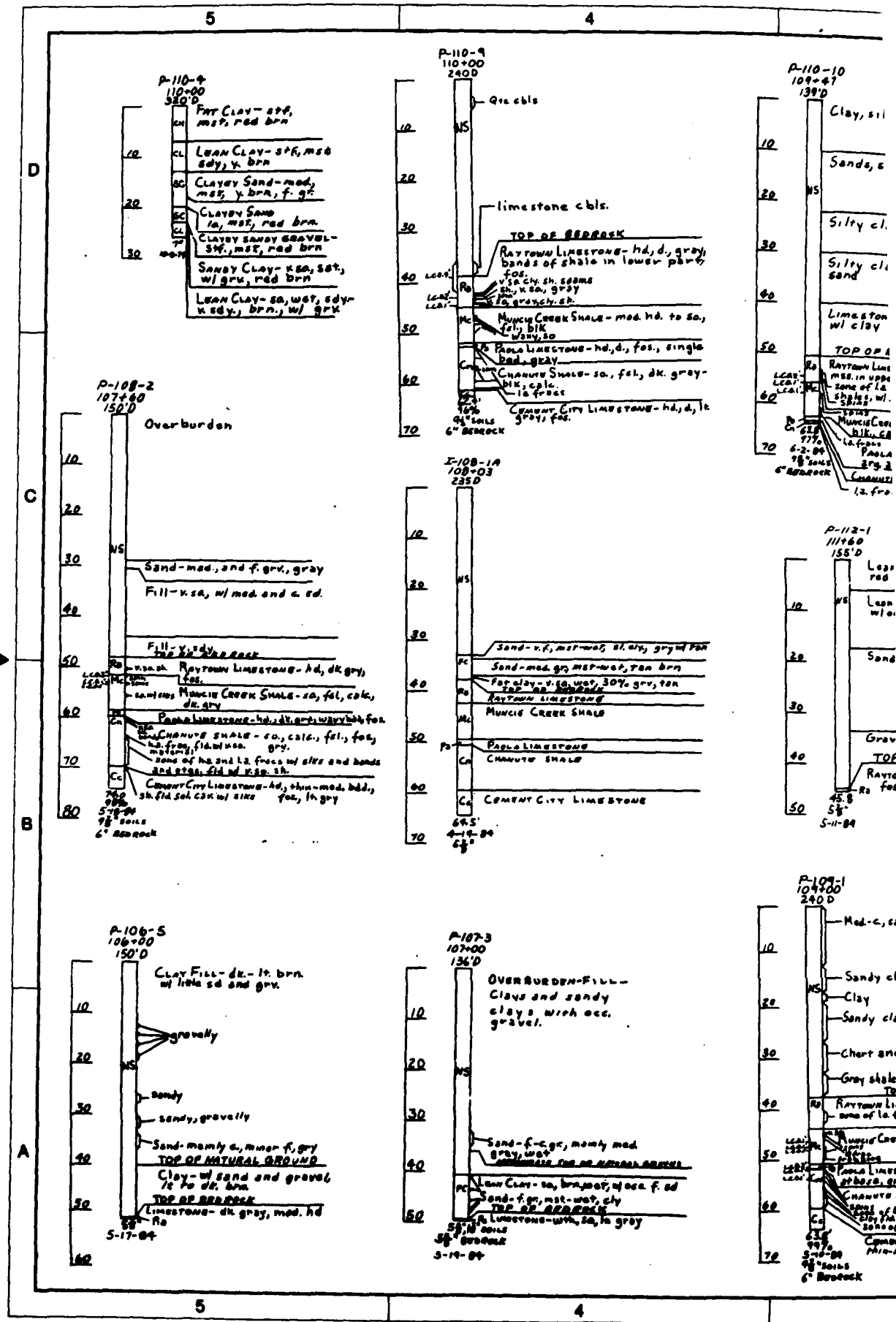
CORPS OF ENGINEERS U. S. ARMY
KANSAS CITY DISTRICT

FILE NO. RP-3-1007 1677

AUGUST 1901

APPENDIX A

CONSTRUCTION FOUNDATION REPORT PLATE 6



5-1-7

3

2

1

silt, sand, gravel

s, silt, clay

clay

clay w/ some fine

stone and chert grn

TOP OF BEDROCK

Limestone - hd, d-film, upper, sh. lower, gray
f. la. frags, mst. frags, n. sa, ch.
w/ some brn. ls. frags.Creek Shale - sa, fol, sl. calc.,
f. 6-7-84Hole Limestone - hd, f. film, gray,
f. at base, single bed
Mudstone - sa, fol, blk.
f. 6-8-84Lean Clay - med, mst, sry,
red brn w/ f. sd.Lean Clay - med, sry, tan, w/ f.-c. sd,
w/ occ. f. grv.

Sandy Silt - sa, tan-y, f. gr. sd.

Gravel - f.-c, 95% ls, sdy, sry, ch

TOP OF BEDROCK

Raytown Limestone - med hd, d-film,
f. 6-8-84

sand w/ clay

clay

clay

sand limestone bldrs

key clay

TOP OF BEDROCK

Limestone - hd, d, f. film, med, f. 6-8-84
f. frags and n. sa. gray sh. bands

Mudstone - sa, fol, blk, calc, waxy

Limestone - hd, f. film, gray, sry.

Mudstone - sa, fol, blk, calc, waxy

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

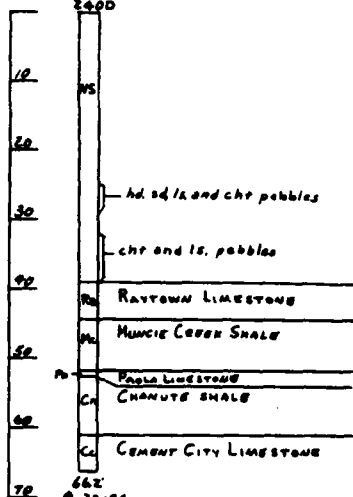
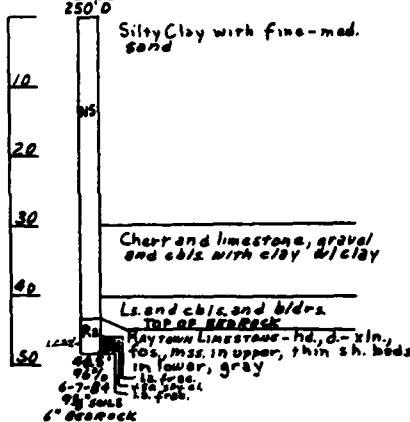
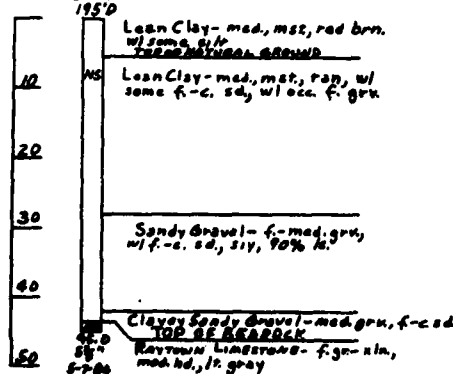
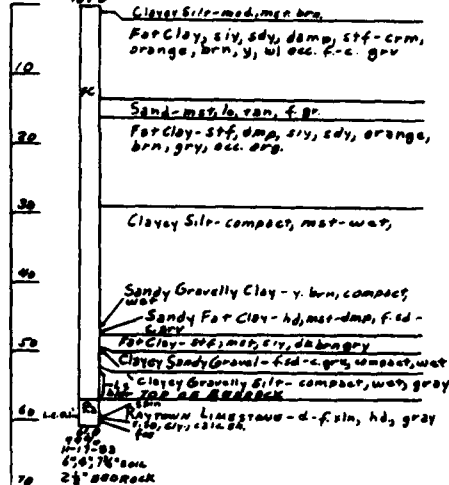
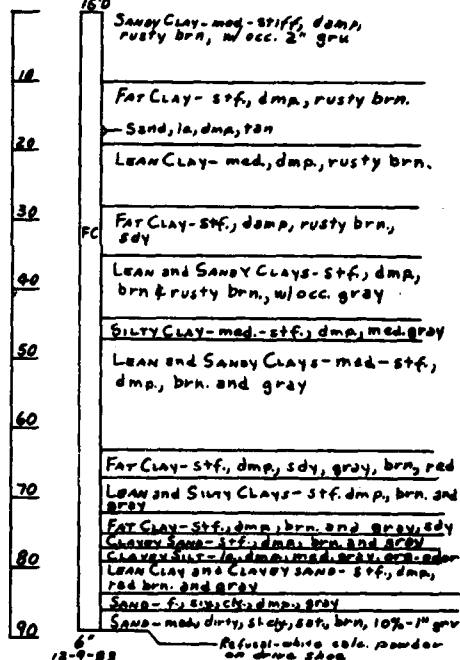
Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

Limestone - sa - n. sa, d. gray, calc.

I-110-1
110+00
24.00P-112-2
112+00
250.0P-113-1
113+00
195.0P-110-8
110+00
100.0P-106-4
106+02
16.0

Revisions			
Symbol	Description	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: [Signature]

Drawn by: [Signature]

Checked by: [Signature]

Submitted by: [Signature]

Book:

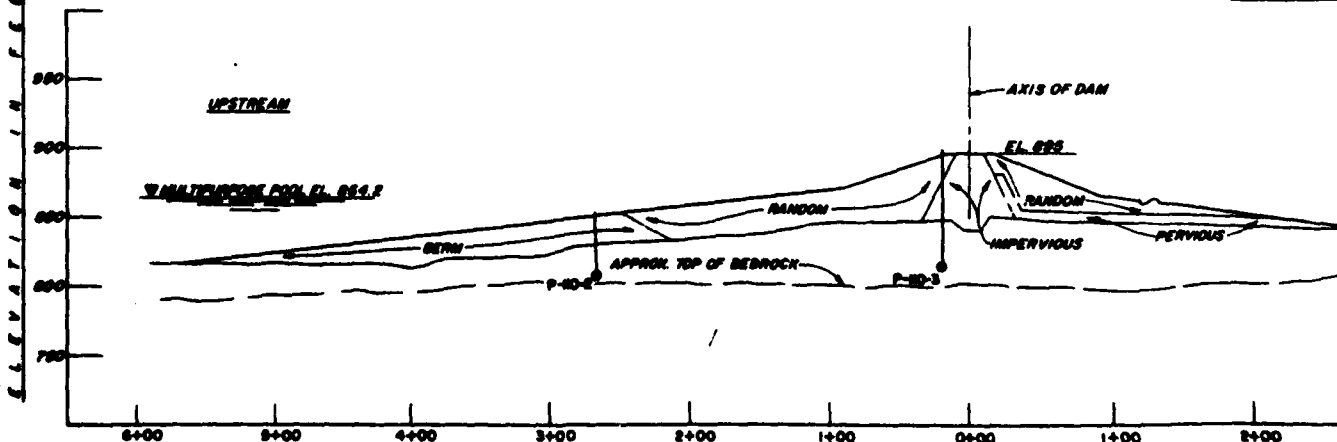
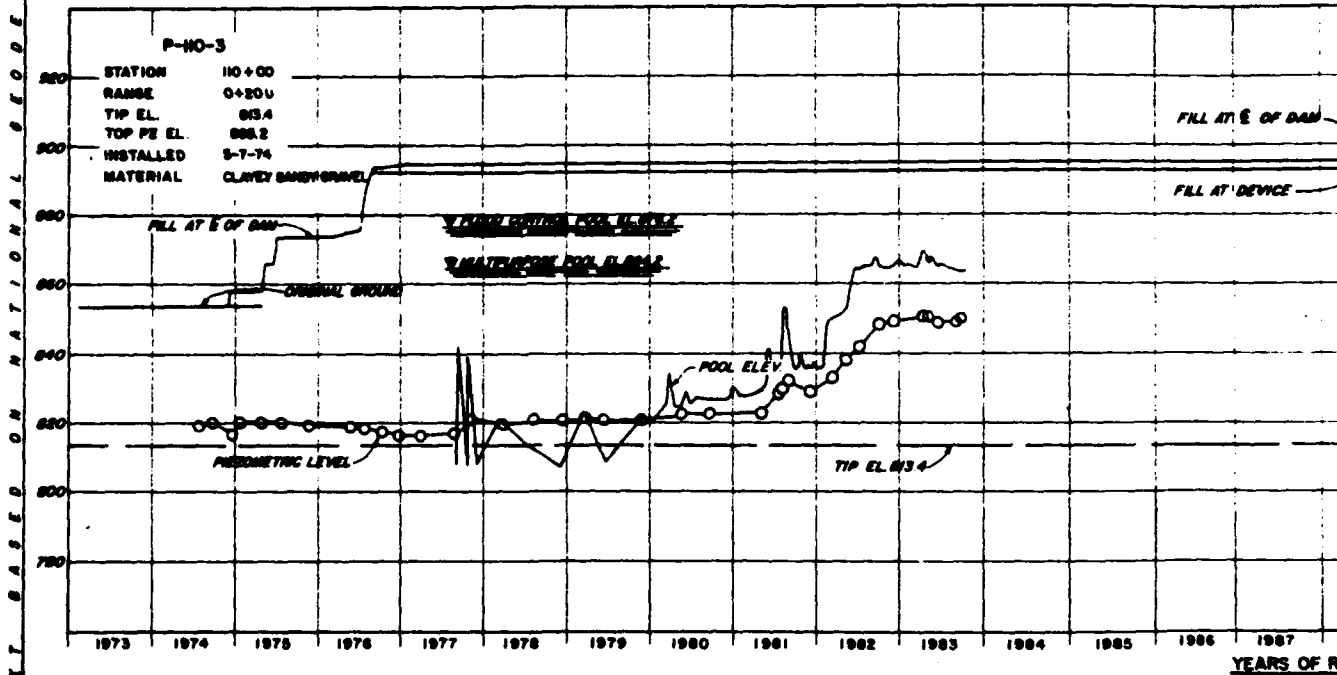
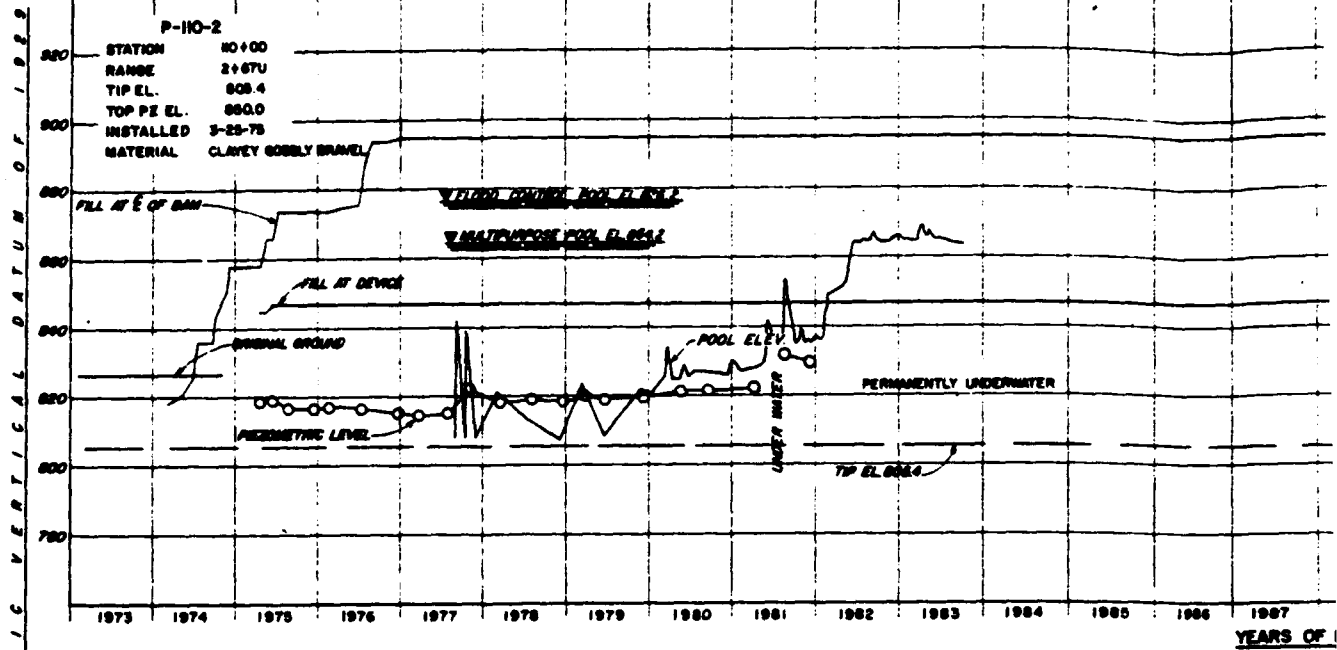
Date:

Sheet No.:

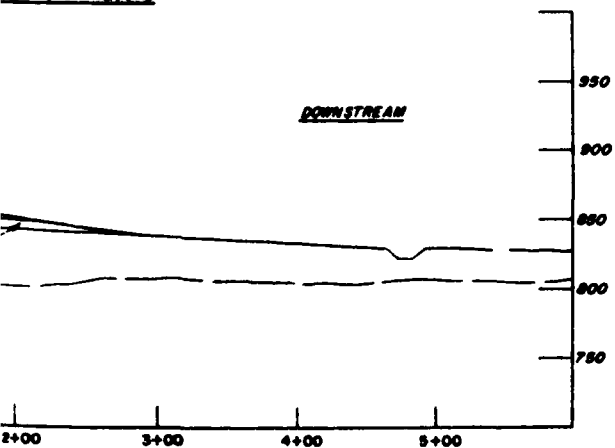
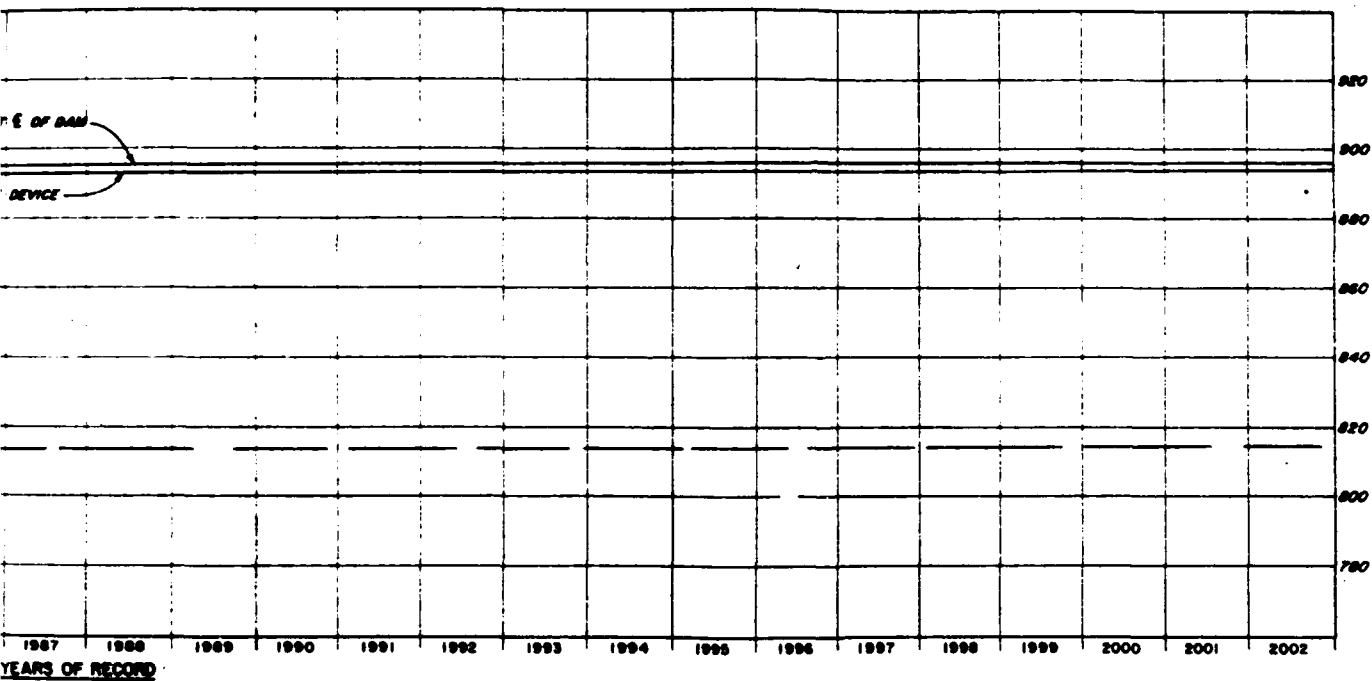
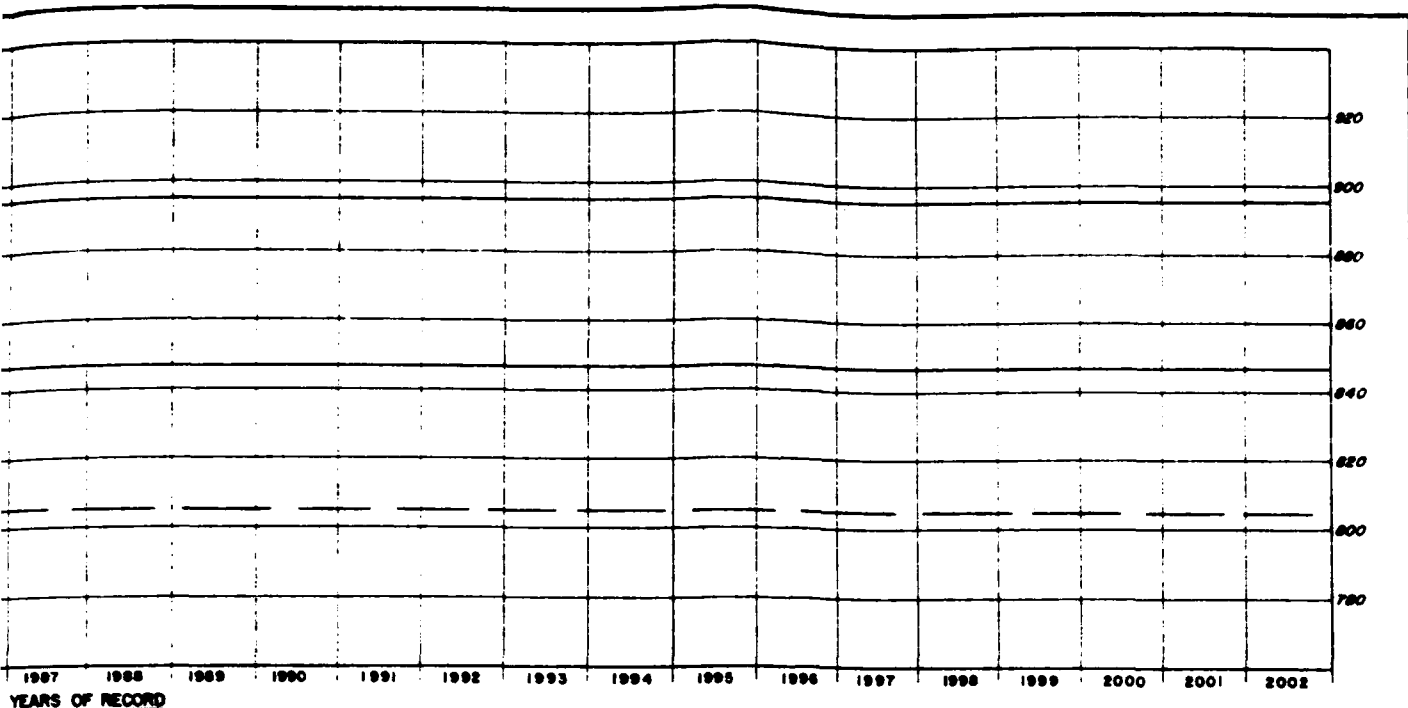
SMITHVILLE LAKE
LOGS OF EXPLORATIONS

RP-3-1678

APPENDIX A PLATE NO. 9



15-1-7



Revised September 1982
 LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
 PERIODIC INSPECTION
 PIEZOMETRIC LEVELS
 P-110-2 (OPEN TUBE)
 P-110-3 (OPEN TUBE)

In 1 sheet

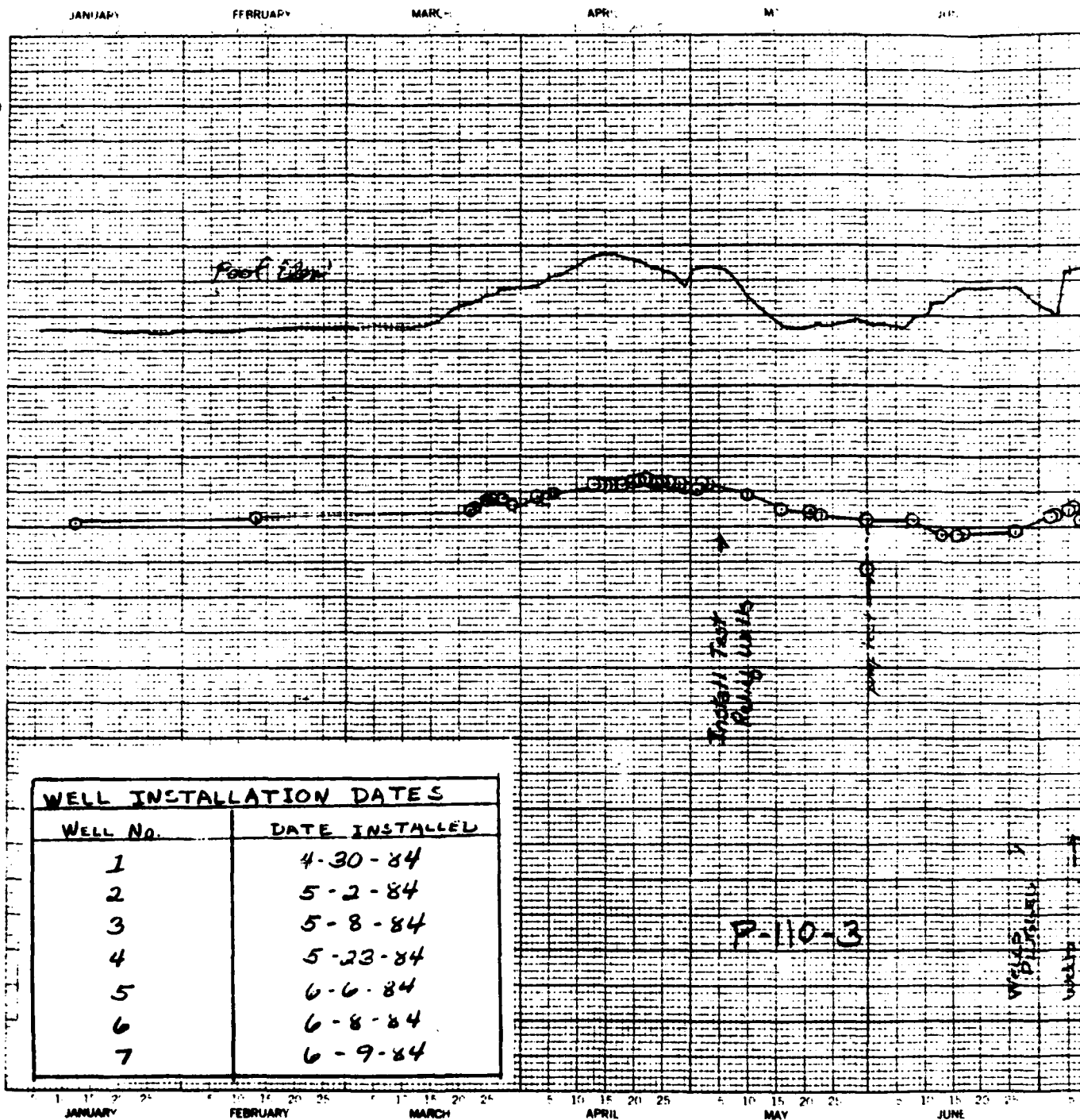
Sheet No. 1

Scale: as shown

CORPS OF ENGINEERS U. S. ARMY
 KANSAS CITY DISTRICT
 FILE NO. RP-3-1680
 MARCH 1981

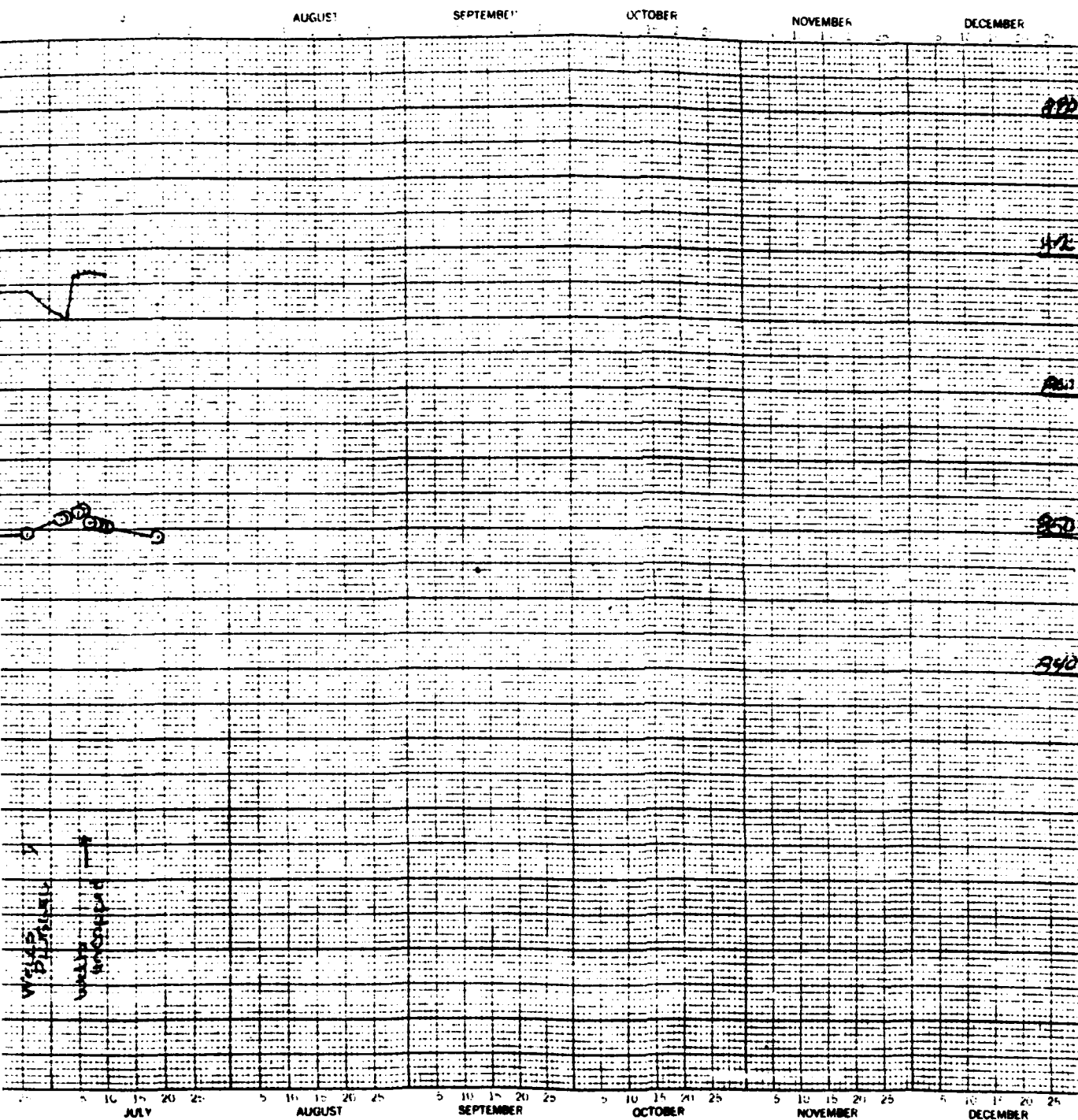
47 2813

K-E 1 YEAR 81 DAYS X 100 DIVISIONS
K-E 1 YEAR 81 DAYS X 100 DIVISIONS



P-110-3

15-1-7



Sta 110+00
Rg 0+20 US

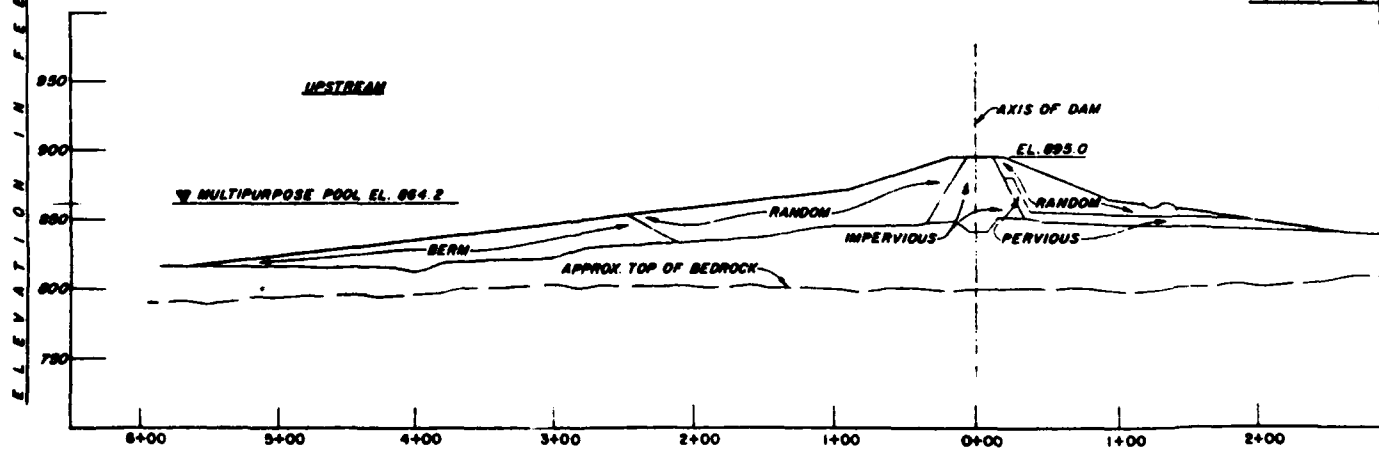
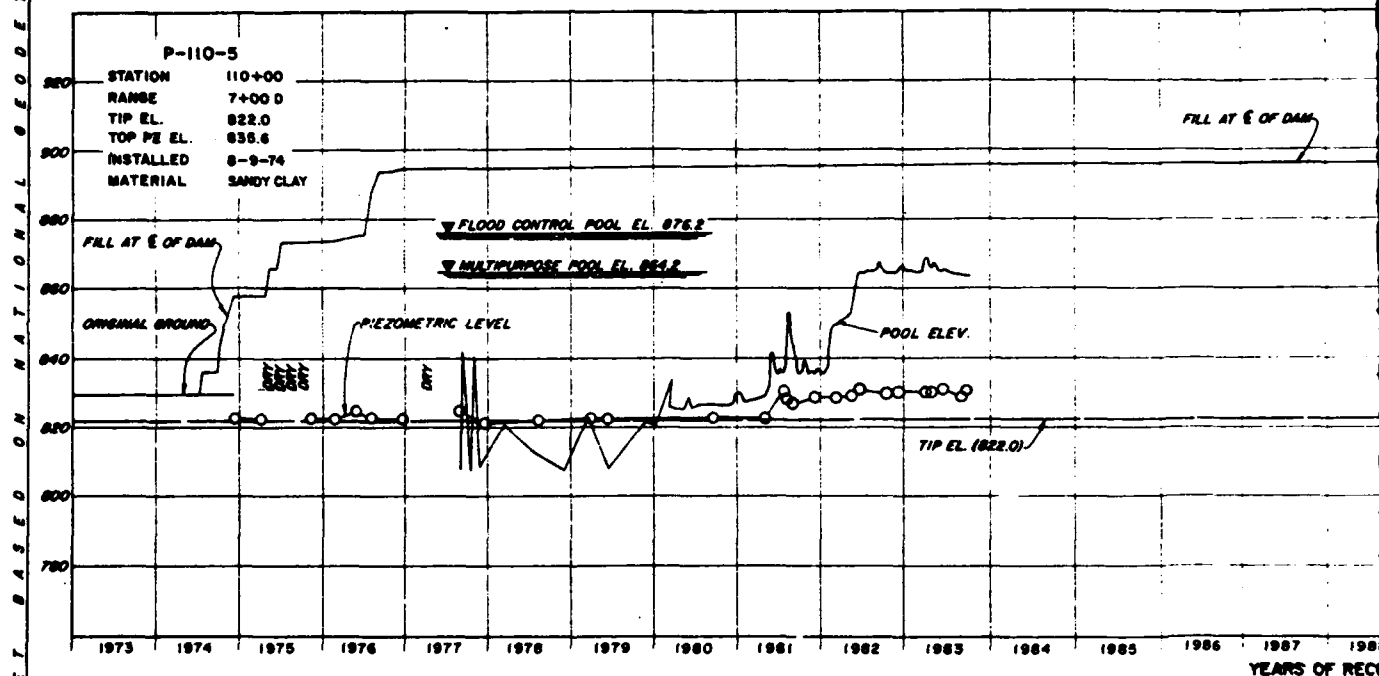
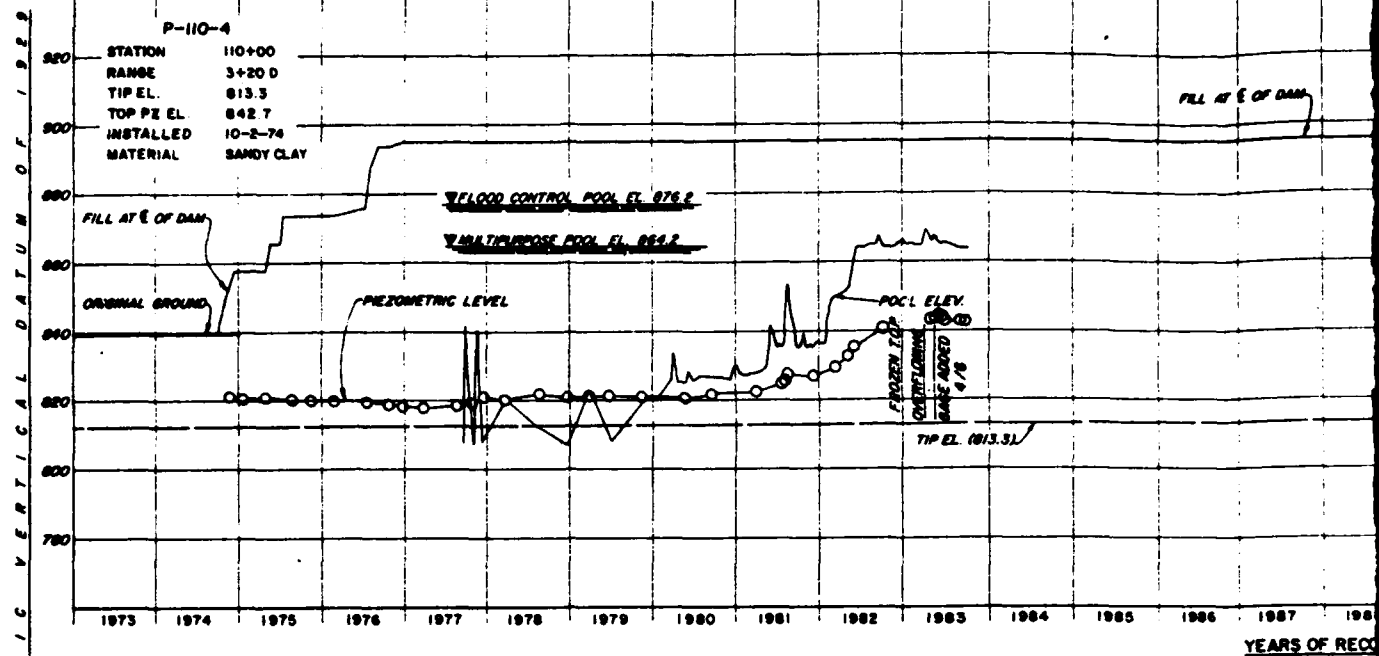
Smithville Dam

P-110-3

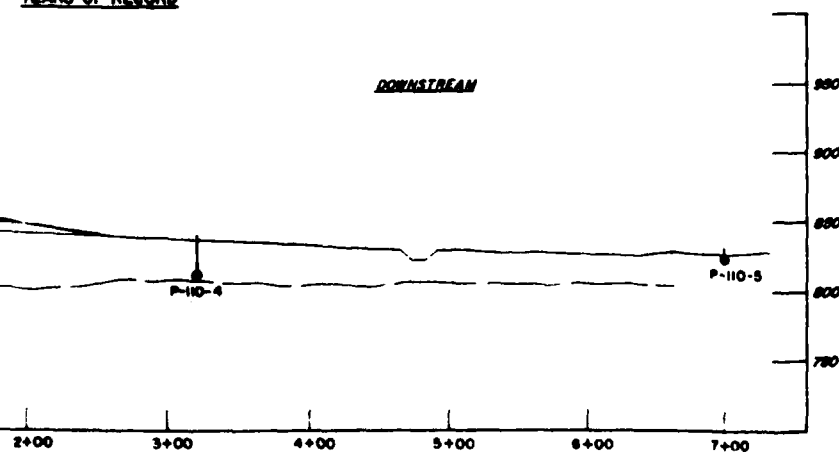
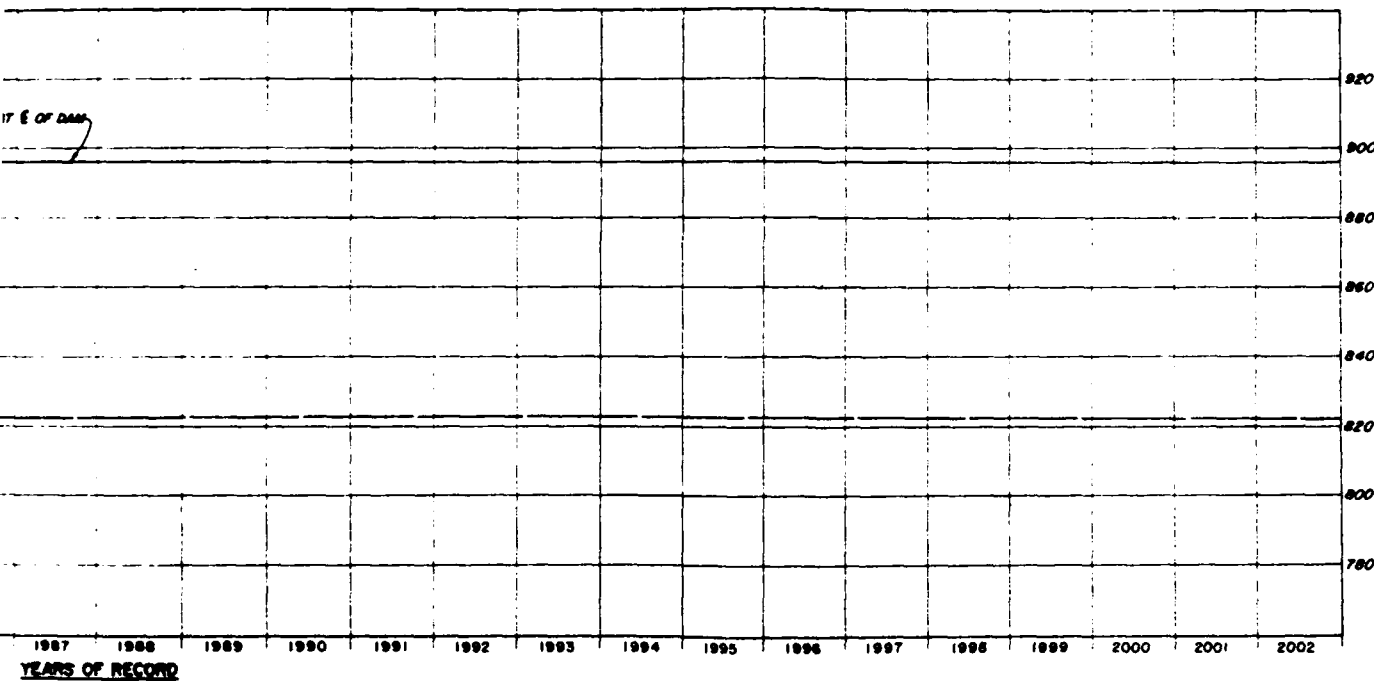
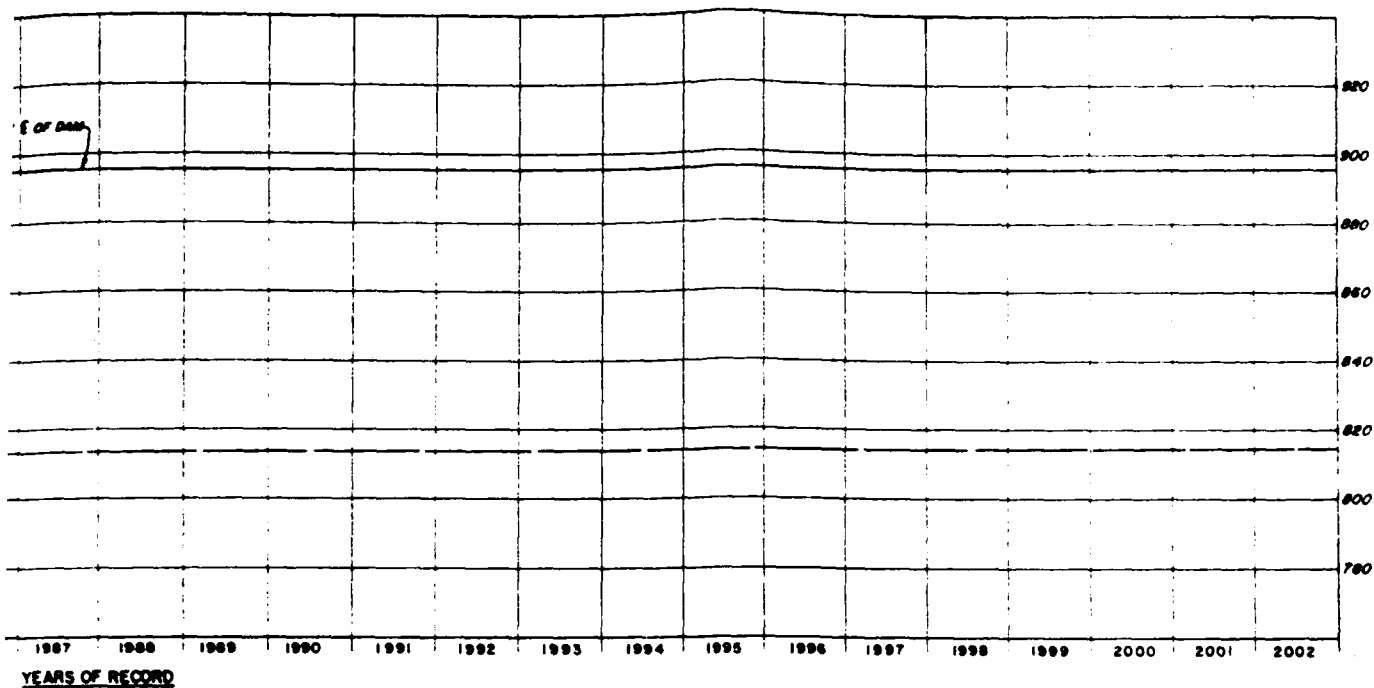
1984 PLOT

RP-3-1681

APPENDIX A PLATE 9A



5-1-7



Revised September 1982
LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
PERIODIC INSPECTION
PIEZOMETRIC LEVELS
P-110-4 (OPEN TUBE)
P-110-5 (OPEN TUBE)

In 1 sheet

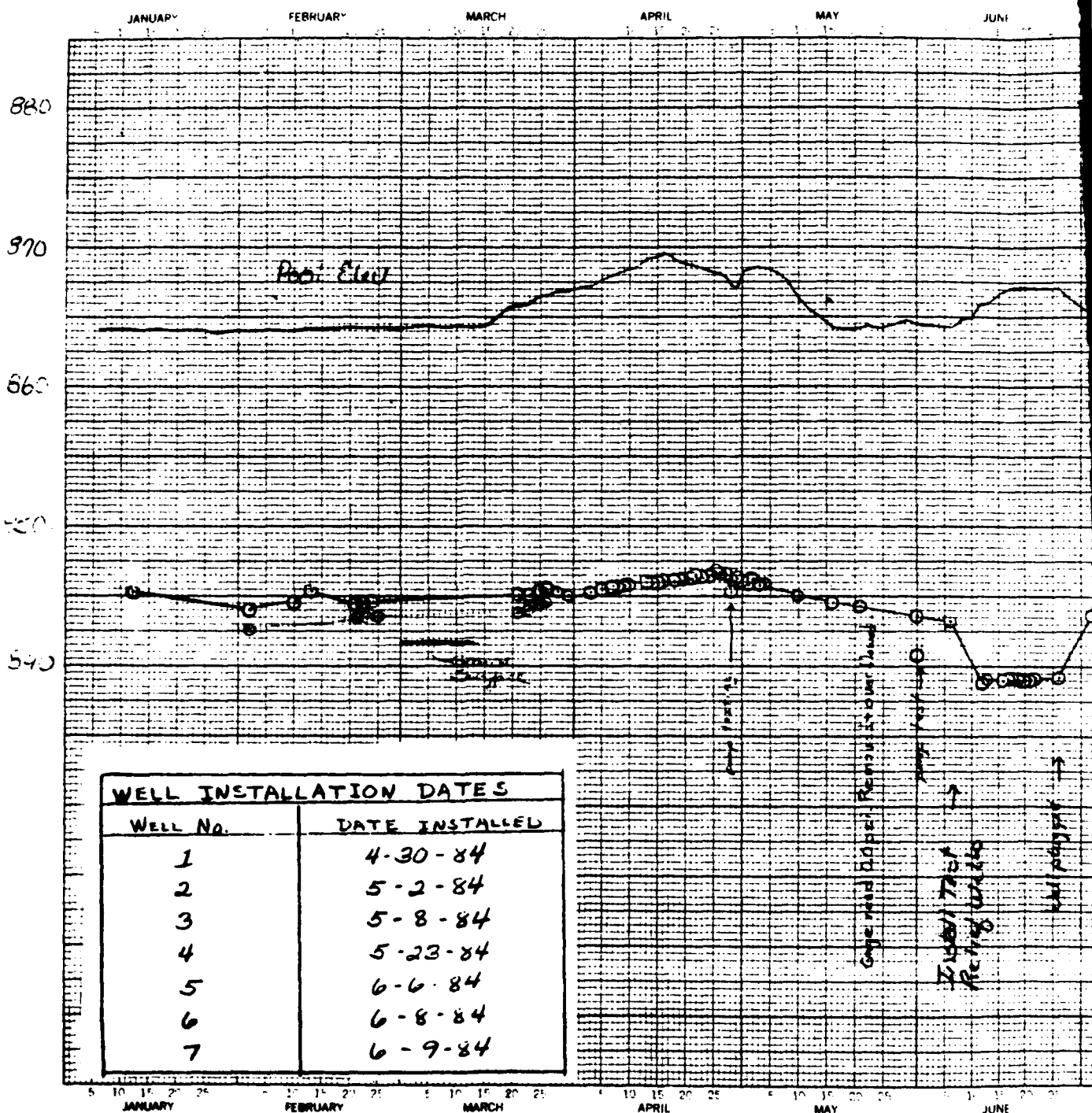
Sheet No. 1
CORPS OF ENGINEERS U. S. ARMY
KANSAS CITY DISTRICT
FILE NO RP-3-~~1682~~ 1682
MARCH 1981

Scale: as shown

APPENDIX A PLATE NO. 10

47 2813

K-E 1 YEAR BY DAYS X 150 DIVISIONS
KEUFFEL & ESSER CO. NEW YORK, N.Y.



P-110-4

15-1-7

JULY

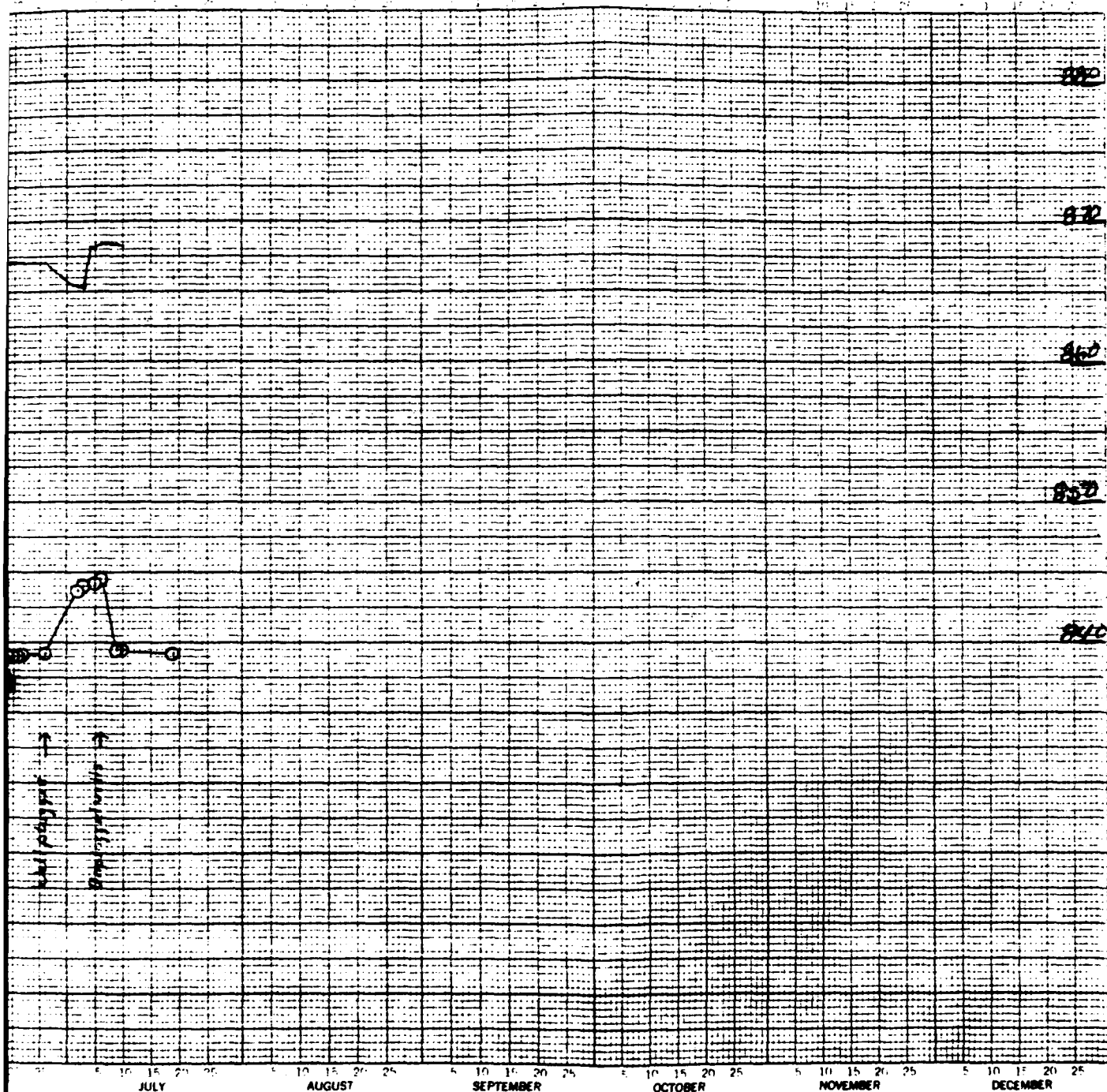
AUGUST

SEPTEMBER

OCTOBER

NOVEMBER

DECEMBER

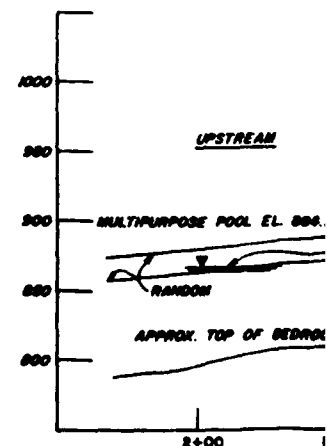
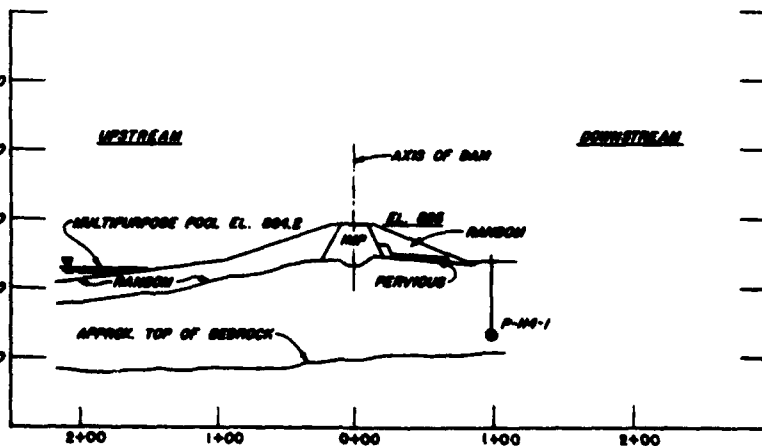
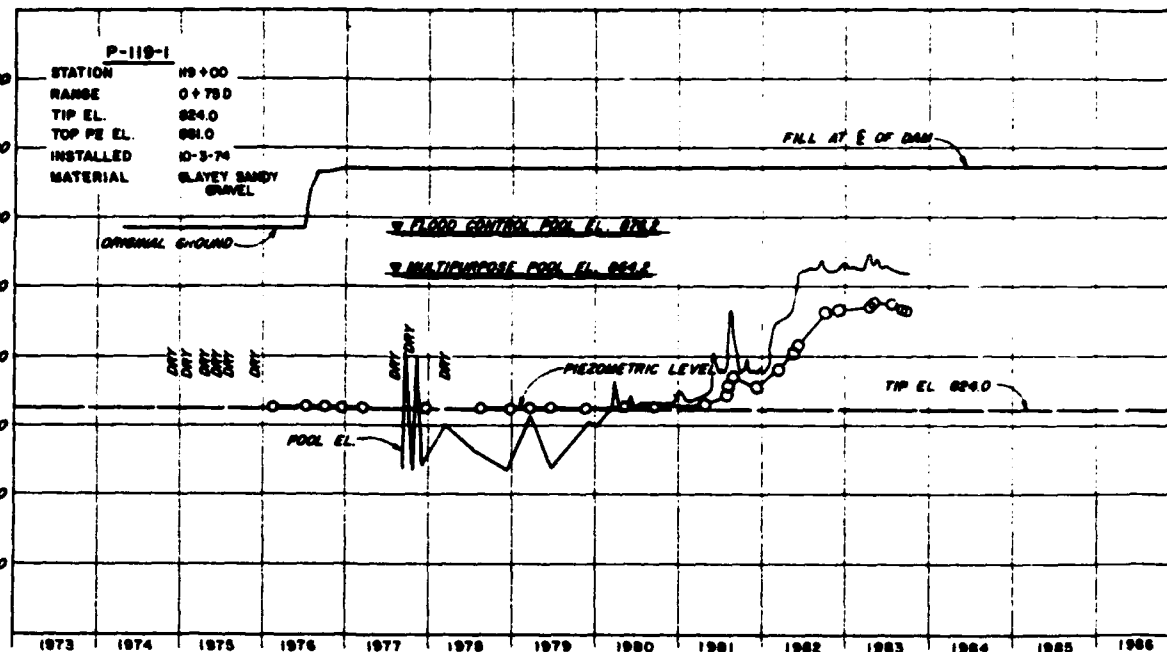
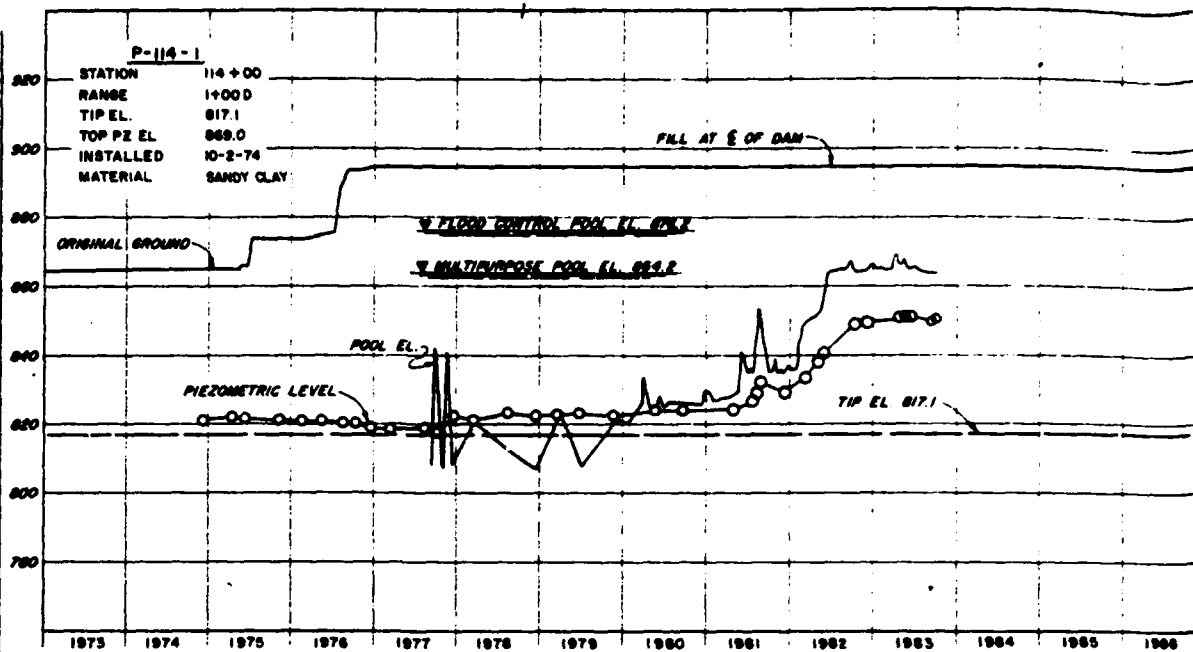


Sta 110+00
R 3+20 DIS

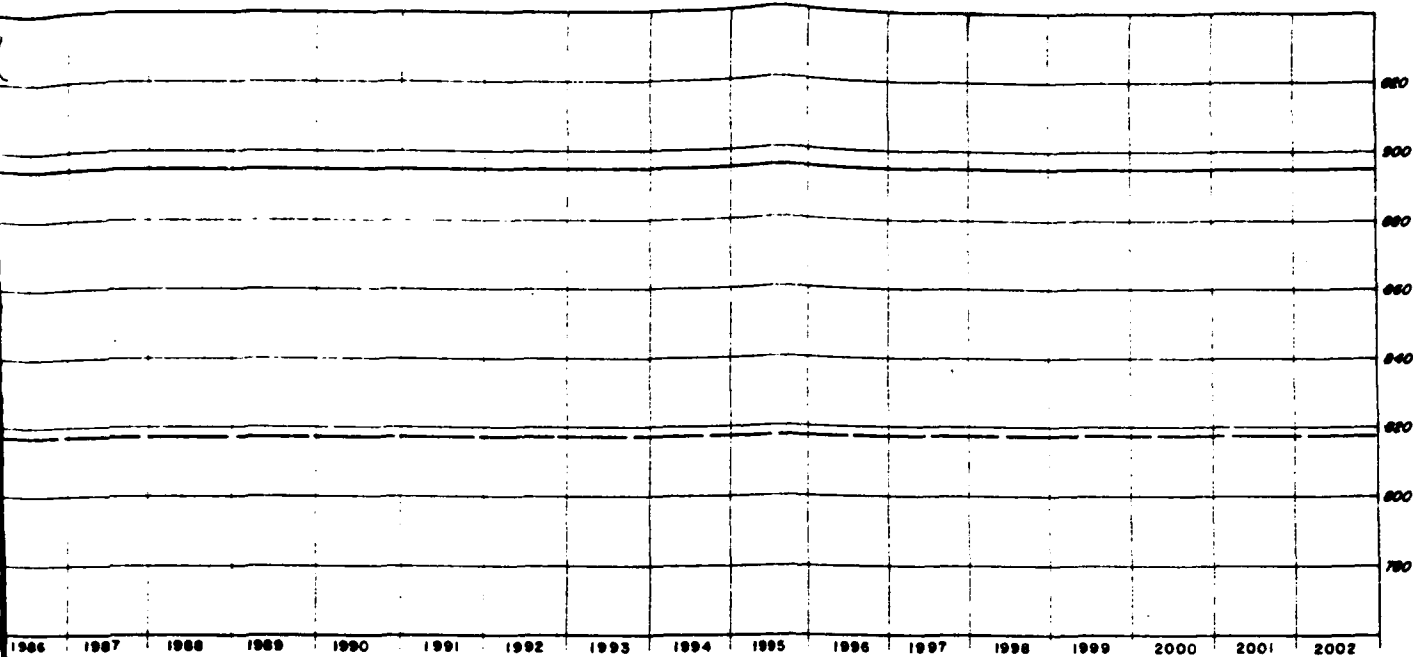
O Reading: changed to correct w/ 5 ft.
gauge. 15# gauge and diesel removed
2-27-84. Replaced w/ 5# Marshall + wr
gauge.
RP-3-1683

APPENDIX A DATE 10A

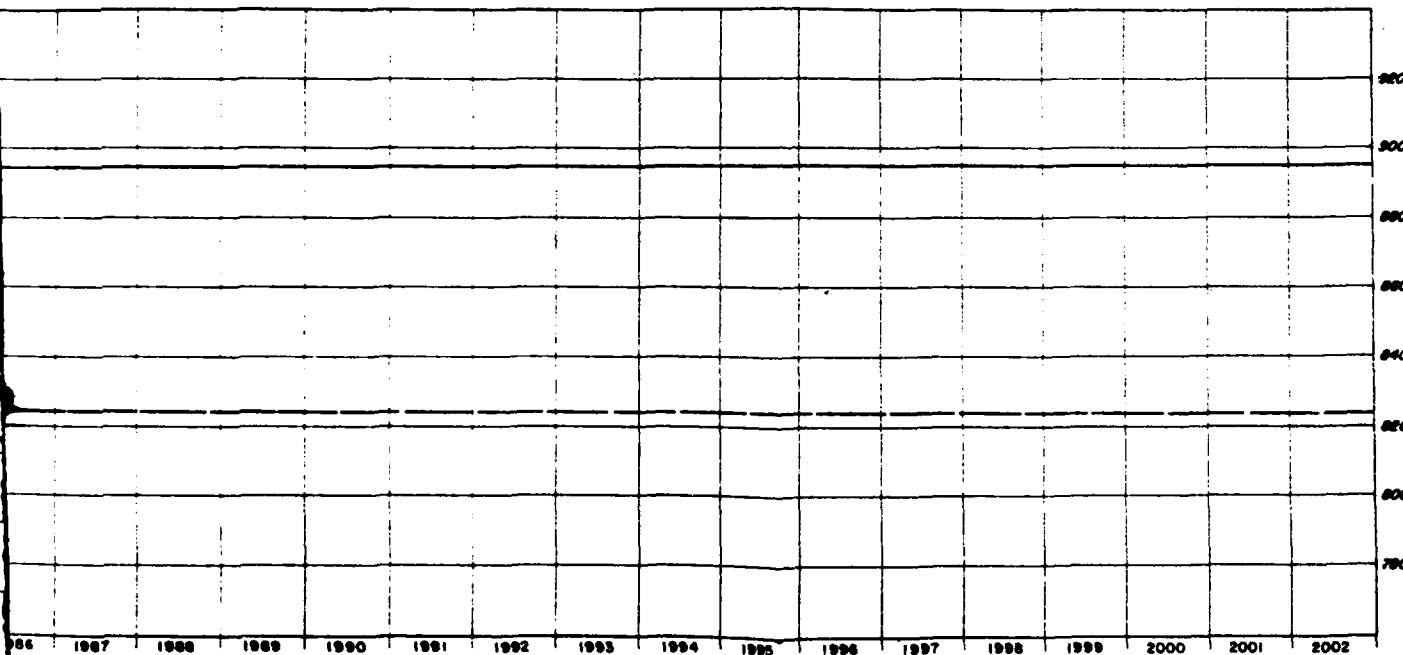
ELEVATION IN FEET BASED ON NATIONAL SECTETIC VERTICAL DATUM OF 1929



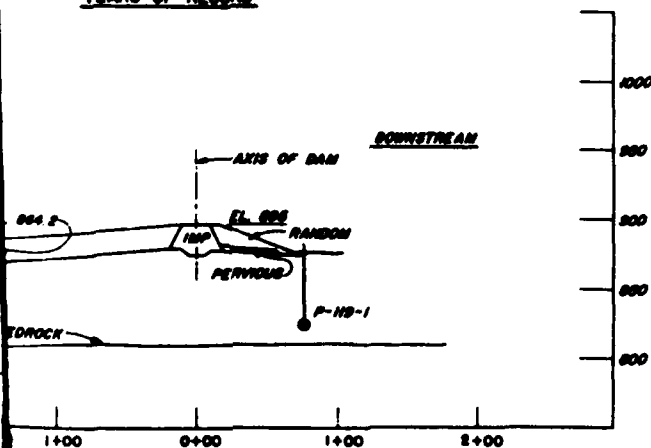
15-1-7



YEARS OF RECORD



YEARS OF RECORD



Revised September 1982
LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
PERIODIC INSPECTION

PIEZOMETRIC LEVELS
P-114-1 (OPEN TUBE)
P-119-1 (OPEN TUBE)

In 1 sheet

Sheet No. 1

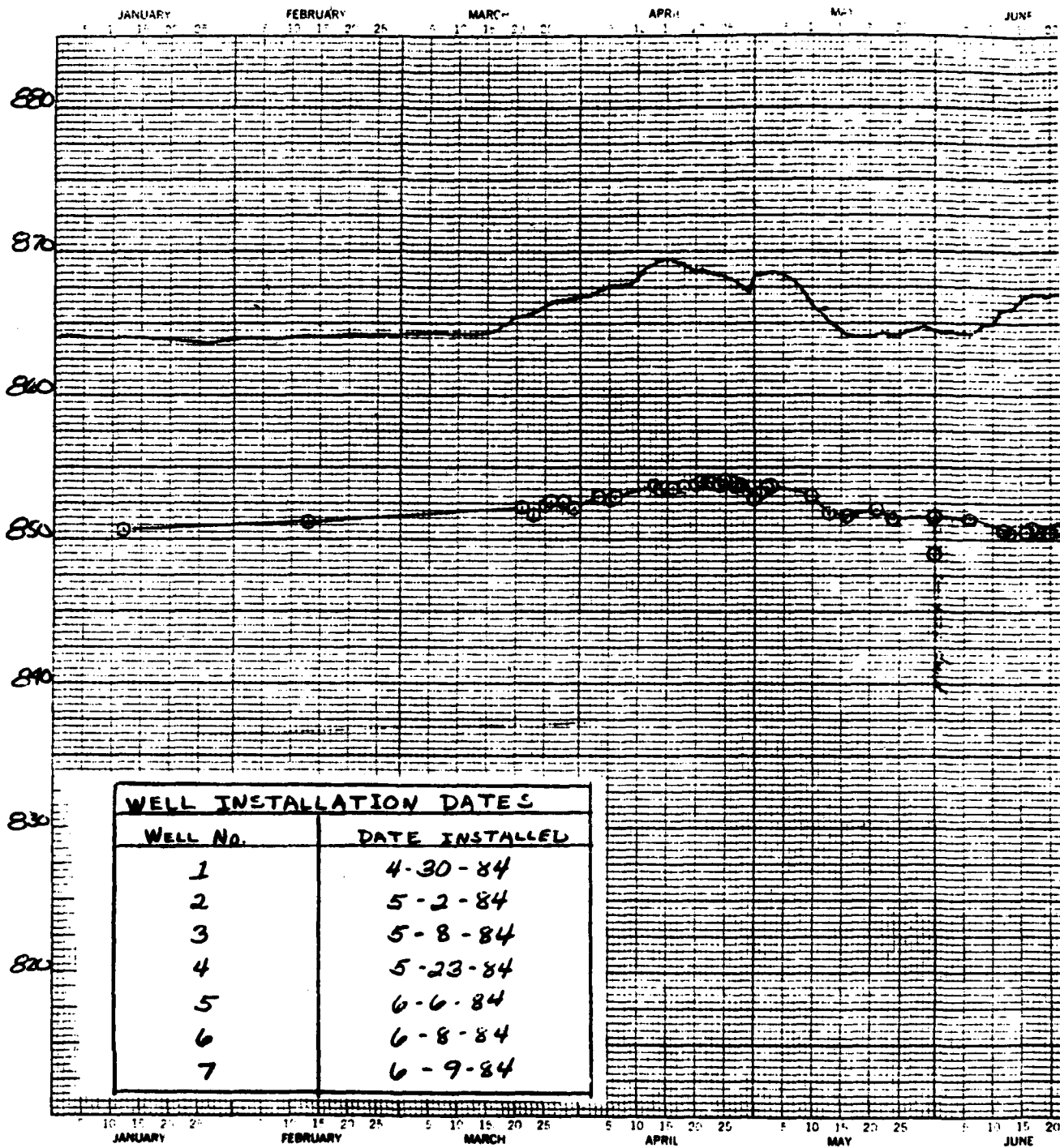
Scale: as shown

CORPS OF ENGINEERS U. S. ARMY
KANSAS CITY DISTRICT
FILE NO. RP-3-1684
MARCH 1981

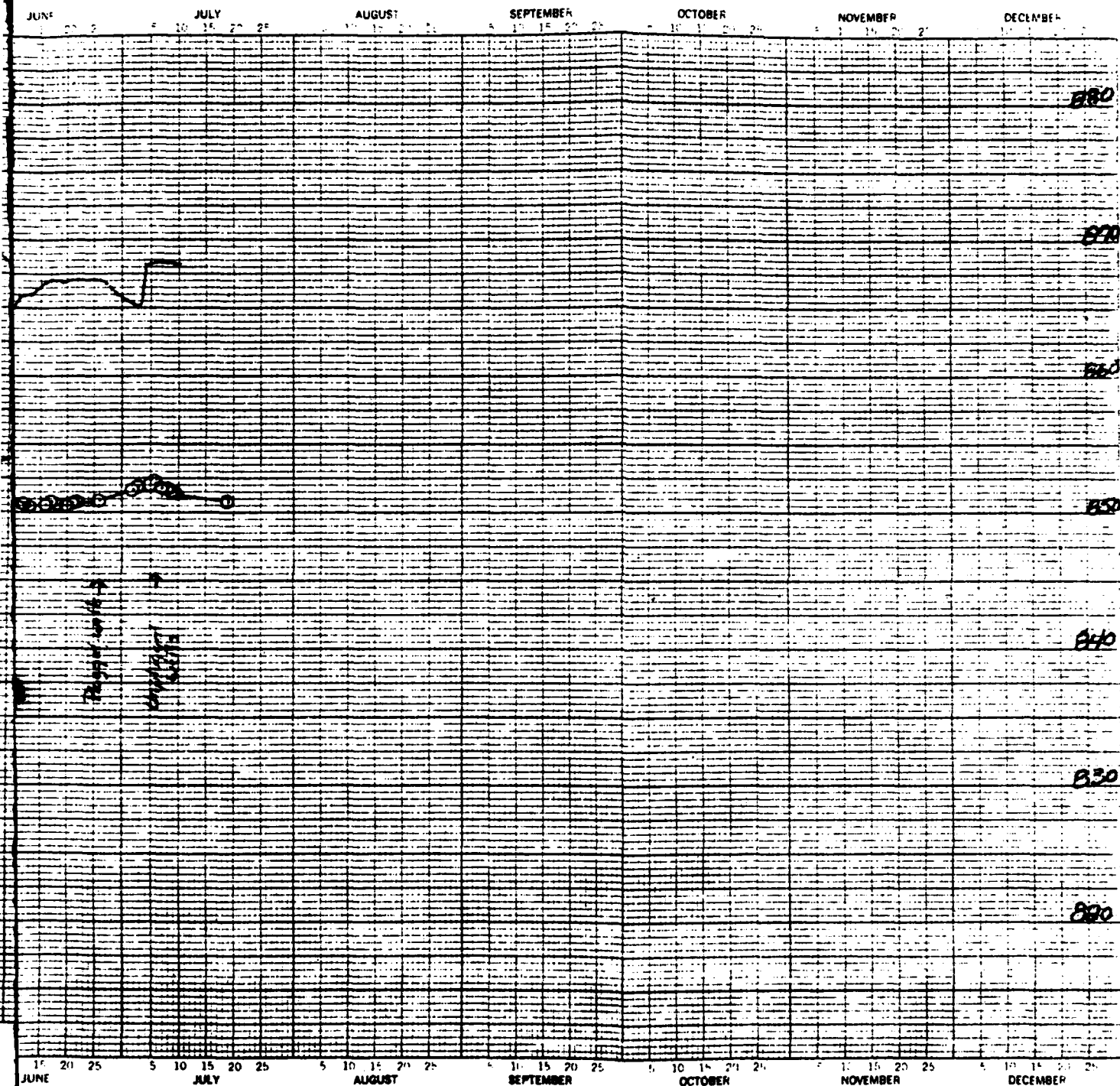
APPENDIX A PLATE NO. 11

47 2813

K-E 1 YEAR BY DAYS X 150 DIVISIONS
KEUFFEL & ESSER CO. NEW YORK, N.Y.



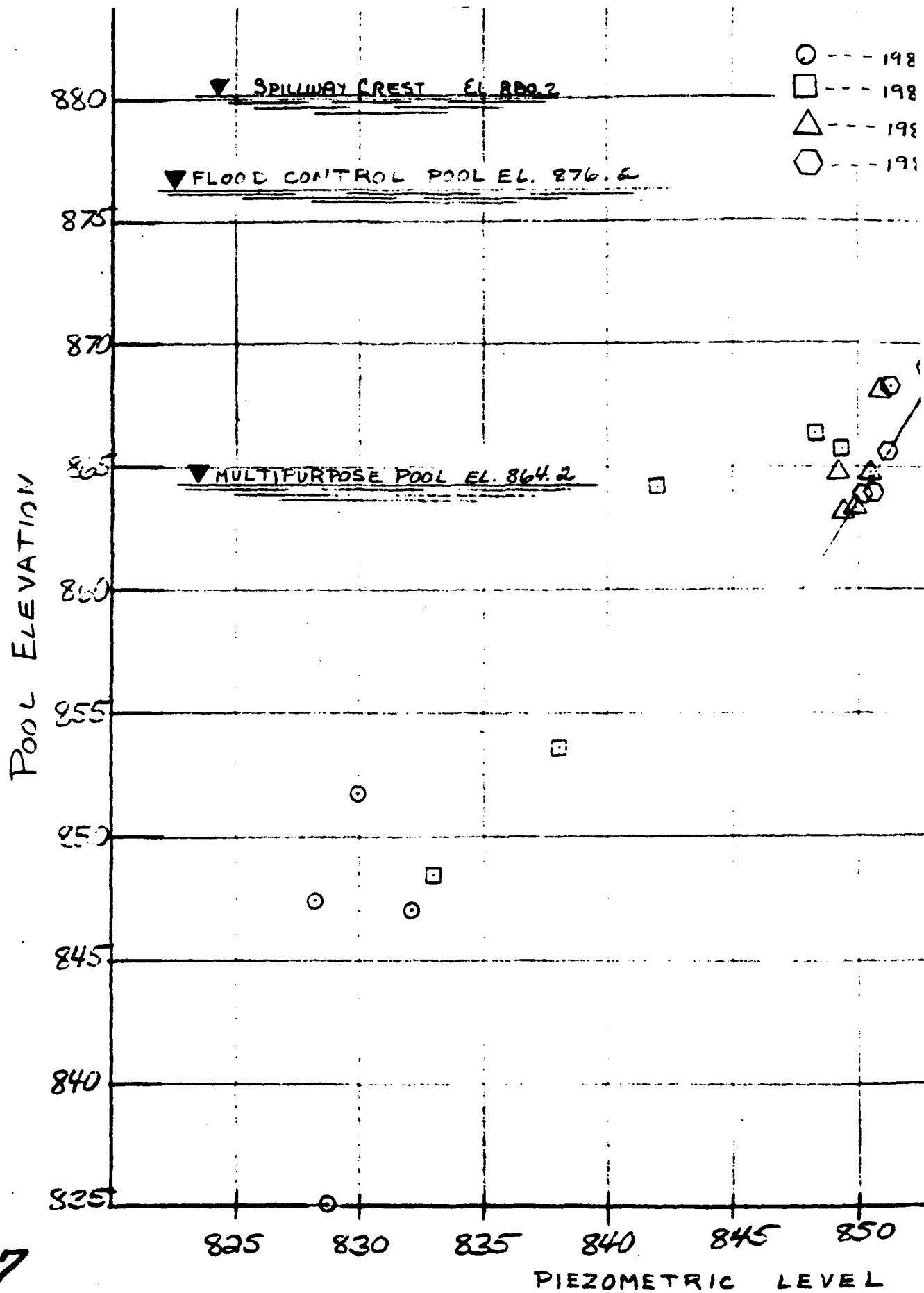
15-1-7



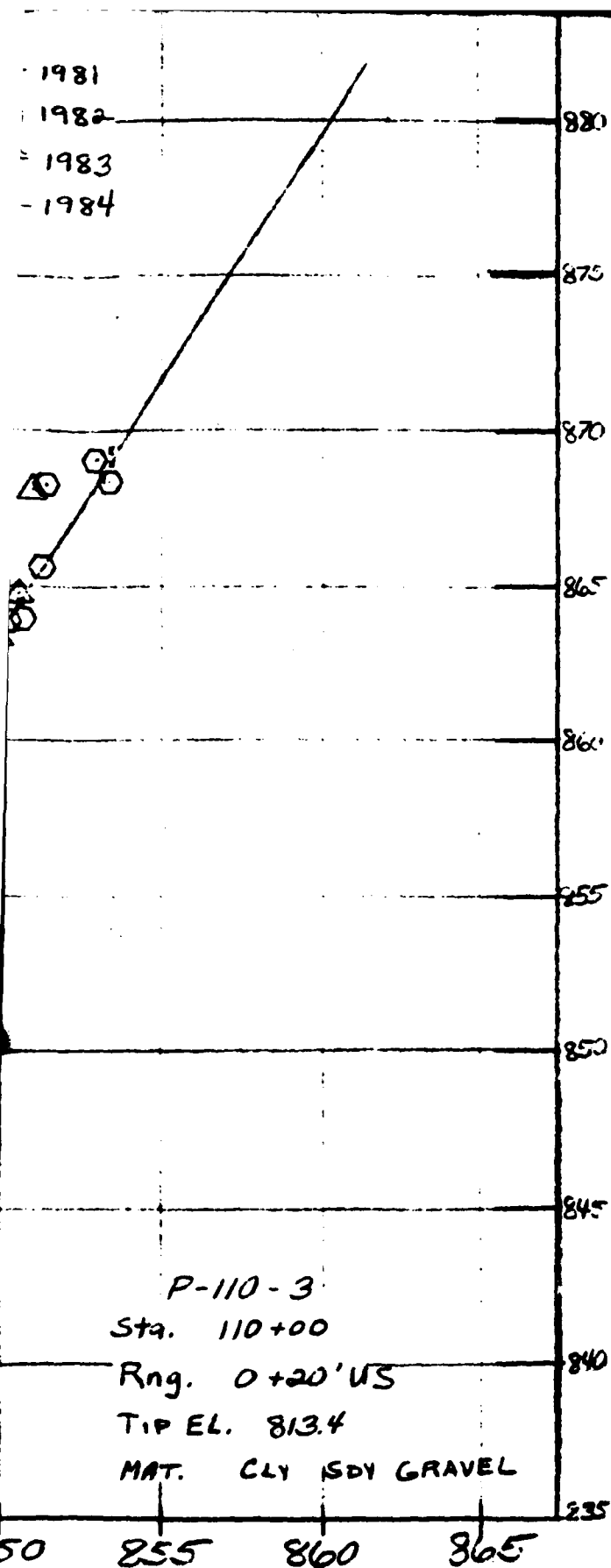
STA 114+00
K 1+00 D/S

SMITHVILLE OAM
P-114-1

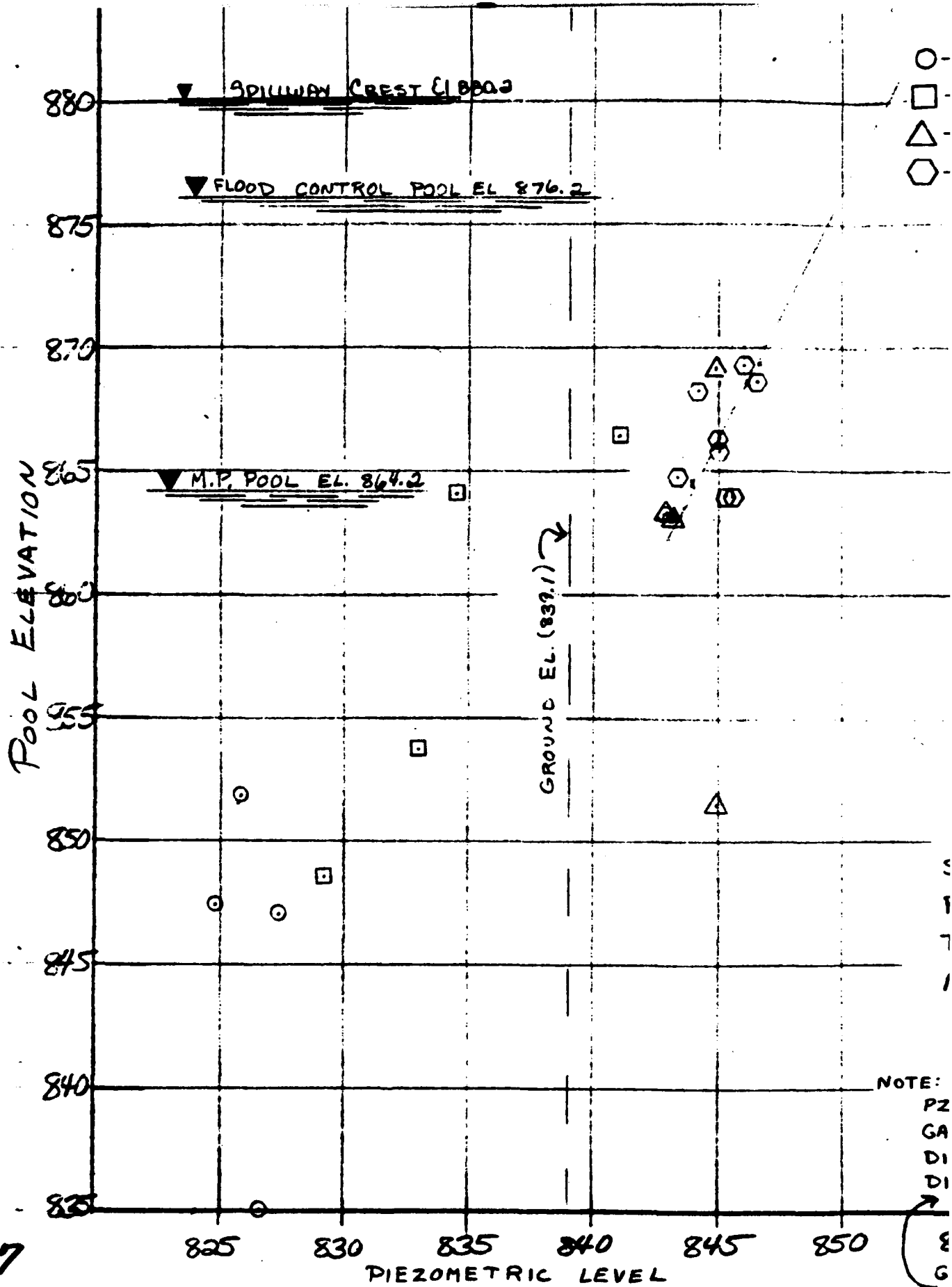
RP-3-1685
APPENDIX A PLATE 11A



15-1-7



SMITHVILLE DAM
LEFT ABUTMENT
Pz vs Pool Plot
P-110-3
APPENDIX A RP-3-1686
PLATE 12 JULY 1984



○ --- 1981
 □ --- 1982
 △ --- 1983
 ⬡ --- 1984

880

875

870

865

860

855

850

845

P-110-4

Sta. 110+00

Rng. 3+00 DS

TIP EL. 813.3

MAT. SANDY CLAY

NOTE: WATER FROZEN @ T.O.P. 12/22.

PZ OVERFLOWING 3/31/83.

GAGE ADDED 4/5/83.

DIESEL ADDED 11/83.

DIESEL REMOVED 3/27/84

SMITHVILLE DAM
 LEFT ABUTMENT

PZ VS POOL Plot
 P-110-4

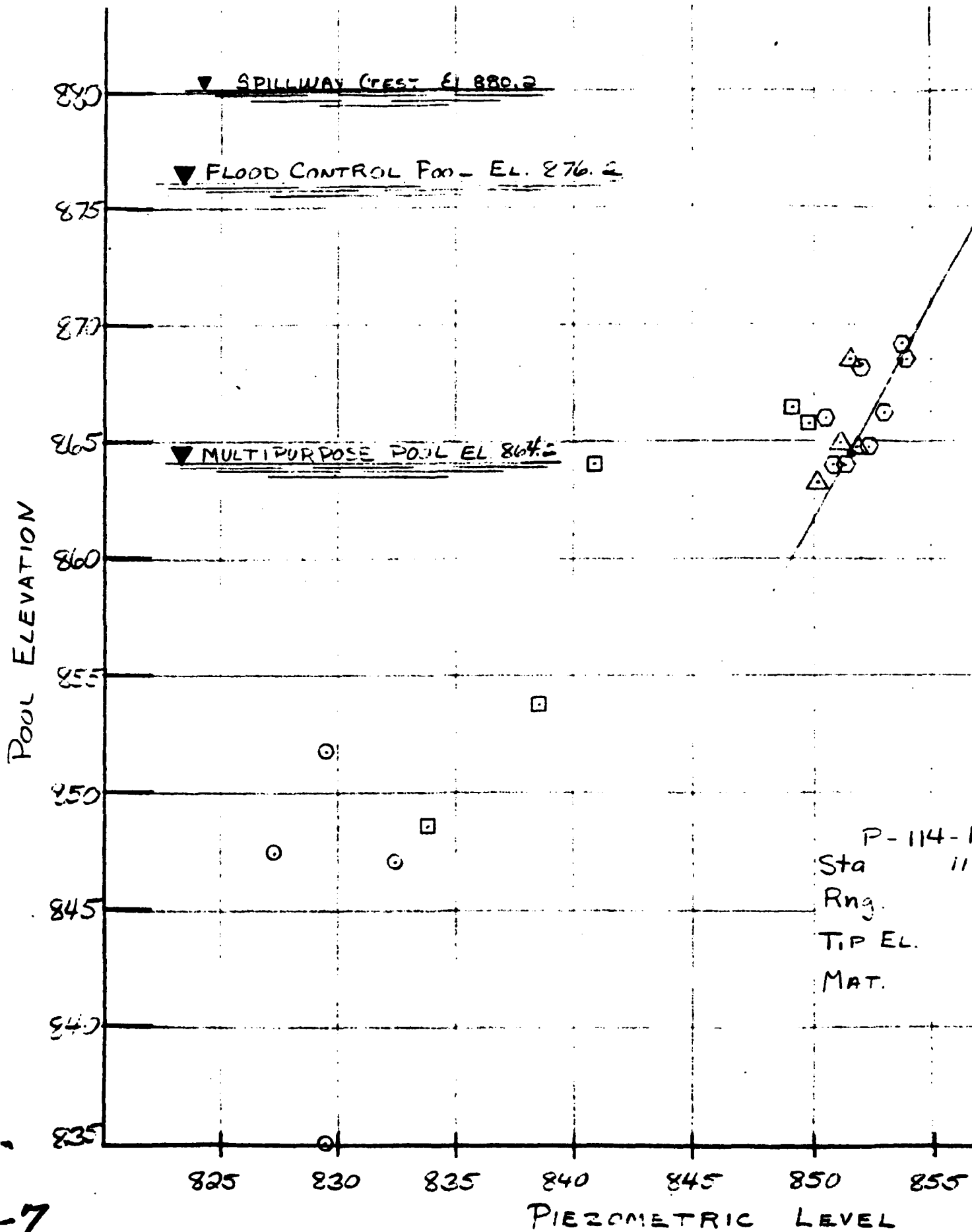
APPENDIX A RP-3-1687

PLATE 13

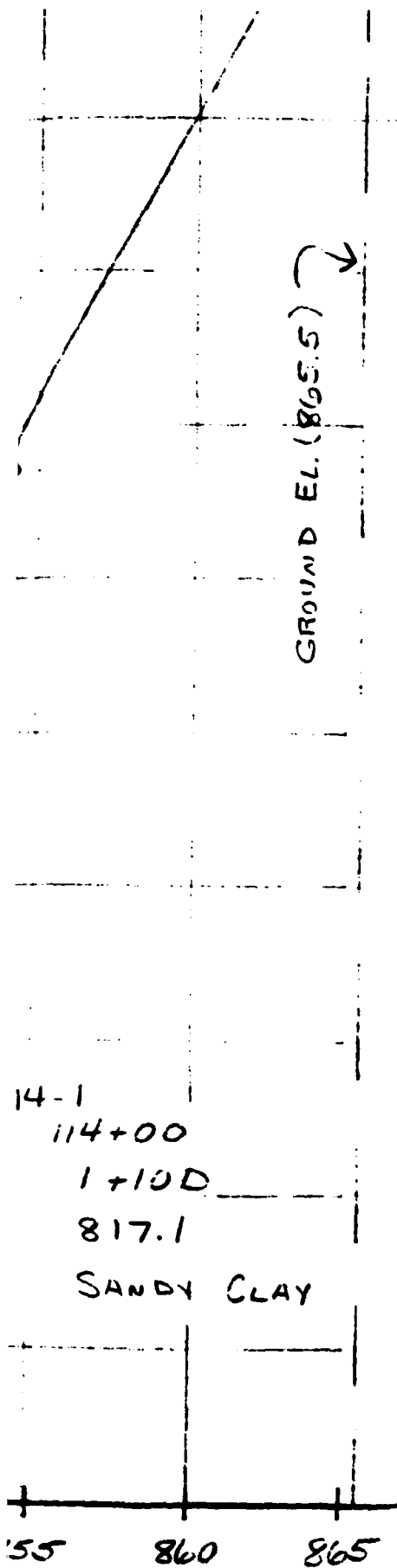
JULY 1984

855 860 865

GAGE REMOVED AND EXT.
 ADDED 5/31/84



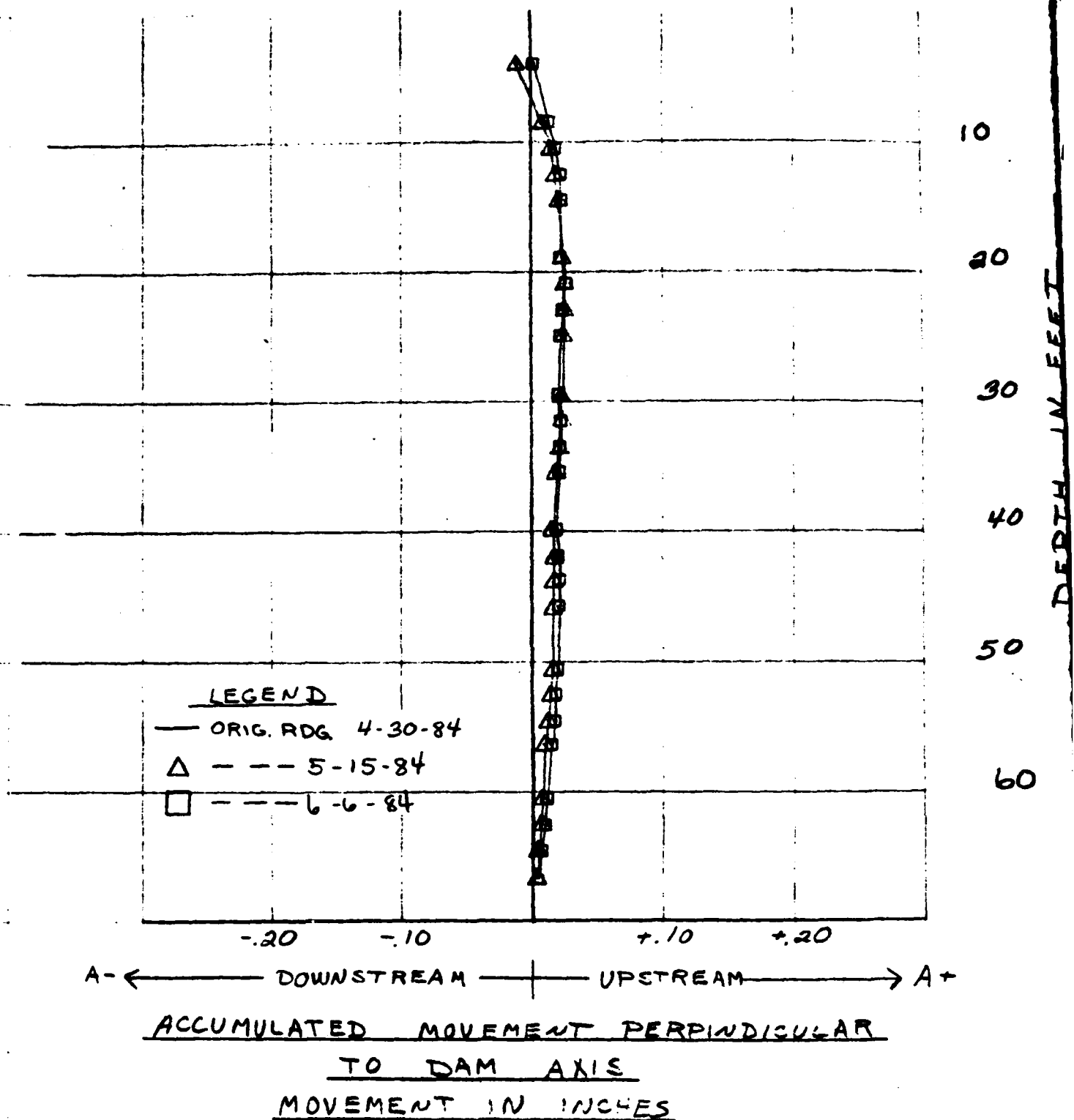
15-1-7



SMITHVILLE DAM
LEFT ABUTMENT

P2 vs Pool Plot
P-114-1

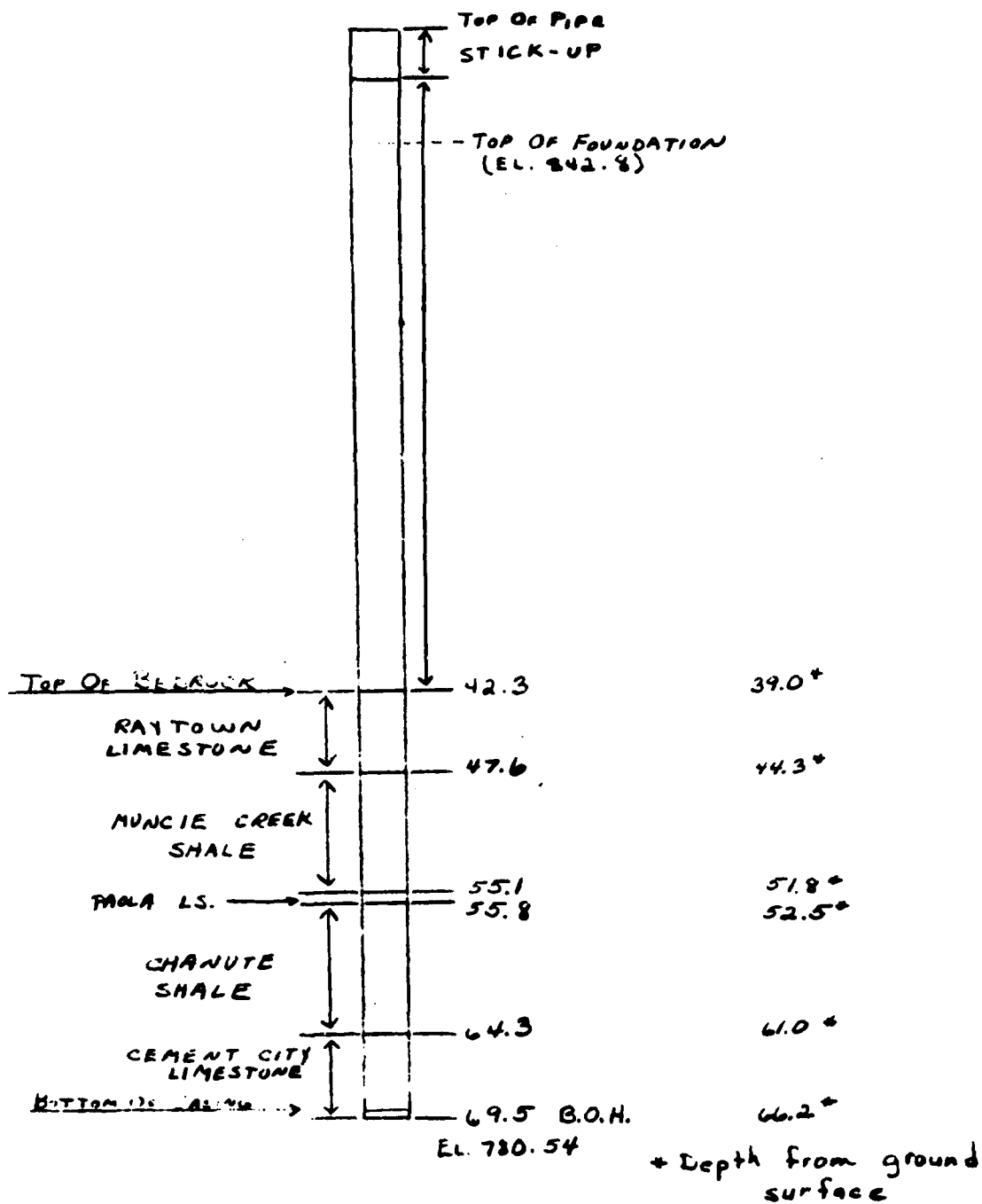
APPENDIX A RP-3-1688
PLATE 14 JULY 1984



I-110-1
Sta. 110+00
Rng. 2+40 DS

15-1-7

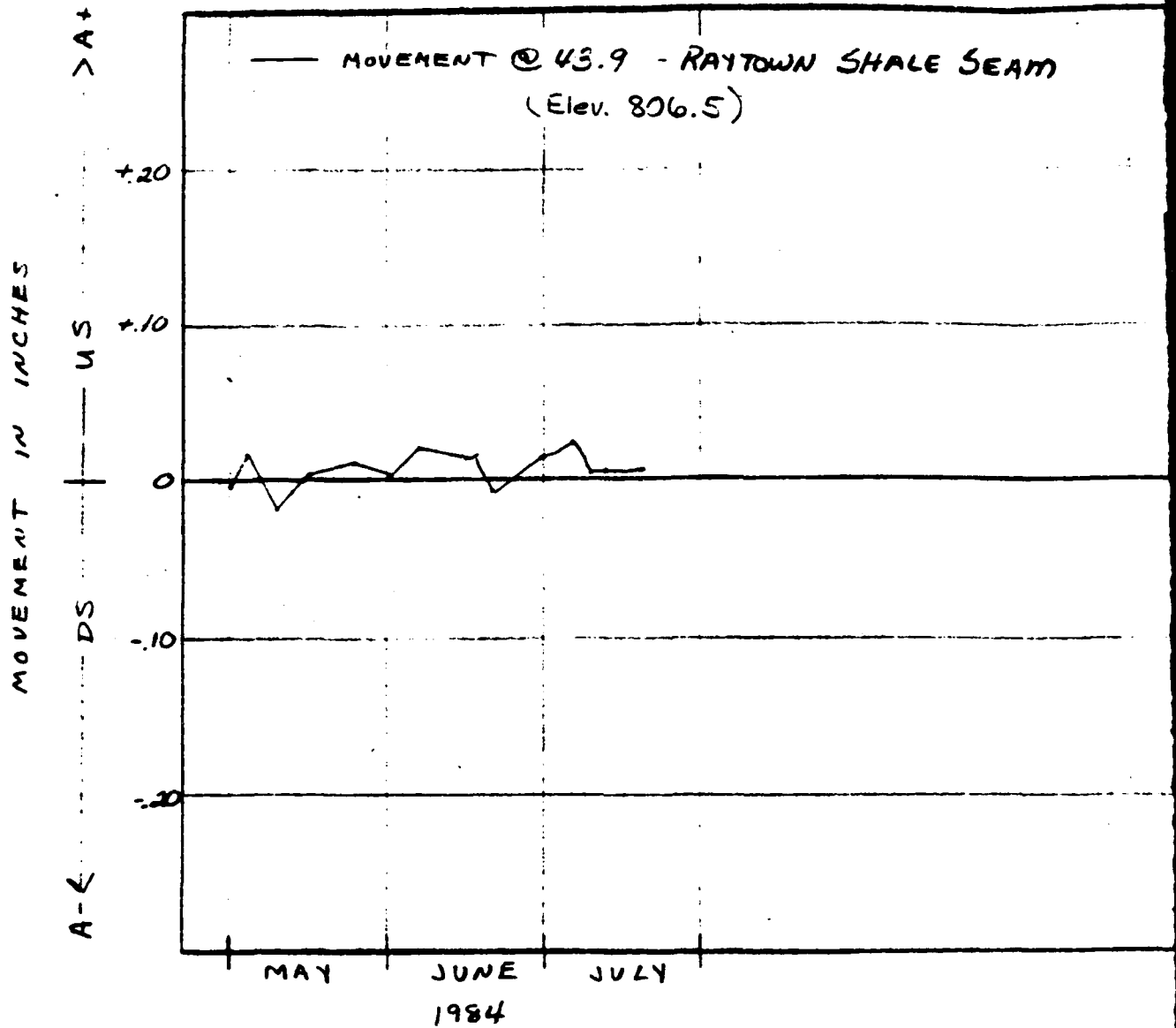
DEPTH IN FEET



SMITHVILLE DAM
LEFT ABUTMENT

Inclinometer Plot
I-110-1

TYPICAL READINGS
APPENDIX A RP-3-1689
PLATE 15 July 1984



MOVEMENT VS. TIME
TIME IN MONTHS

I-110-1

Sb 110100

Rg 2+40 DS

15-1-7

m

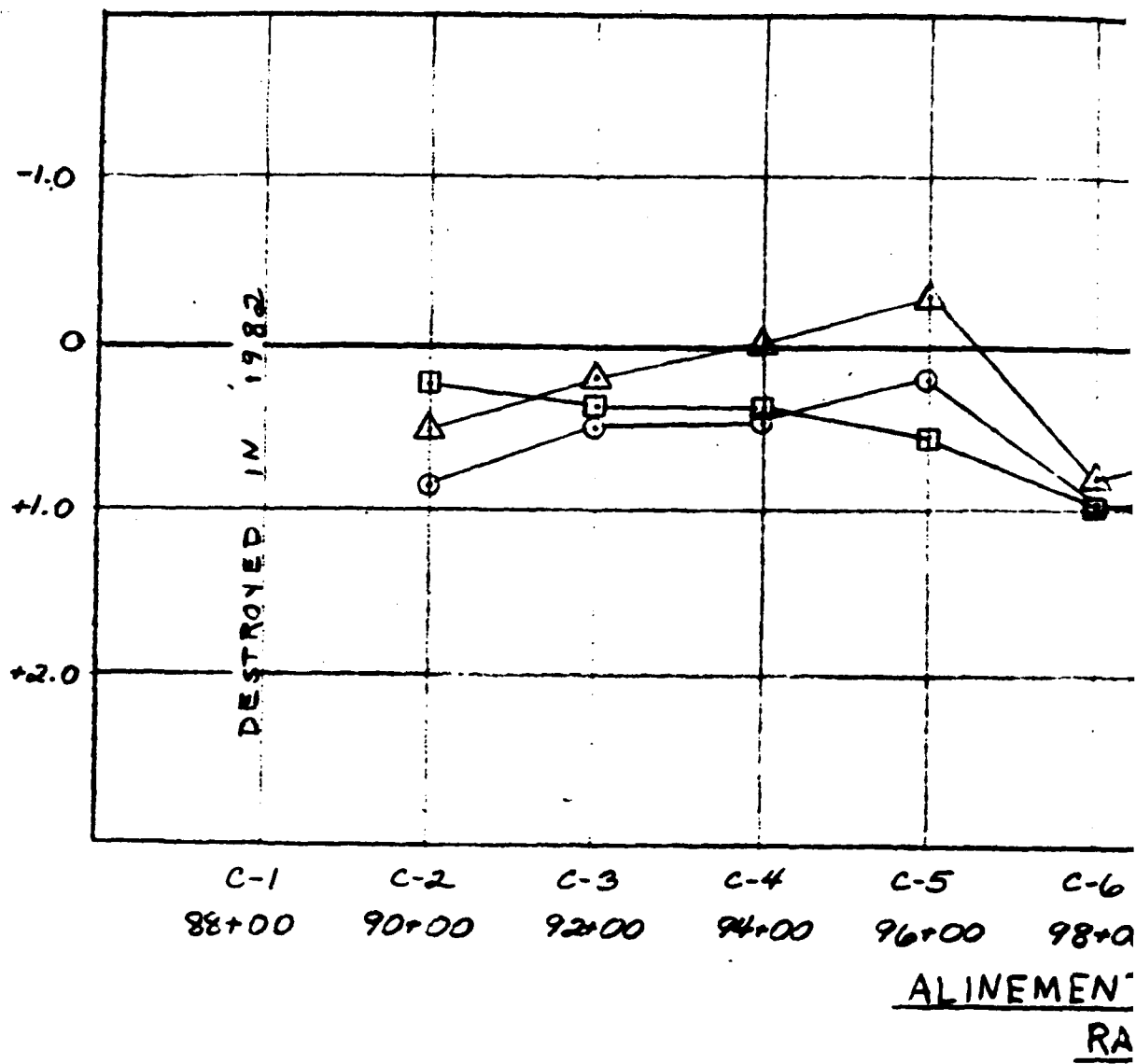
VS. TIME
MONTHS

SMITHVILLE DAM
LEFT ABUTMENT
INCLINOMETER PLOT
I-110-1
MOVEMENT VS TIME AT
RAYTOWN SHALE SEAM
PLATE 16 JULY 1981
APPENDIX A RP-3-1690

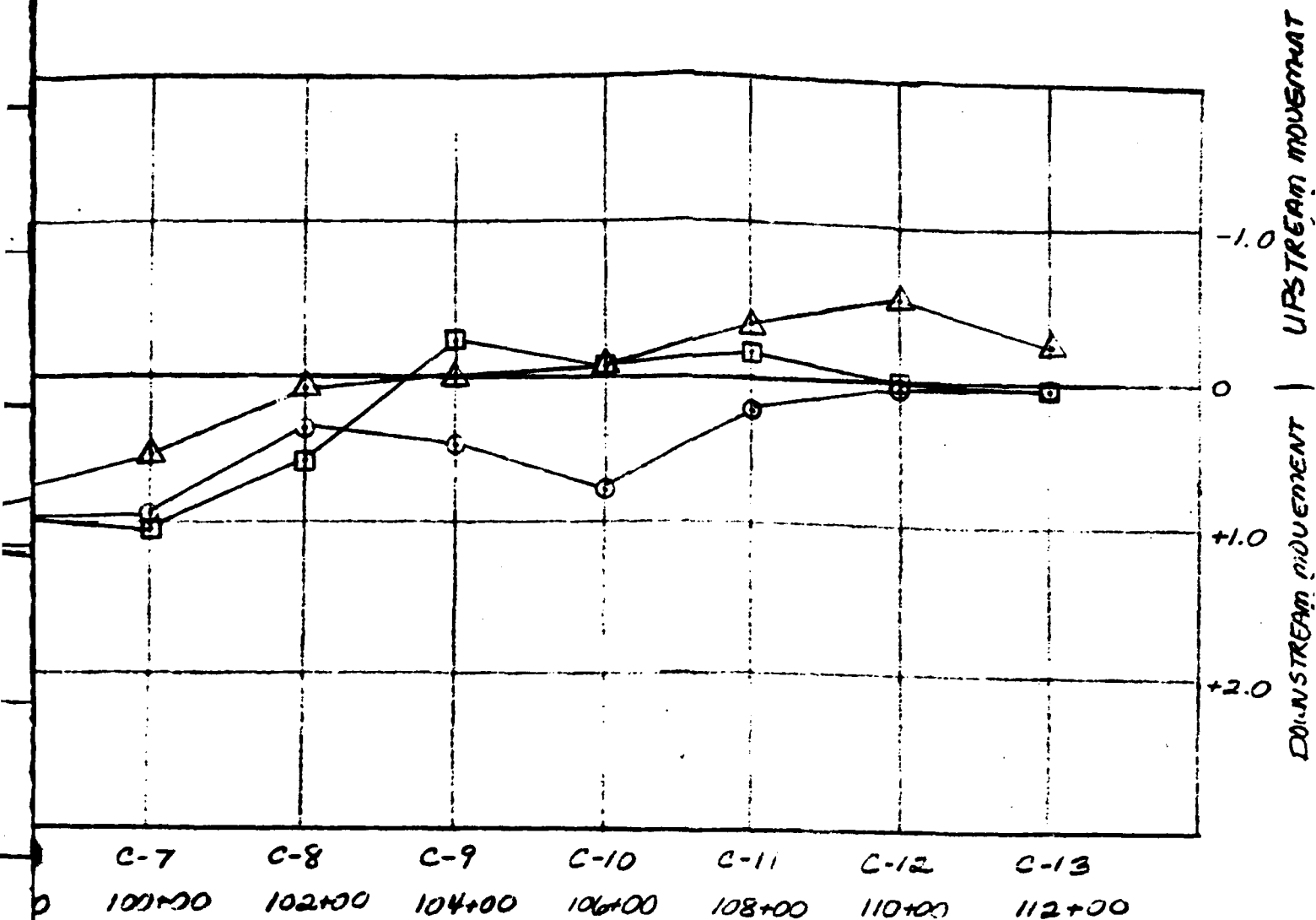
1

2

HORIZONTAL MOVEMENT IN CENTIMETERS



15-1-7



MONUMENT LINE "C"

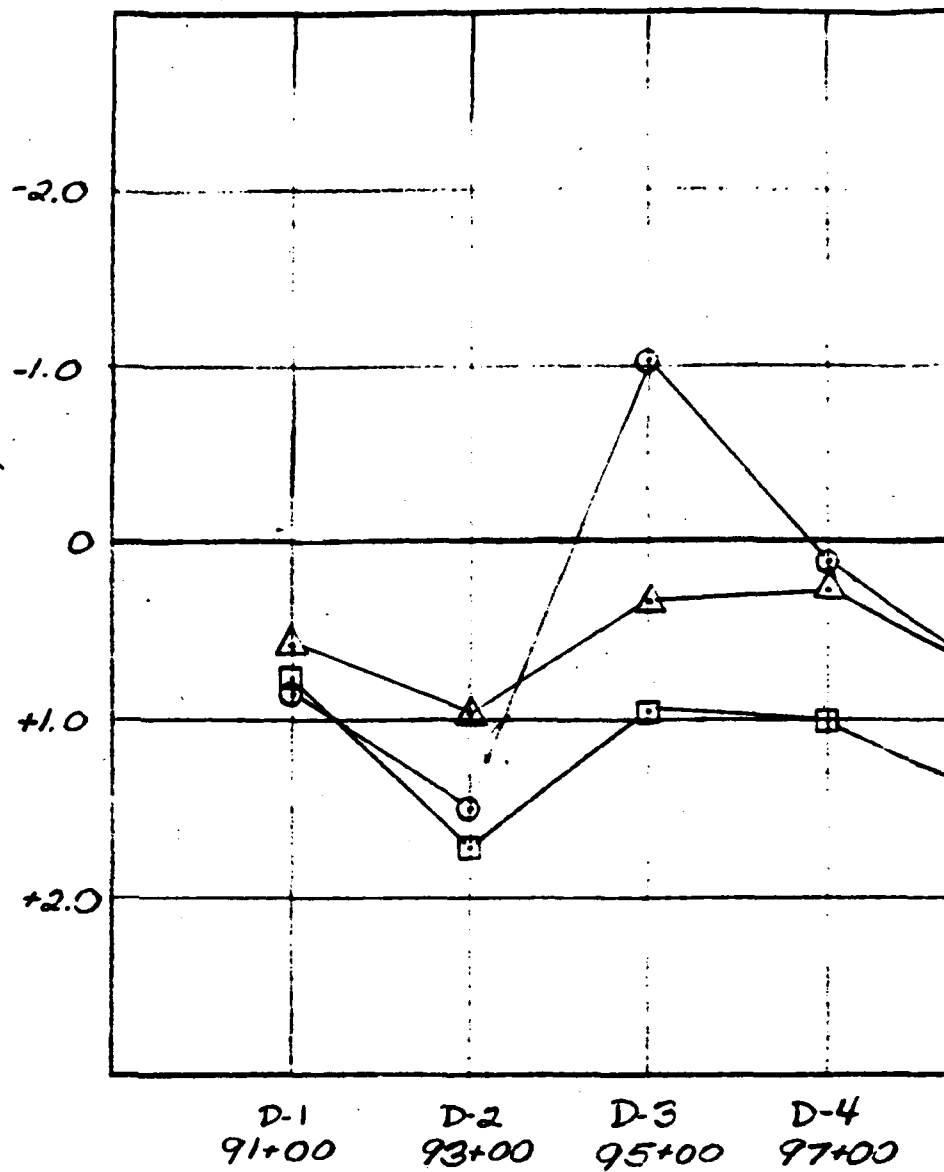
NGE 2+00 D/S

LEGEND

- 1981 BASELINE
- 16 APRIL 1984
- 14 MAY 1984
- △ 11 JUNE 1984

Smithville Dam
 RP-3-1691
 APPENDIX A PLATE 17 July 1984

$- = W/S$
 MOVEMENT
 HORIZONTAL MOVEMENT IN CENTIMETERS
 $+ = D/S$
 MOVEMENT

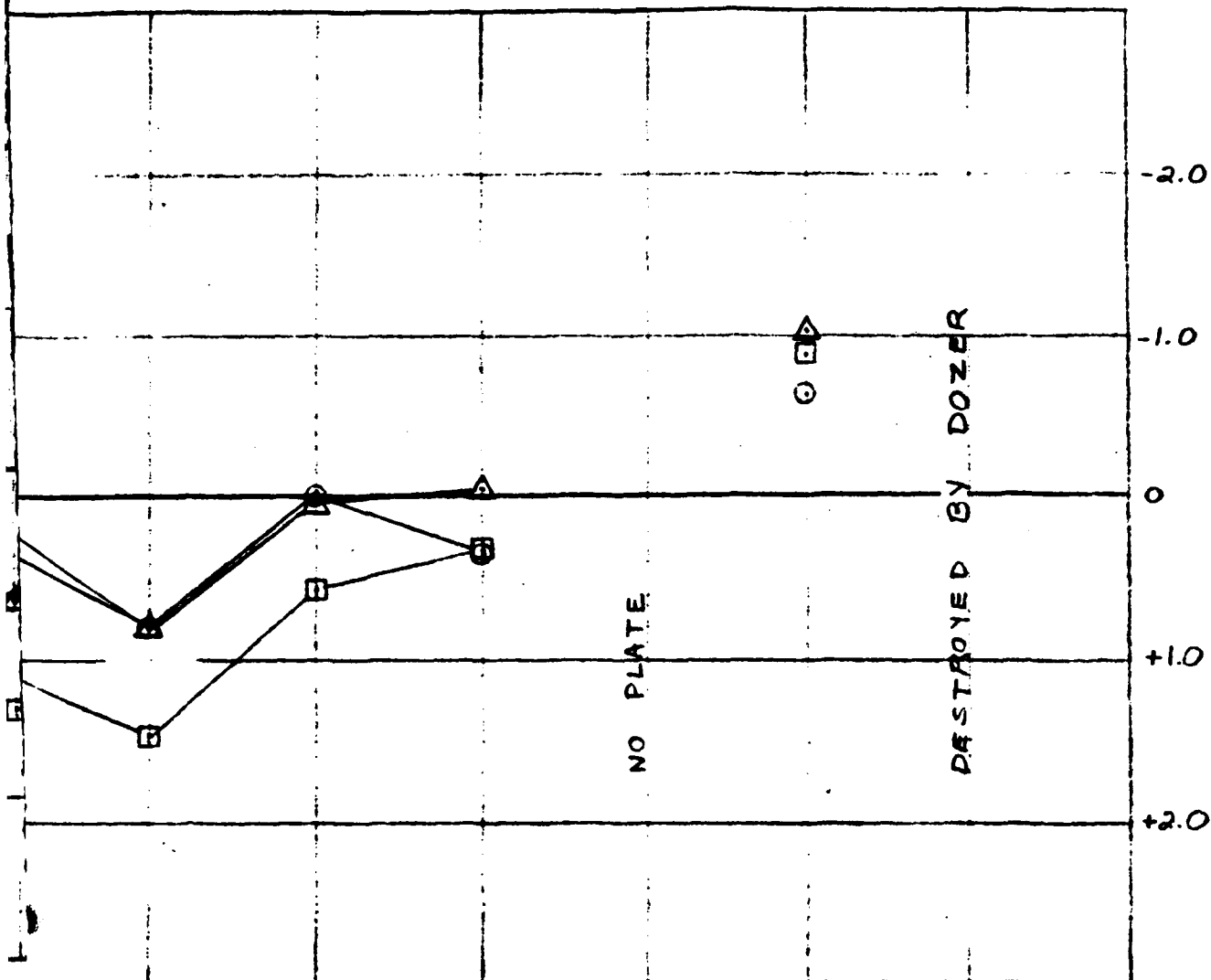


ALINEMENT
 R

LEGEND

- 1980 BASELINE
- 16 April 1984
- 14 May 1984
- △ 4 June 1984

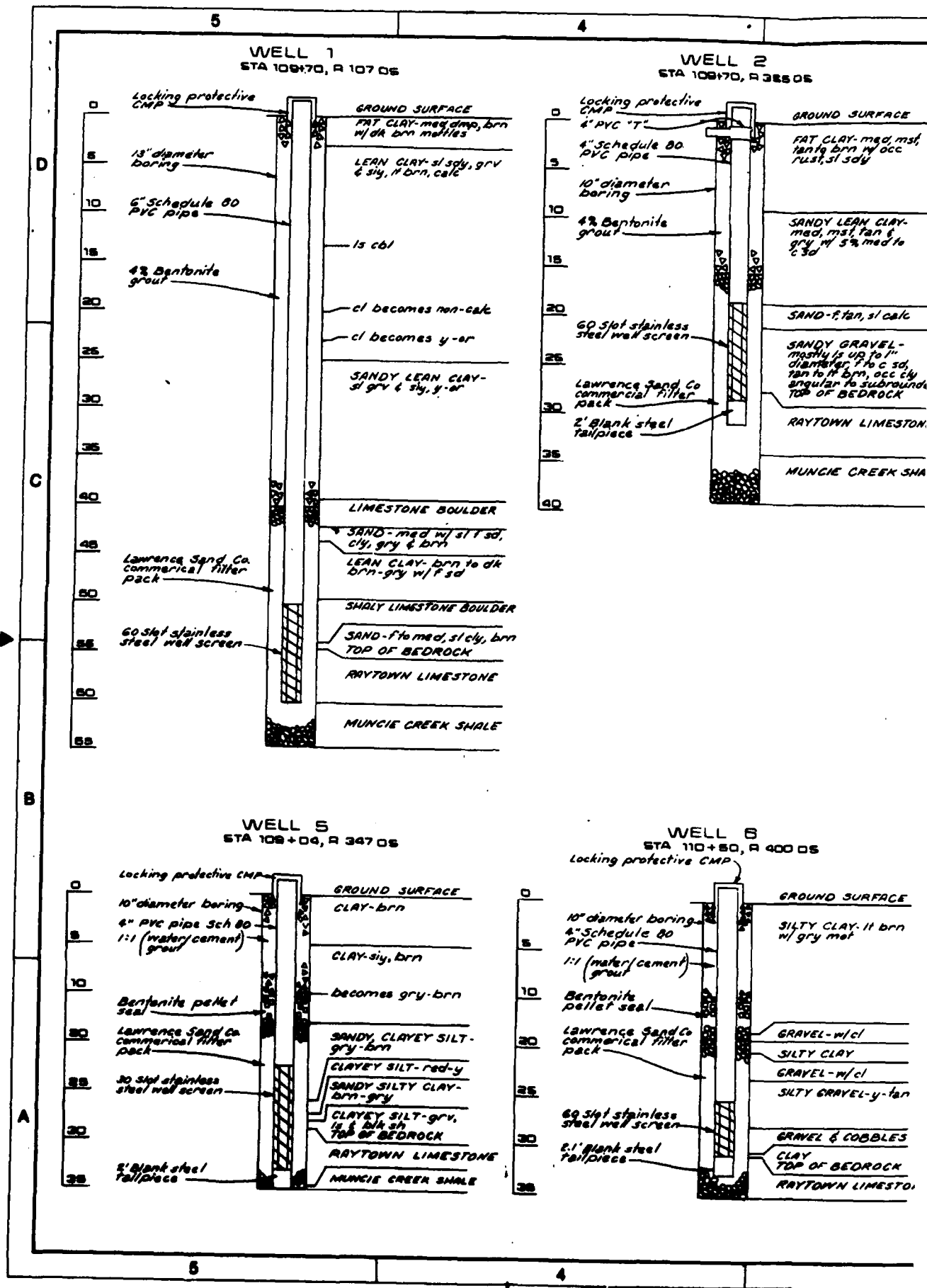
15-1-7



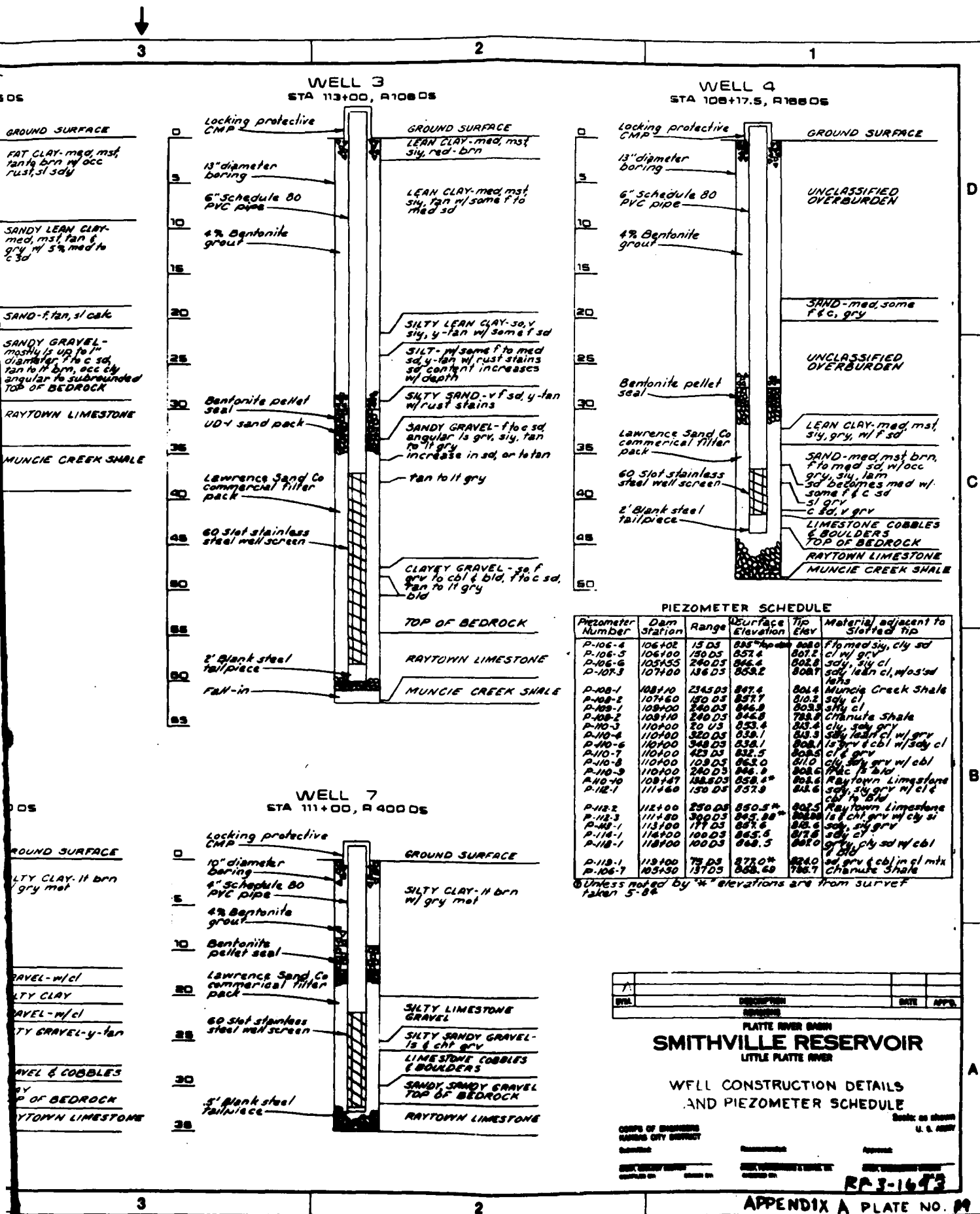
D-5 D-6 D-7 D-8 D-9 D-10
 99+00 101+00 103+00 105+00 107+00 109+00

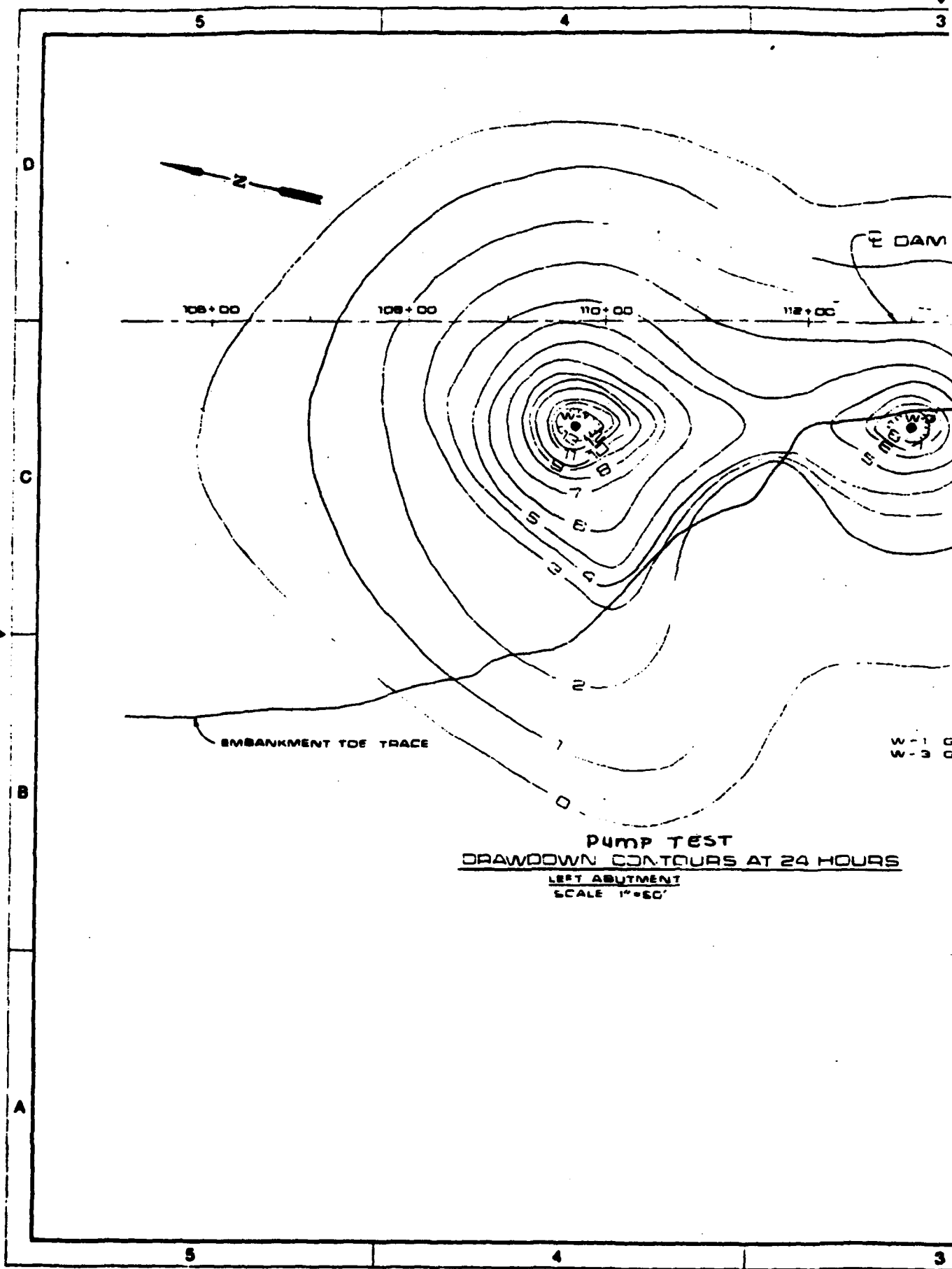
MONUMENT LINE "D"
 RANGE 4+00D/S

SMITHVILLE DAM
 LEFT ABUTMENT
 ALINEMENT LINE "D"
 TYPICAL DATA SHEET
 APPENDIX A RP-3-1692
 PLATE 18 JULY 1964

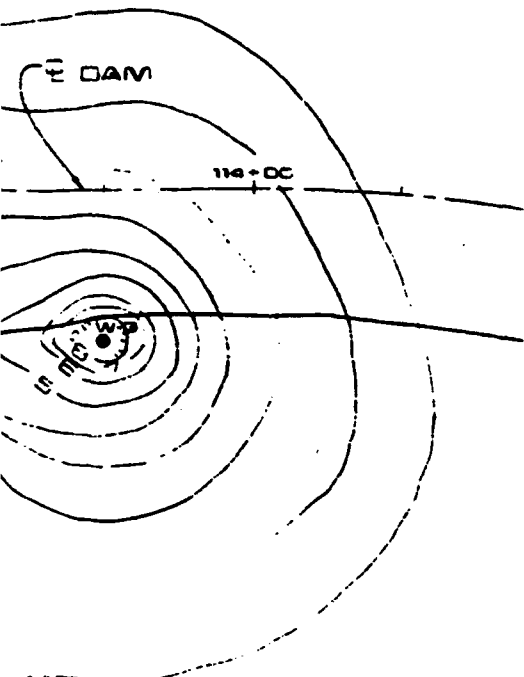


15-1-7





15-1-7



W-1 Q = 7.5 GPM
W-3 Q = 7.7 GPM

OURS

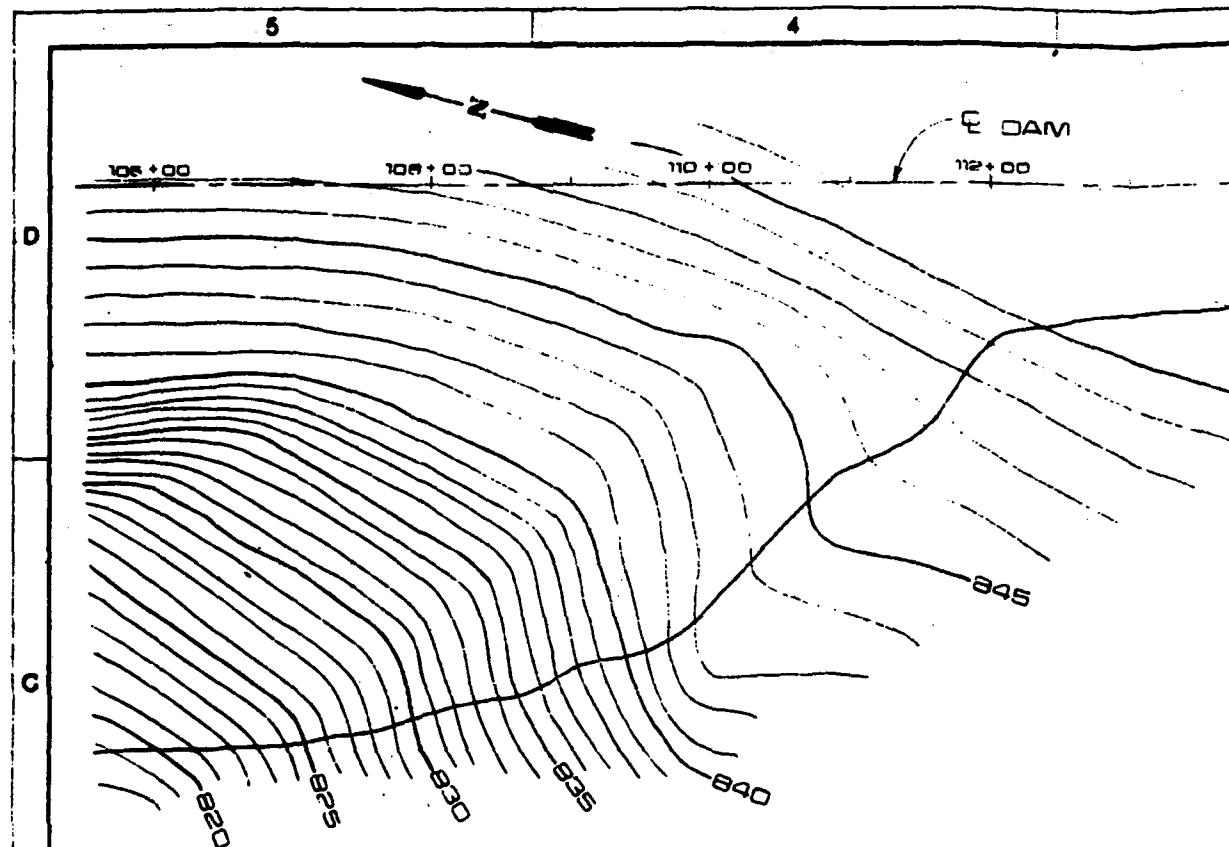
DATE	DESCRIPTION	DATE	APP'D.
	PLATTE RIVER DAM		
SMITHVILLE RESERVOIR LITTLE PLATTE RIVER PUMP TEST DRAWDOWN CONTOURS			
CORPS OF ENGINEERS KANSAS CITY DISTRICT		Scale: as shown U. S. ARMY	
DESIGNED BY	CHECKED BY	APPROVED BY	DATE
			RP-3-1698

APPENDIX A PLATE NO. 20

3
↑

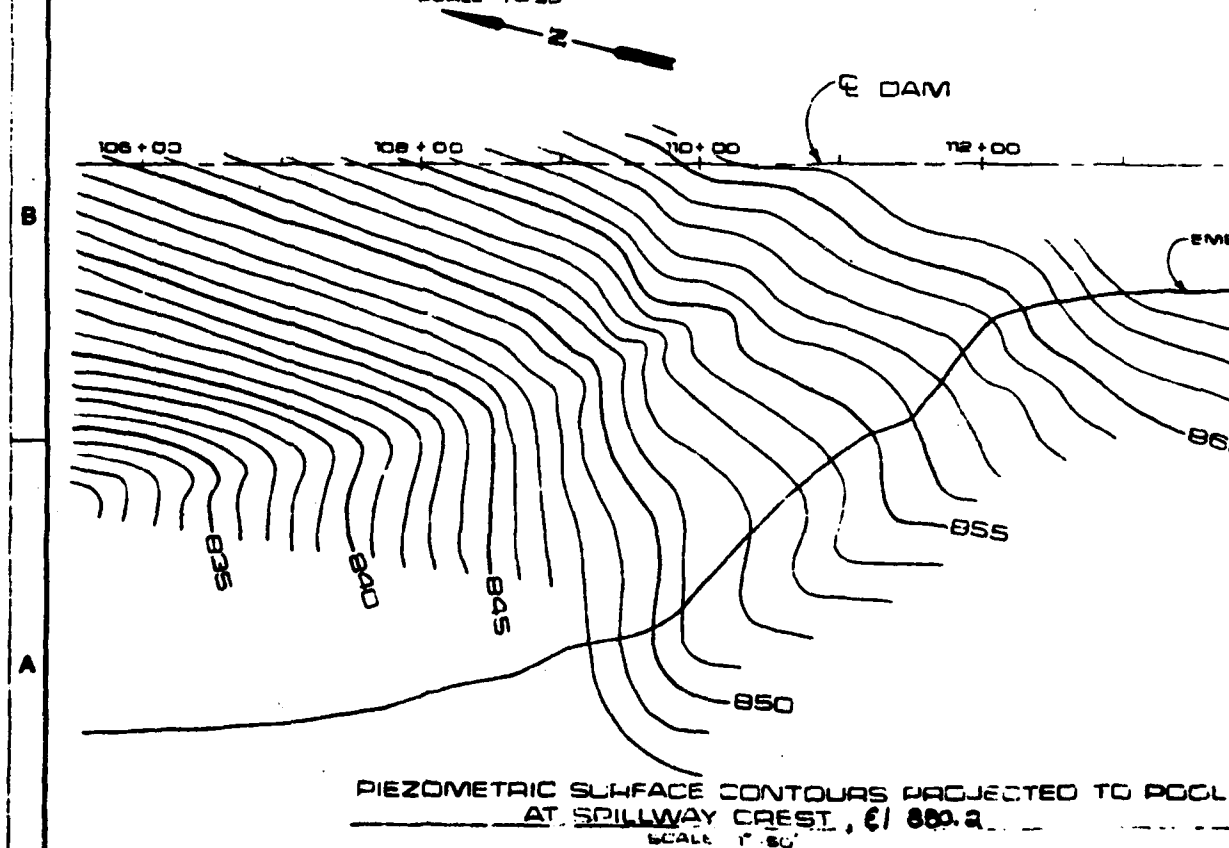
1

2



PIEZOMETRIC SURFACE CONTOURS
AT MULTIPURPOSE POOL, $\text{E} 864.3$

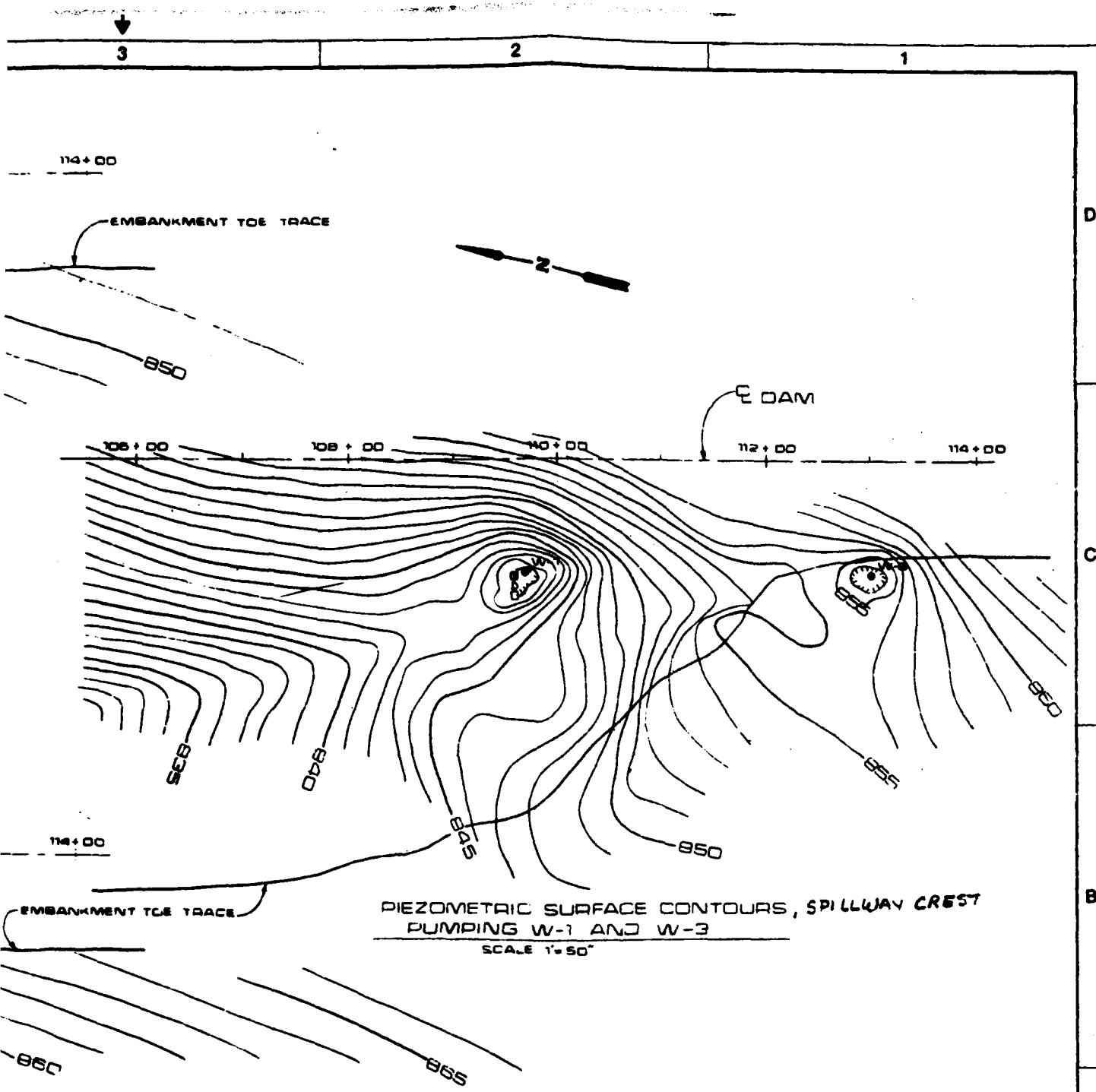
SCALE 1"=50'



PIEZOMETRIC SURFACE CONTOURS PROJECTED TO POOL
AT SPILLWAY CREST, $\text{E} 860.2$

SCALE 1"=50'

15-1-7



PIEZOMETRIC SURFACE CONTOURS, SPILLWAY CREST
PUMPING W-1 AND W-3
SCALE 1"=50'

DATE	DESCRIPTION	DATE	APP'D.
	REVISIONS		

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE

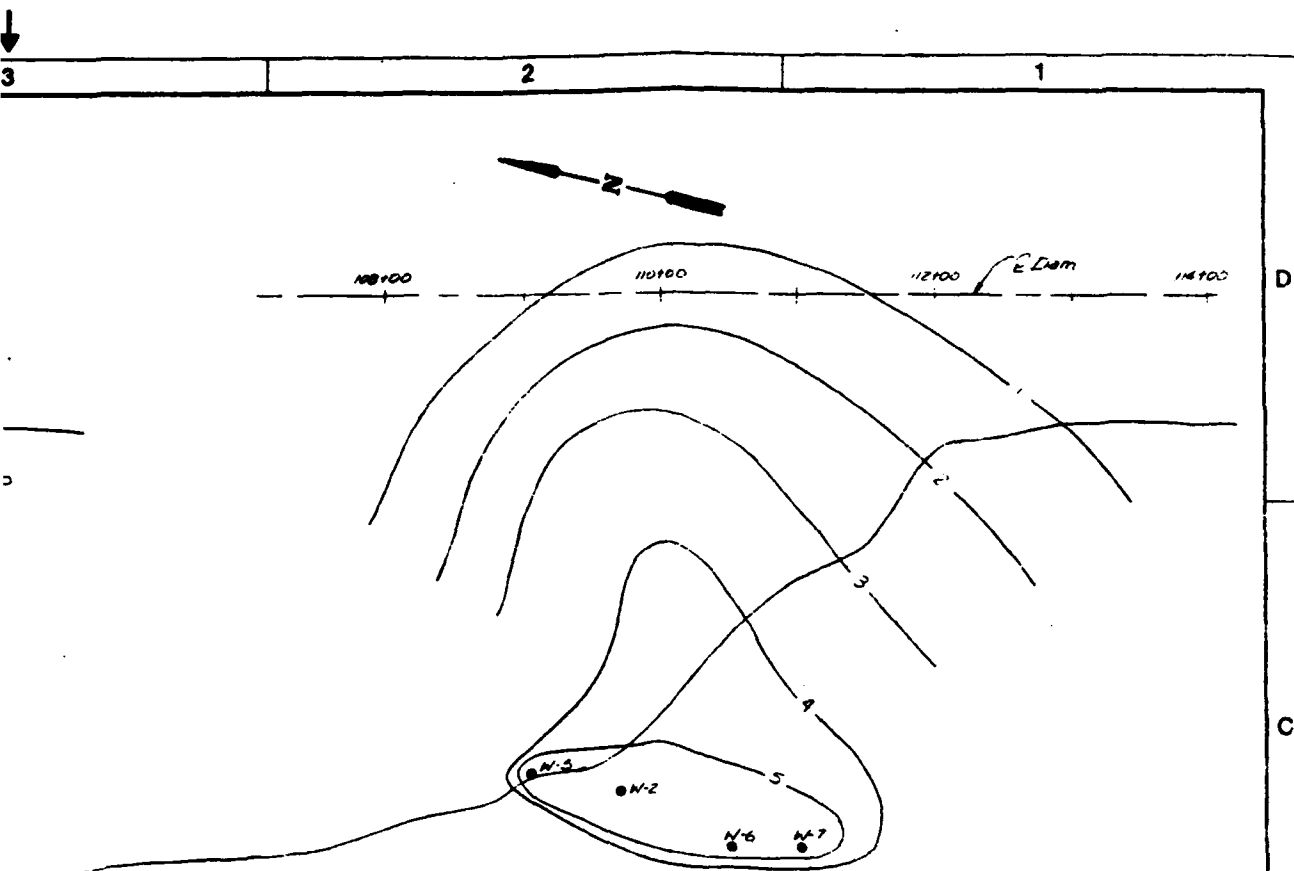
PIEZOMETRIC CONTOURS
INTERIM ANALYSIS

Desg. No. _____
 CORPS OF ENGINEERS
 KANSAS CITY DISTRICT
 Submitted _____
 Date _____

Recommended _____
 Date _____
 Approved _____
 Date _____

Scale: as shown
 U. S. ARMY

RP-3-1695



RELIEF WELLS)

Drawdown Contours W-2, W-5, W-6, W-7 (TEST RELIEF WELLS)
Scale 1" = 50'

Revisions		Date	Approved
Symbol	Description		

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by:	SMITHVILLE LAKE PIEZOMETRIC CONTOURS FINAL ANALYSIS	RP3-1696
Drawn by:		
Checked by:		
Submitted by:		

RELIEF WELLS (TEST RELIEF WELLS)

DIRECT SHEAR & RESIDUAL SHEAR RESULTS					
SYMBOL	SAMPLE NUMBER	GEOLOGIC MEMBER	NORMAL STRESS σ_n	DRAINED SHEAR STRENGTH	
				PEAK $\tan \phi$	RESIDUAL $\tan \phi_r$
1	J-108-1	RALS - THK SH SEAM	6.0	.328*	.147*
2	C-525	RALS - SH SEAM @ 40'	6.0	.561	***
3	C-525	RALS - THK SH SEAM w/ SLICKENSIDES	6.0	.416	.196
4	C-525	RALS - THK SH SEAM w/o SLICKS	6.0	.469	.191
5	C-525	RALS / Mc SH CONTACT	6.0	.639	.387***
6	C-525	CH SH - Lo ANGLE SOFT ZONE	6.0	.499	.226***
7	C-526	RALS - THK SH SEAM	6.0	.230**	.13**
8	C-526	RALS - THK SH SEAM	6.0	.296**	.13**
9	C-526	Mc SH	6.0	.423	.213**
10	C-527	RALS - THK SH SEAM	6.0	.59	.23***
11	C-528	RALS - THK SH SEAM	6.0	.289	.129***
12	C-529	Mc SH	6.0	.559	.215***
13	C-532	RALS - THK SH SEAM	4.0	.313	

* REMOLDED SPECIMEN

** PRECUT SPECIMEN

*** DID NOT DEVELOP RESIDUAL CONDITION

15-1-7

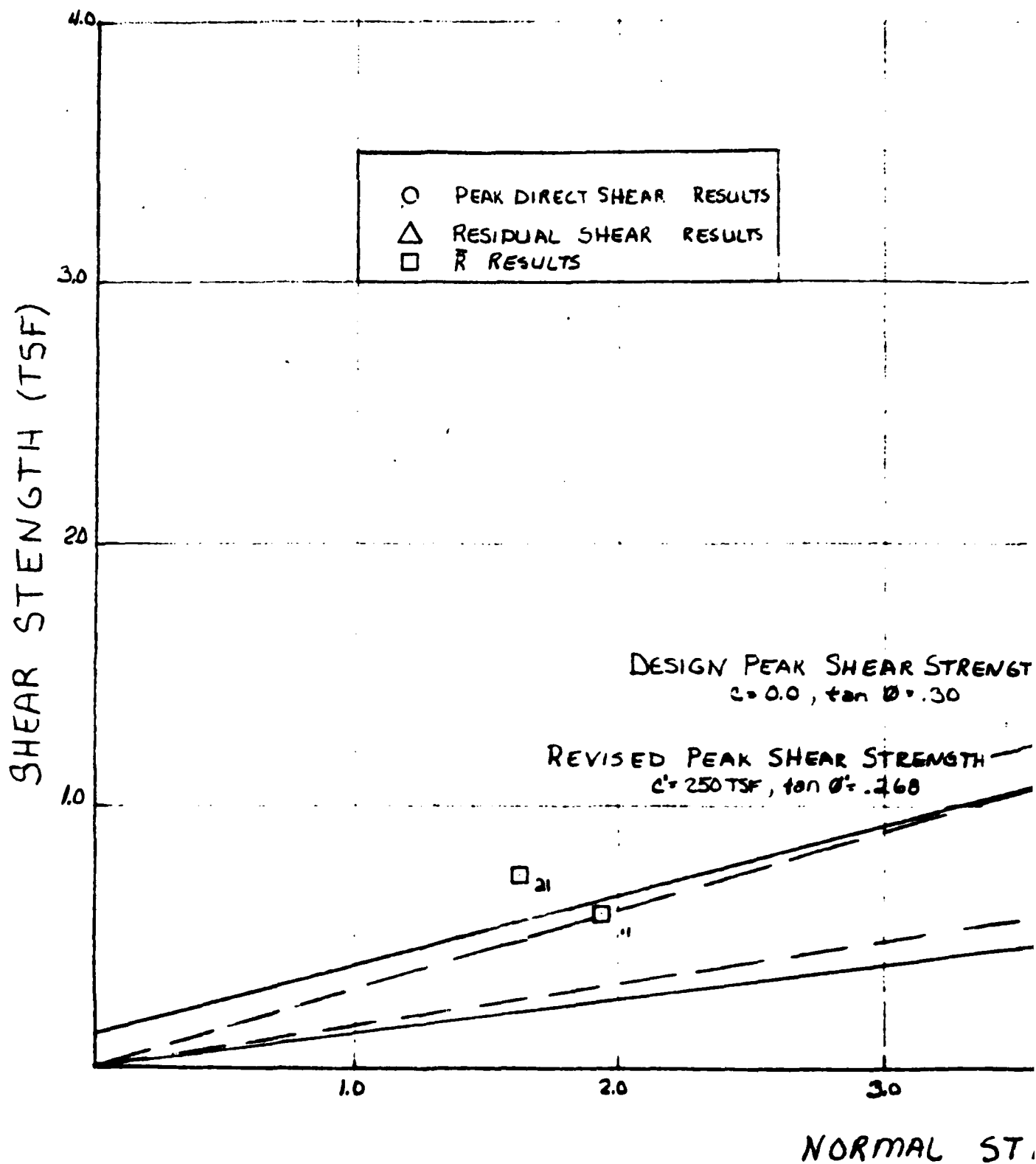
TRIAXIAL "R" RESULTS				
SYMBOL	SAMPLE	σ_{3c}	τ_{ff}	σ_{ff}
$\bar{R}_{11}$	C-527	2	.59	1.94
$\bar{R}_{12}$	C-527	6	1.40	5.22
$\bar{R}_{21}$	C-533	2	.73	1.62
$\bar{R}_{22}$	C-533	6	1.81	4.96

SMITVILLE DAM
LEFT ABUTMENT

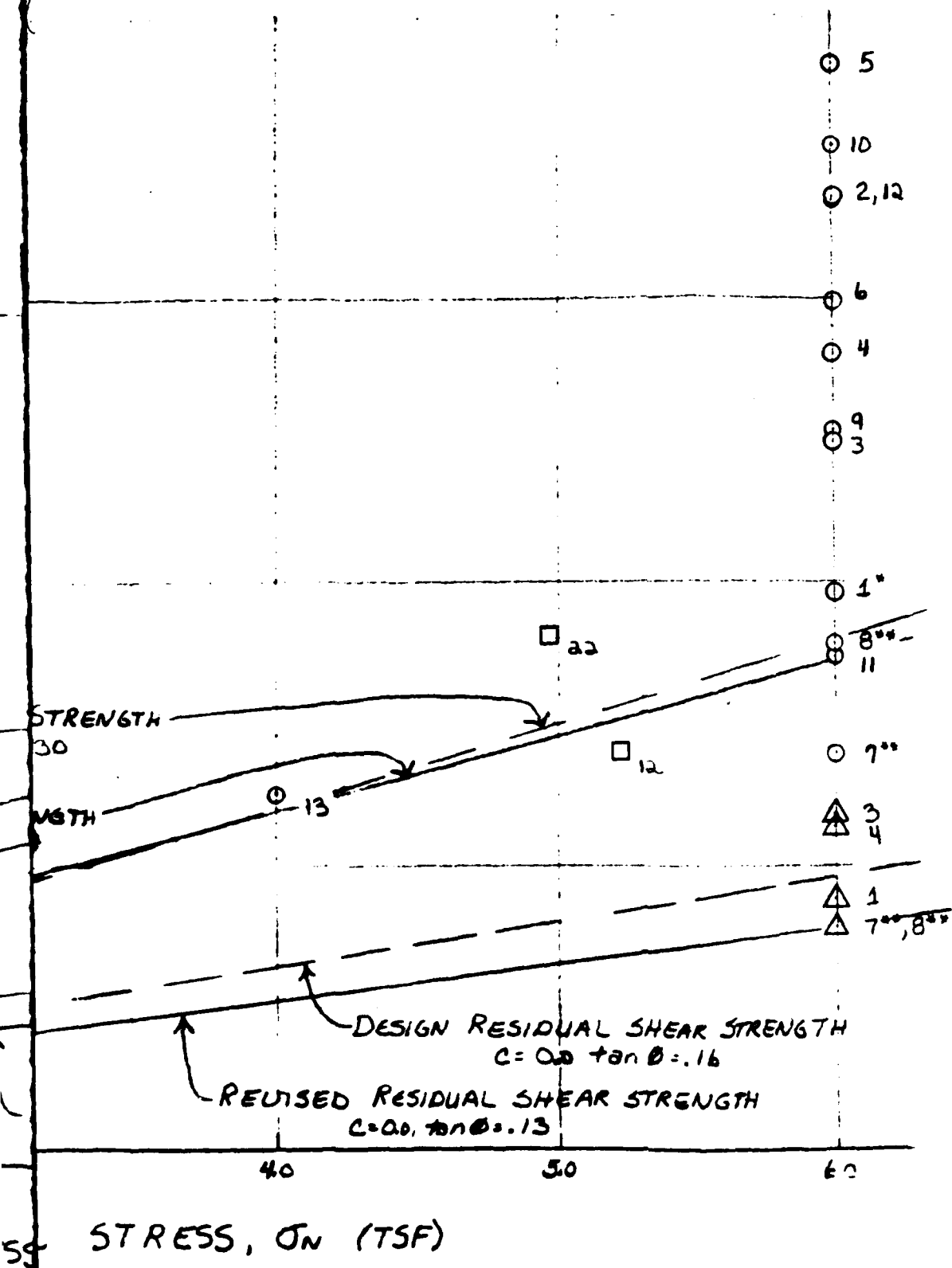
SUMMARY OF SHEAR STRENGTH
RESULTS

APPENDIX A RP-3-1697
PLATE 23 JULY 1984

2



15-1-7

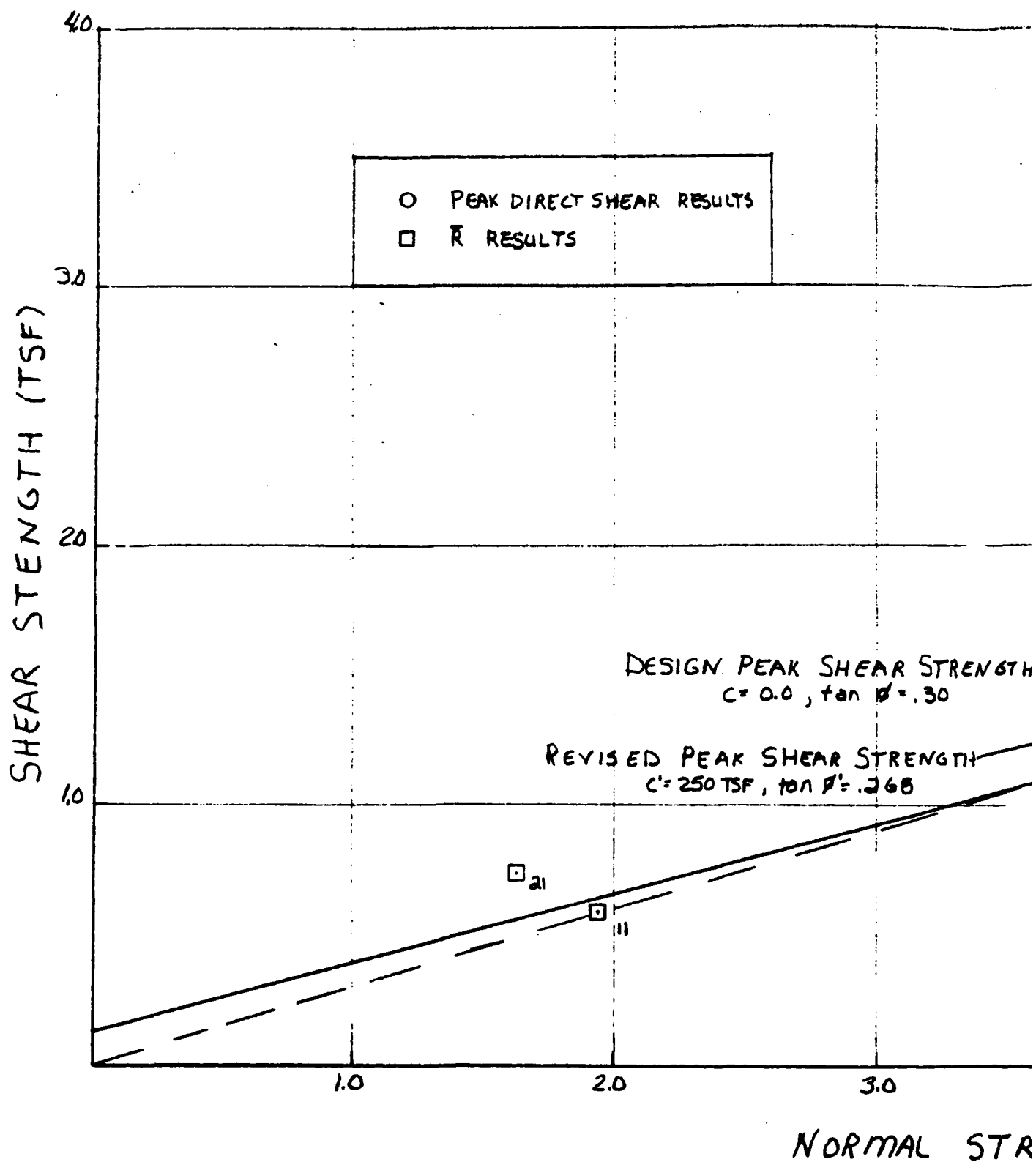


SMITHVILLE
 DAM:
 LEFT ABUTMENT

PLOT OF
 SHEAR STRENGTH
 RESULTS

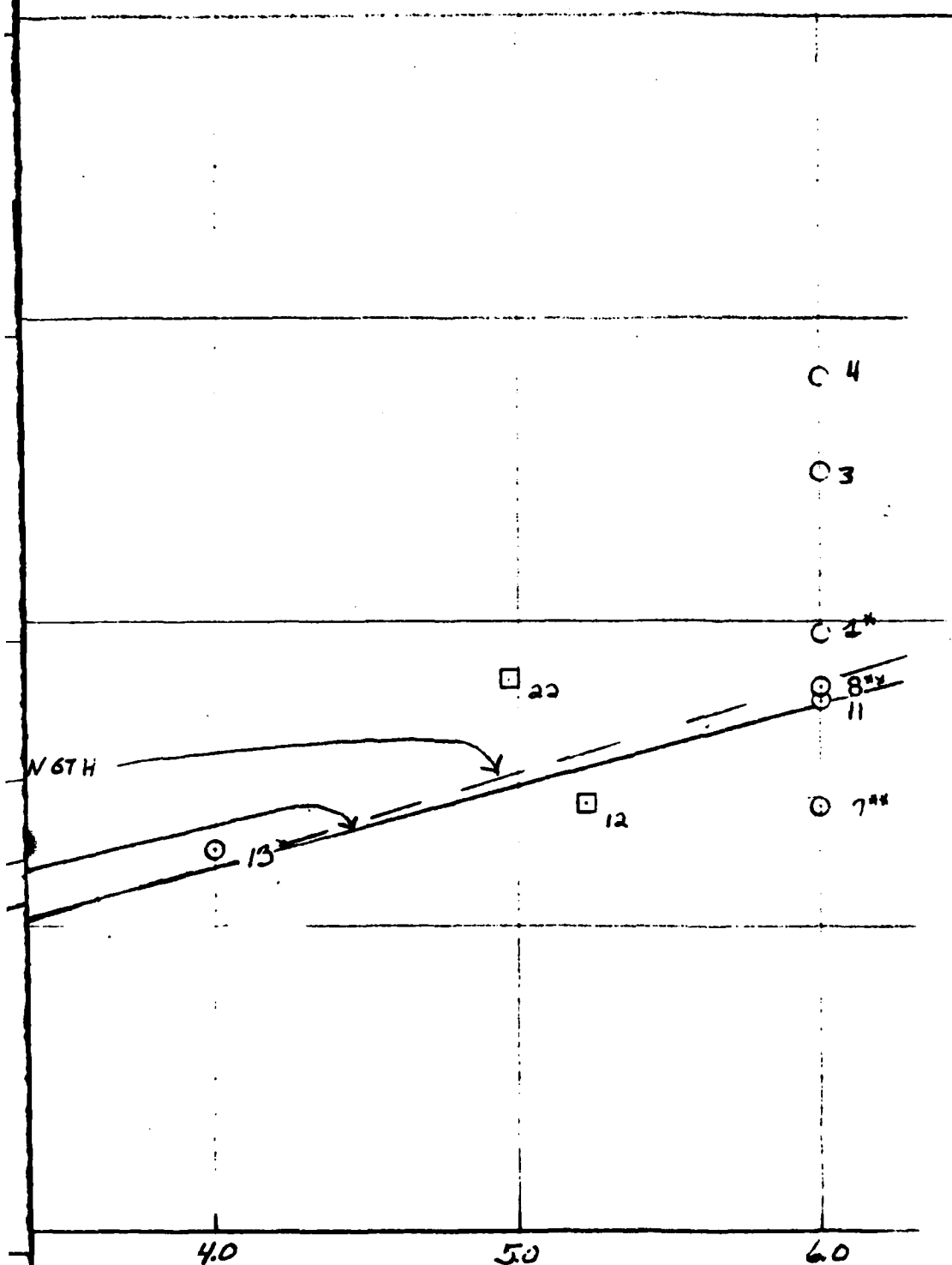
RP-3-1698
 APPENDIX A
 PLATE 24 JULY 1984

2



15-1-7

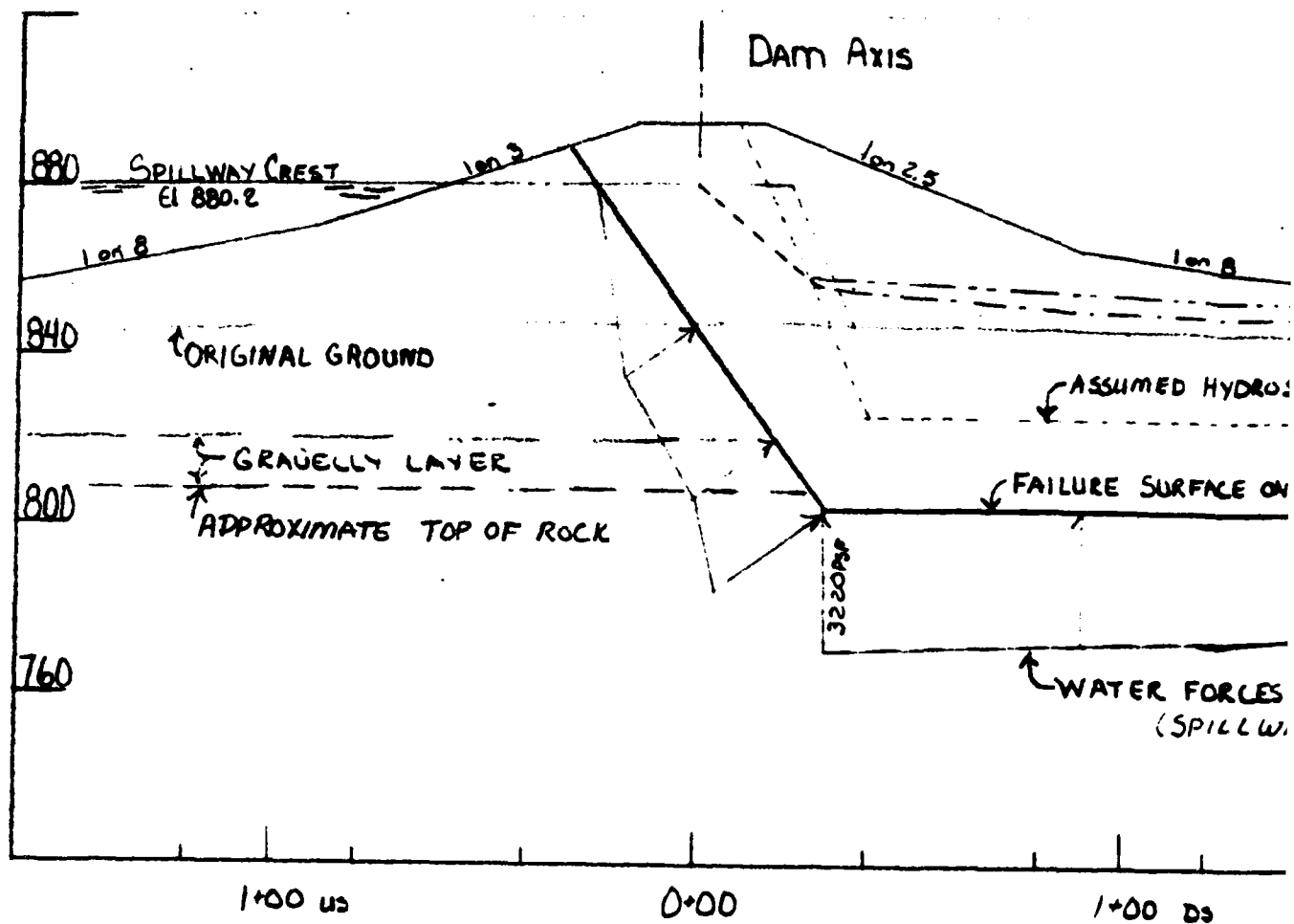
1



PRESS, IN (TSF)

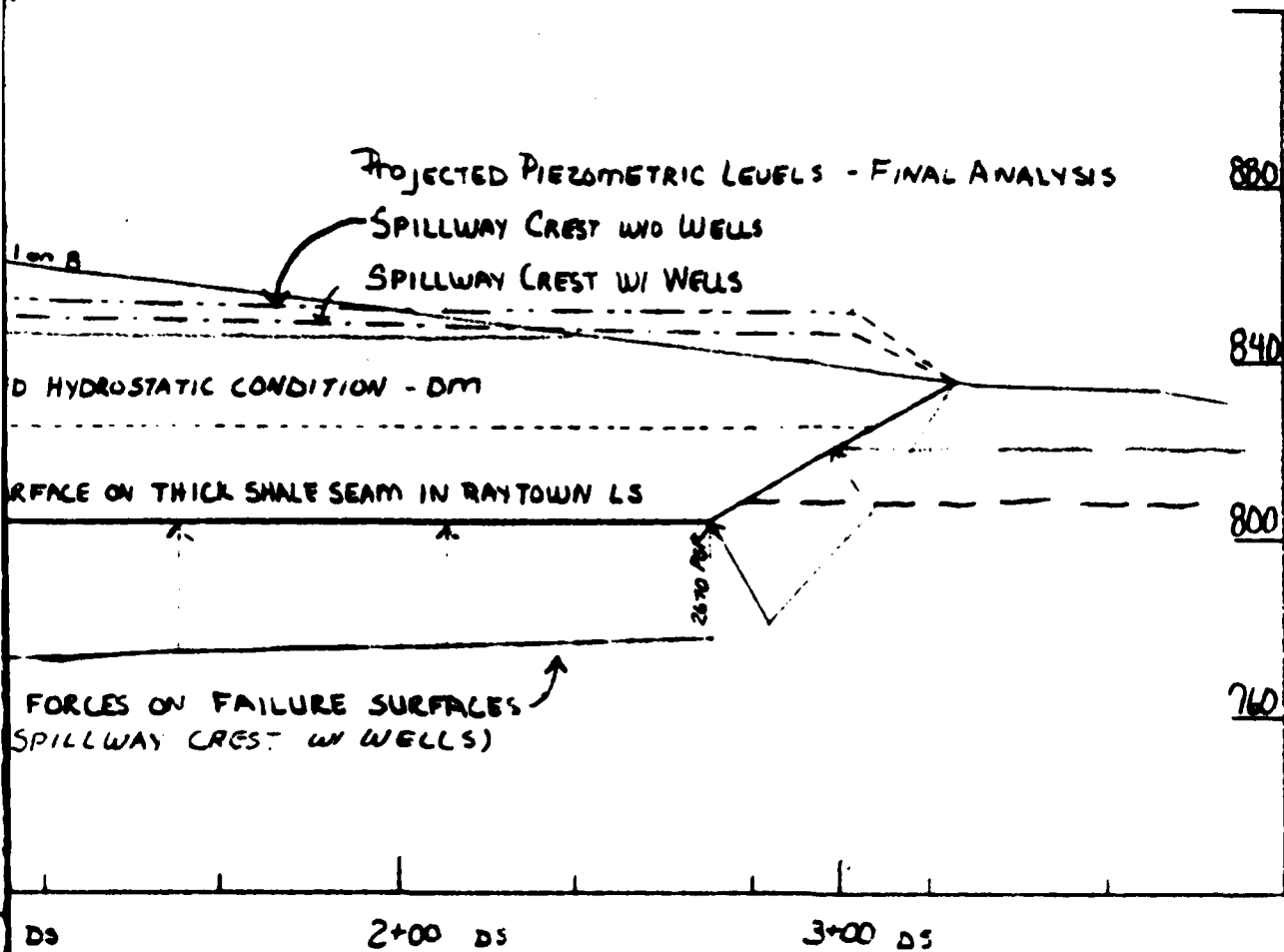
SMITHVILLE
DAM
LEFT ABUTMENT

PLOT OF
SHEAR STRENGTH
RESULTS
RAYTOWN SHALE
SEAM
RP-3-1699
APPENDIX A
PLATE 25 JULY 1984



STABILITY SECTION STA 110+

15-1-7



110+00

SMITHVILLE DAM
LEFT ABUTMENT
STABILITY SECTION
STA 110+00
APPENDIX A RP-3-1700
PLATE 26 JULY 1984

SHEAR STRENGTH PARAMETERS						
ANALYSIS	DM; PRELIM		INTERIM		FINAL	
PEAK STRENGTH	c (PSF)	tan ϕ	c (PSF)	tan ϕ	c' (PSF)	
EMBANKMENT*	0/300	.45/.325	0	.45	0	
DUBN - FINEGRAINED ²	0/200	.45/.325	0	.45	0	
DUBN - GRAVELLY	200	.325	0	.577	0	
JOINT IN RAYTOWN	~	~	0	.577	0	
RAYTOWN LS SH SEAM	0	.30	160	.264	250	
RESIDUAL STRENGTH						
RAYTOWN LS SH SEAM	0	.16	~	~	~	
STR @ 0.5% STRAIN						
DUBN - PASSIVE WEDGE	~	~	40	.13	40	

* DM and PRELIMINARY ANALYSES USED ϕ S, (R+S)/2 ENVELOPE
 INTERIM AND FINAL ANALYSES USED A ϕ ENVELOPE

15-1-7

S
AL
1000
.45
.45
.577
6
268
~
.13

PHYSICAL SOIL PROPERTIES		
MATERIAL	UNIT WEIGHT LBS/FT ³	
	SAT	MOIST
EMBANKMENT	125	120
OVERBURDEN	120	115
BEDROCK	140	140

SMITHVILLE DAM
LEFT ABUTMENT

DESIGN PARAMETERS

APPENDIX A RP-3-1701
PLATE 27 JULY 1984

DM ANALYSIS		
POOL CONDITION ¹	STRENGTH APPROACH ²	METHOD
		HAND WEDGE ³
SPILLWAY CREST	1	1.64
SPILLWAY CREST	2	1.08

PRELIMINARY ANALYSIS				
POOL CONDITION ¹	STRENGTH APPROACH ²	METHOD		
		HAND WEDGE ³	SLOPE ⁴	
			F	θ
FULL POOL	1	1.22	1.42	8.0
FULL POOL	2	0.92	1.00	6.9

INTERIM ANALYSIS*				
POOL CONDITION ¹	STRENGTH APPROACH ²	METHOD		
		HAND WEDGE ³	SLOPE ⁴	
			F	θ
MULTIPURPOSE	3	1.40	1.66	8.5°
SPILLWAY CREST	3	1.20	1.41	8.0°
SPILLWAY CREST W/ PUMPED WELLS	3	1.32	1.55	8.9°

15-1-7

Pool COND
SPILL w/o R SPILL w/ FL

¹ Pool Con

² STRENGTH

³ HAND W

⁴ SLOPE

* SEE F

FINAL ANALYSIS

POOL CONDITION ¹	STRENGTH APPROACH ²	METHOD		
		HAND WEDGE ³	SLOPE ^{BR} ⁴	
			F	θ
SPILLWAY CREST w/o RELIEF WELLS	3	1.25	1.47	8.7
SPILLWAY CREST w/ FLOWING TEST WELLS	3	1.30	1.53	8.7

¹ POOL CONDITION : HYDROSTATIC PRESSURE WAS ASSUMED IN EMBANKMENT CORRESPONDING TO THE GIVEN POOL CONDITION. IN THE DM ANALYSIS, DOWNSTREAM UPLIFT CORRESPONDED TO A SATURATION LINE AT EL. 825 IN THE FOUNDATION. IN ALL OTHER ANALYSES, DOWNSTREAM UPLIFT CORRESPONDED TO ACTUAL OR PROJECTED PIEZOMETRIC LEVEL IN THE PERMEABLE BASAL LAYER IN THE FDM.

² STRENGTH APPROACH:

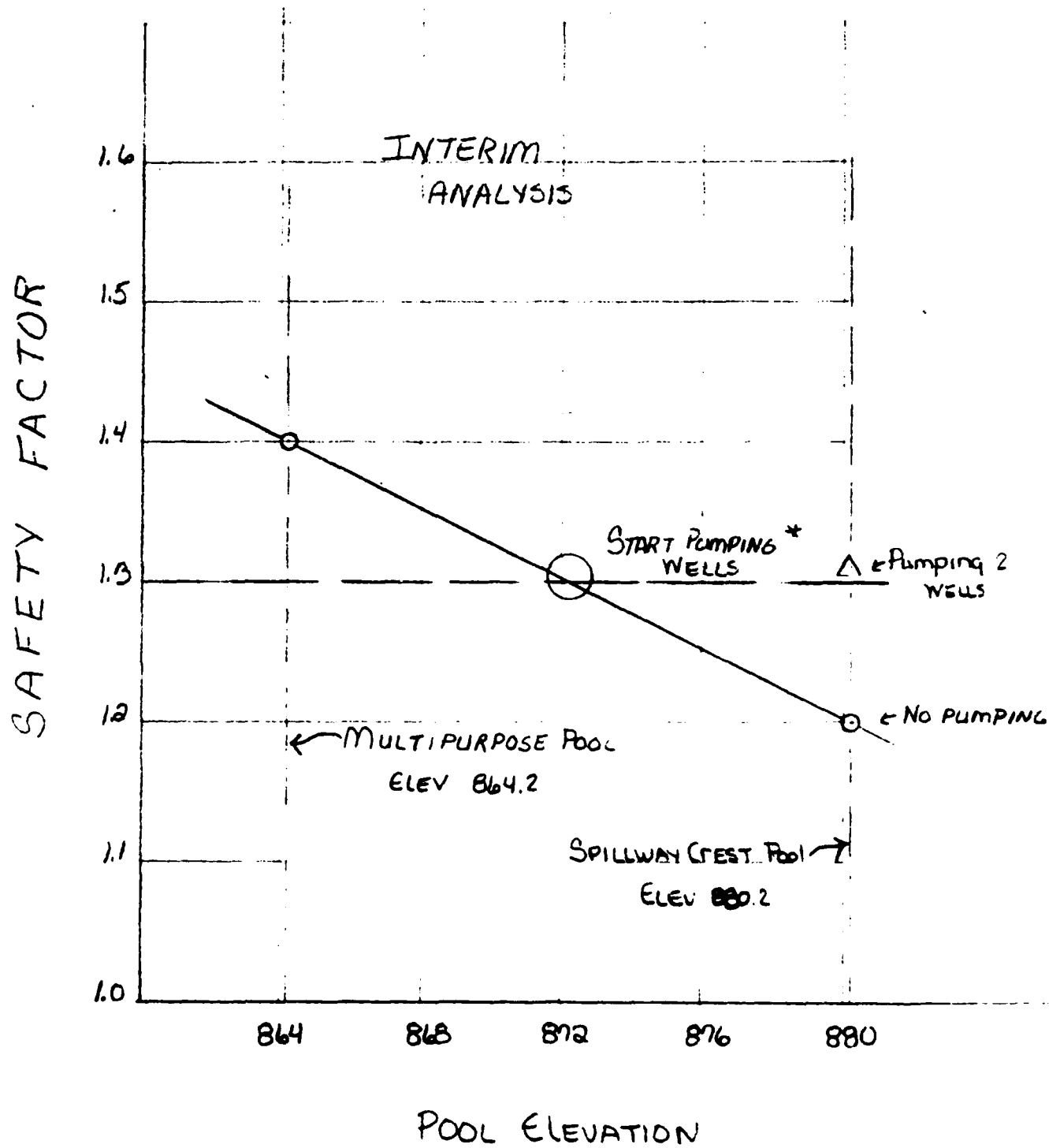
APPROACH 1 : PEAK STRENGTH USED ALONG ENTIRE SLIDE SURFACE
 APPROACH 2 : PEAK STRENGTH USED IN ACTIVE AND PASSIVE WEDGE, RESIDUAL STRENGTH USED IN CENTRAL BLOCK
 APPROACH 3 : PEAK STRENGTH USED IN ACTIVE WEDGE AND CENTRAL BLOCK; STRENGTH @ 0.5% STRAIN USED IN PASSIVE WEDGE

³ HAND WEDGE : SAFETY FACTOR OBTAINED USED HAND WEDGE METHOD ACCORDING TO EM-1110-2-1906 (APRIL 1970)
 IN DM ANALYSIS, $E_A = 0.8$, $E_P = 0.0$
 IN OTHER ANALYSIS, $E_A = E_P = 0.0$

⁴ SLOPE^{BR} : F = FACTOR OF SAFETY
 θ = SIDE FORCE INCLINATION

* SEE PLATE 29 FOR PLOT OF F VS POOL ELEV.
 FOR HAND WEDGE METHOD

SMITHVILLE DAM
 LEFT ABUTMENT - STA 110+00
 STEADY SEEPAGE STABILITY
 ANALYSIS SUMMARY
 APPENDIX A RP-3-1702
 PLATE 28 JULY 1984



15-1-7

STABILITY STUDIES WERE CONDUCTED AT
STATION 110+00 USING ACTUAL PIEZOMETRIC
LEVELS AT MULTIPURPOSE POOL, PROJECTED
PROJECTED LEVELS AT A SPILLWAY CREST,
AND PREDICTED DRAWDOWN AT A SPILLWAY
CREST POOL PUMPING 2 WELLS

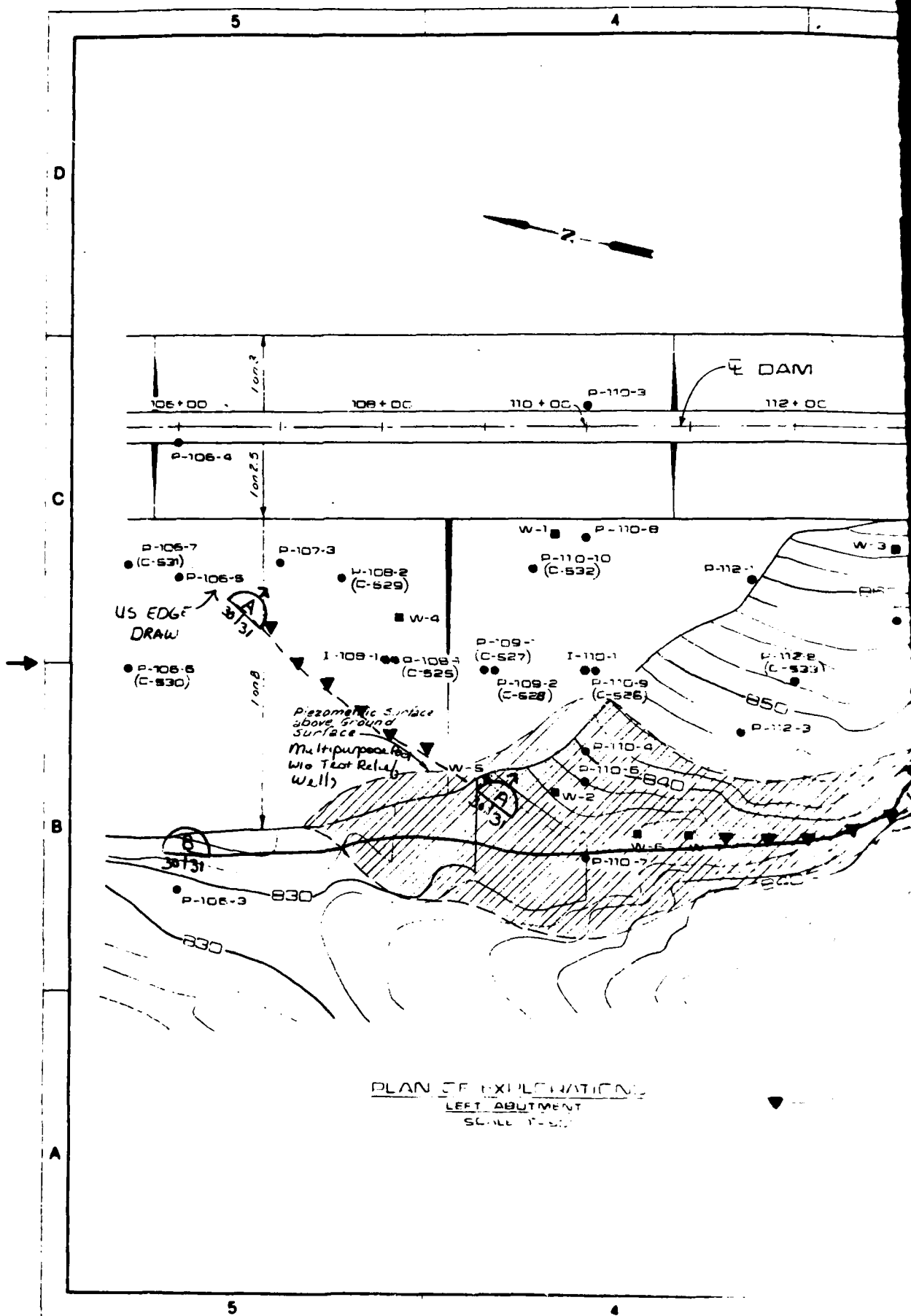
* TO ASSURE A SAFETY FACTOR > 1.3 ,
PUMPING WILL BE STARTED WHENEVER
A POOL OF ELEV 872 OR HIGHER
IS FORECAST.

SMITHVILLE DAM
LEFT ABUTMENT

SAFETY FACTOR VS POOL ELEV
INTERIM ANALYSIS

APPENDIX A RP-3-1703
PLATE 29 JULY 1984

15-1-7



AD-A182 106

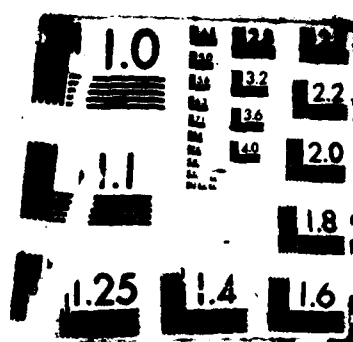
MULTIPLE-PURPOSE PROJECT: PLATTE RIVER BASIN LITTLE
PLATTE RIVER MISSOURI (U) CORPS OF ENGINEERS KANSAS
CITY MO KANSAS CITY DISTRICT F C WALBERG ET AL. MAR 87

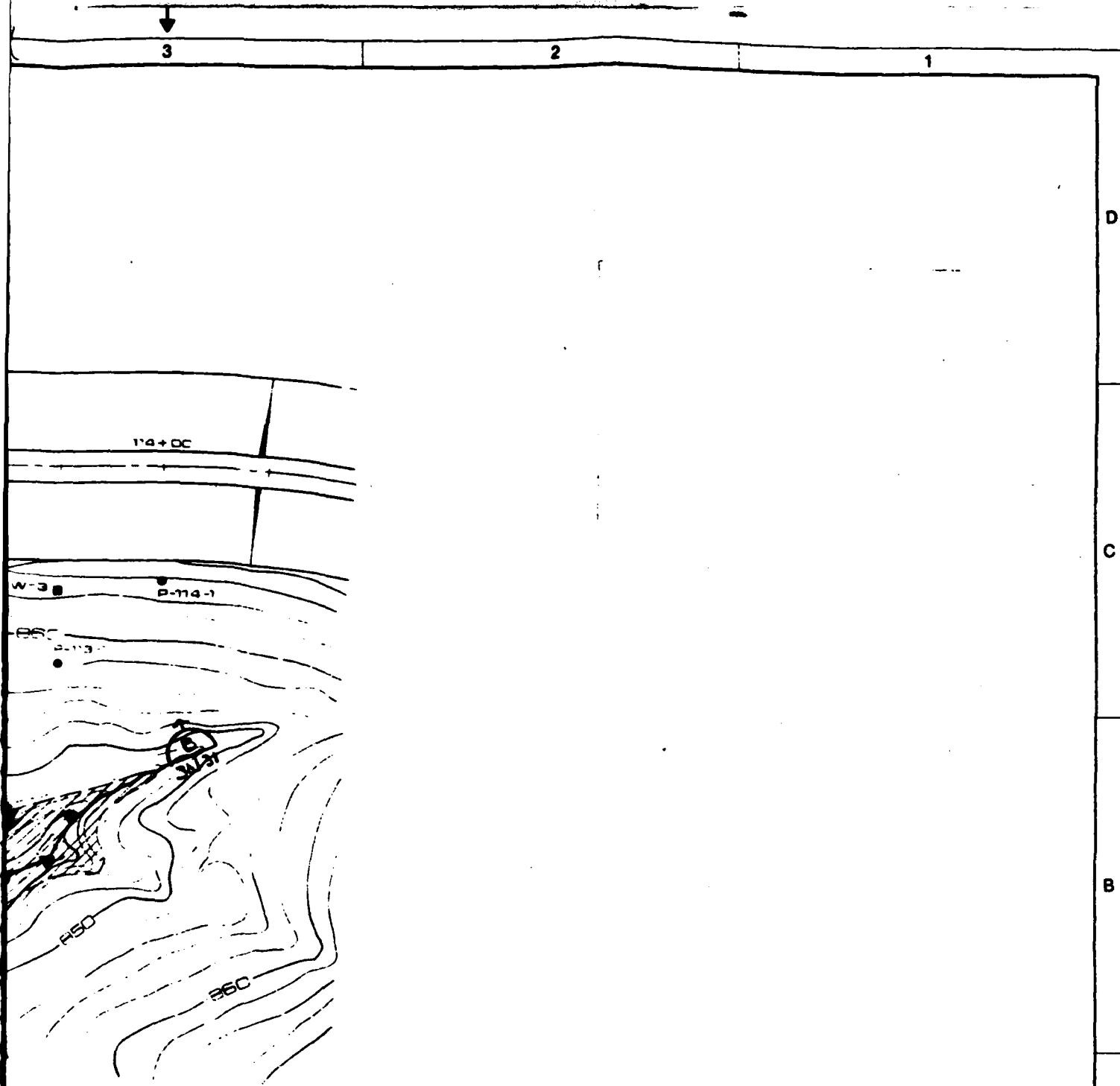
3/3

UNCLASSIFIED

F/G 13/2

NL



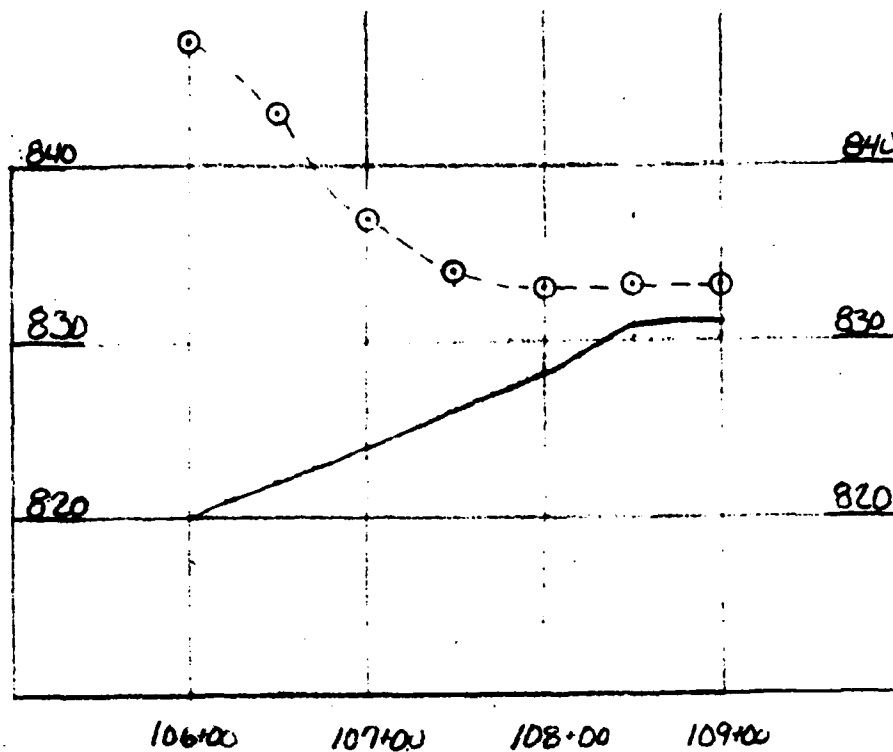


proposed pressure
relief wells

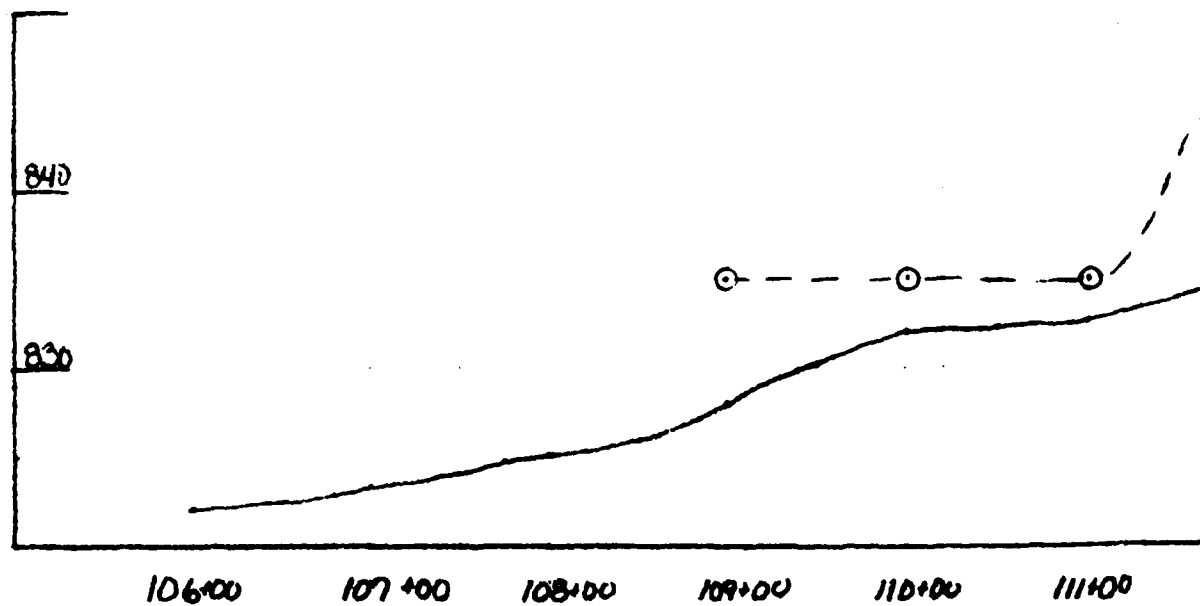
DATE	DESCRIPTION	DATE	APPROVED
LITTLE PLATTE RIVER, MISSOURI			
SMITHVILLE LAKE			
<i>Proposed Pressure</i>			
<i>Relief Wells - Plan</i>			
Desig. No. CORPS OF ENGINEERS KANSAS CITY DISTRICT Submitted		Scale: as shown U. S. ARMY Approved:	
DESIGNED BY _____	CHECKED BY _____	RECOMMENDED BY _____	APPROVED BY _____
			RP-3-1704

APPENDIX A PLATE NO. 30

2



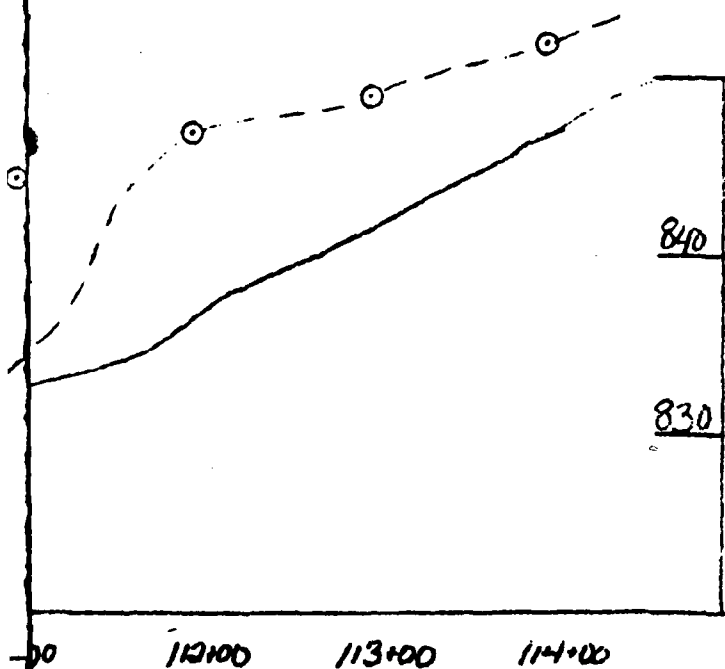
Profile of bottom
ELEVATION OF
HORIZONTAL PERVIOUS BLANKET



Profile ALONG DOWNST
AND DOWNSTREAM

15-1-7

○ Piezometric Level
PROJECTED FOR
SPILLWAY CREST
W/ TEST RELIEF
Wells
(see plate 22)

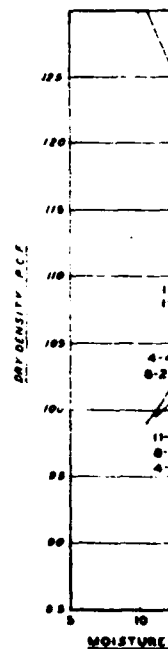
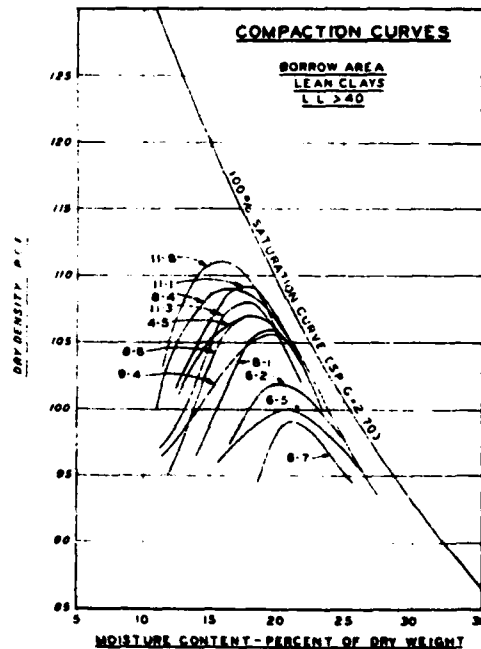
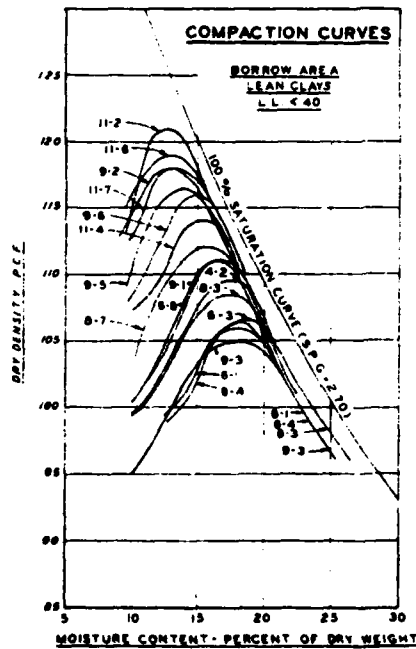
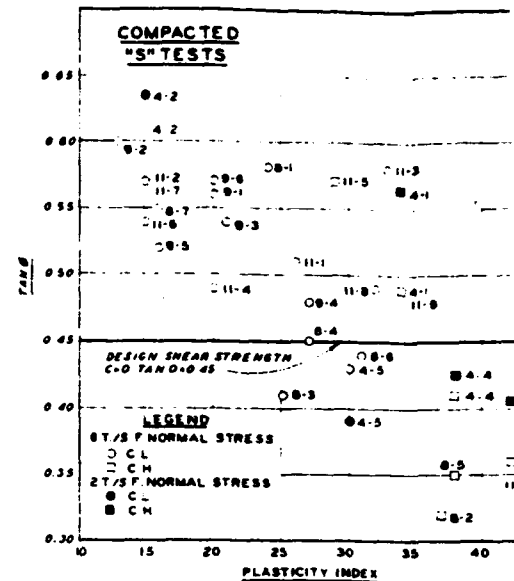
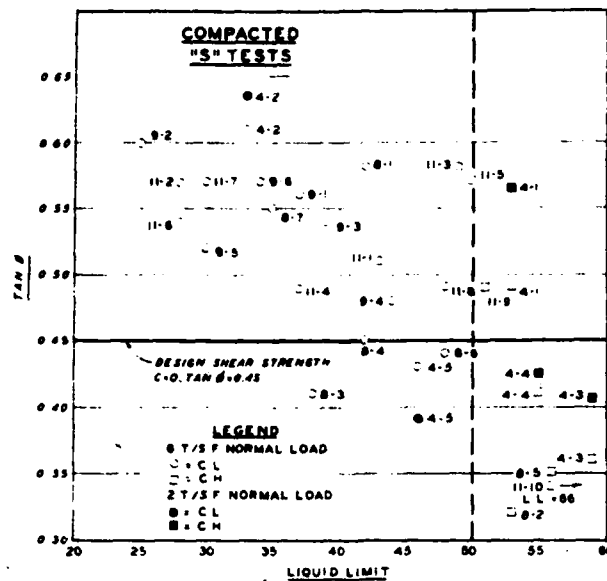


SMITHVILLE DAM
LEFT ABUTMENT
PROPOSED PRESSURE RELIEF
WELL LOCATION
PROFILE
APPENDIX A RP-3-1705
PLATE 31 JULY 1984

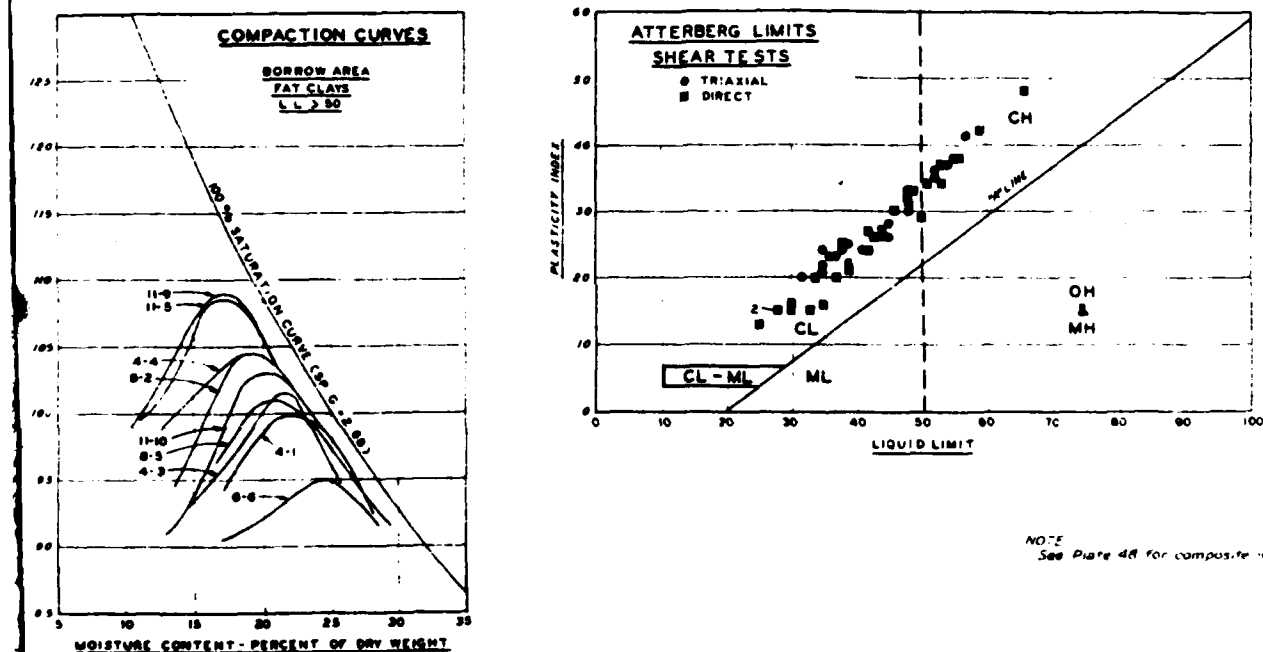
DOWNSTREAM TOE
HEAVY DRAW



2



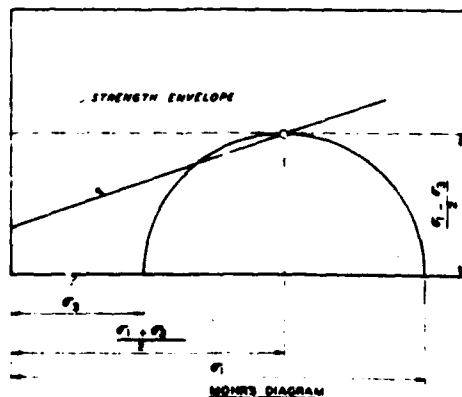
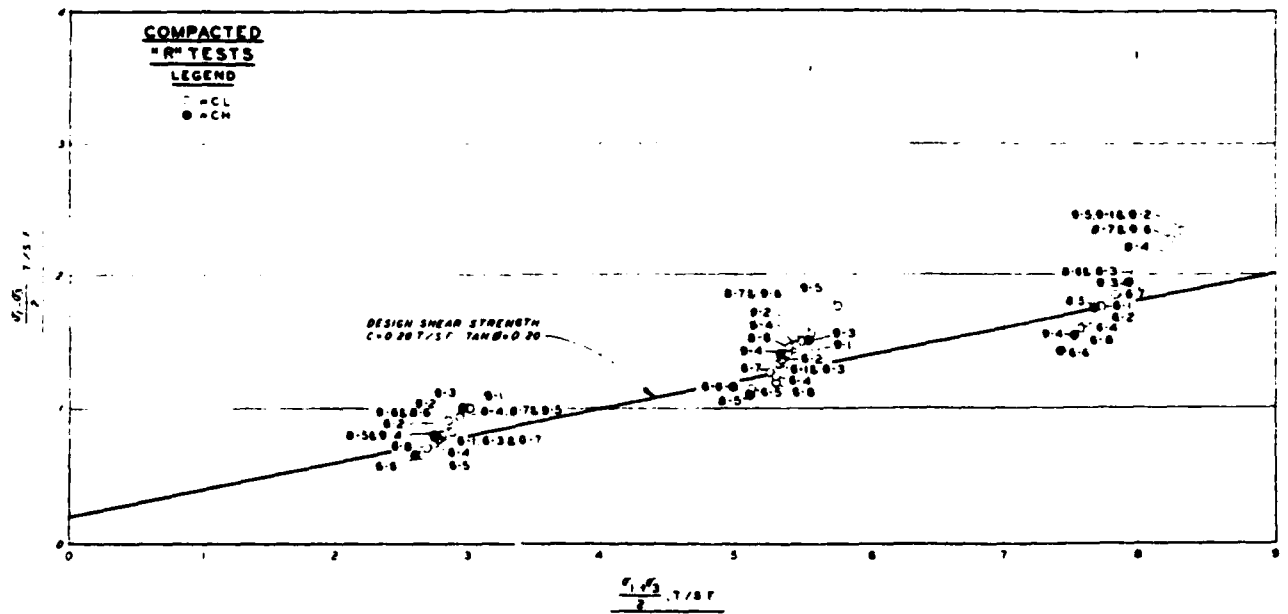
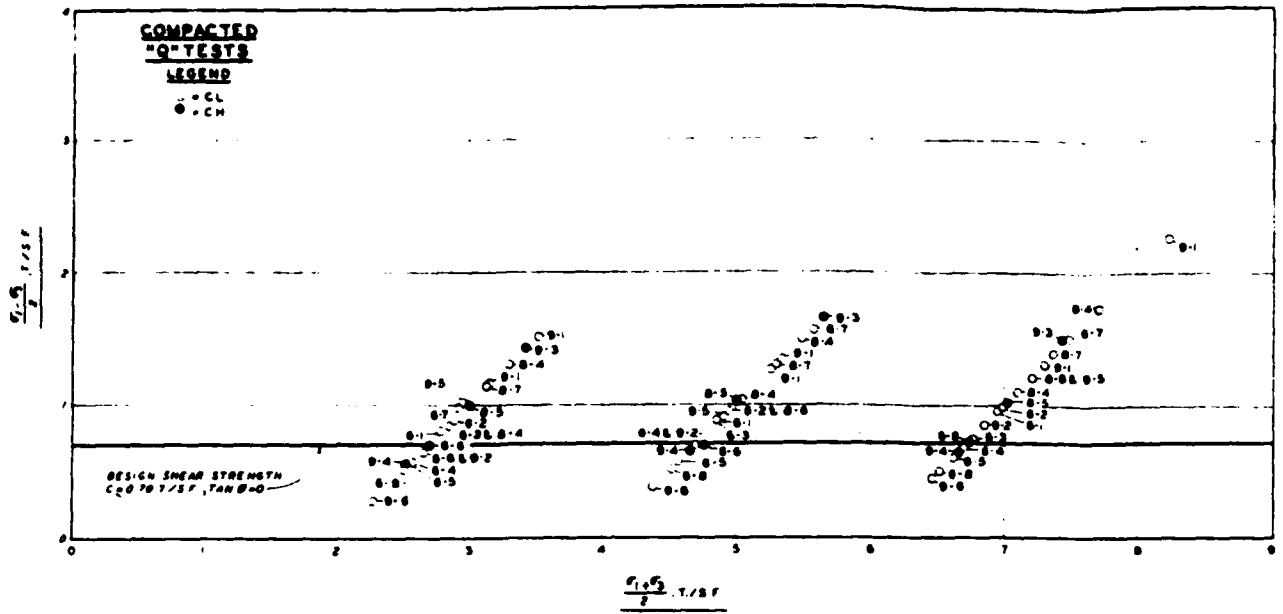
15-1-7



NOTE
See Plate 48 for composite identification.

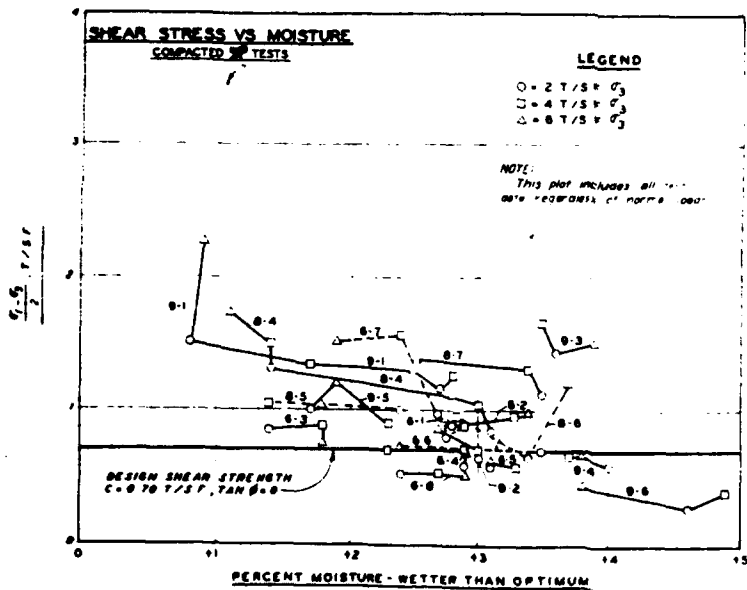
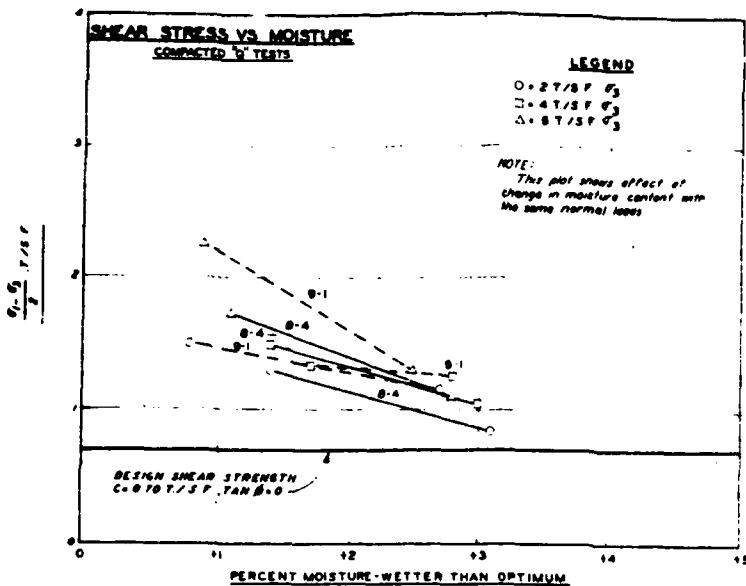
SYN	DESCRIPTION REVISIONS	DATE	APP'D
PLATTE RIVER BASIN SMITHVILLE RESERVOIR LITTLE PLATTE RIVER TEST DATA SUMMARY COMPACTED EMBANKMENT MATERIALS			
In 2 sheets	Sheet No 1	Scale, as shown	
CORPS OF ENGINEERS KANSAS CITY DISTRICT		U. S. ARMY OCTOBER 1970	
Submitted <i>[Signature]</i>	Recommended <i>[Signature]</i>	Approved <i>[Signature]</i>	
CHIEF ENGINEERING DIVISION KANSAS CITY DISTRICT	CHIEF MATERIALS DIVISION KANSAS CITY DISTRICT	CHIEF TRAINING DIVISION KANSAS CITY DISTRICT	
DESIGNED BY L.E.H.	LABORATORY BY S.R.F.	DATE OCT 10 1970	

RP-3-1706 APPENDIX A PLATE NO 32



1-7

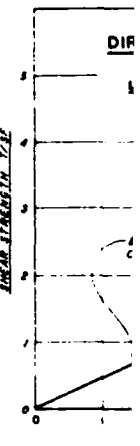
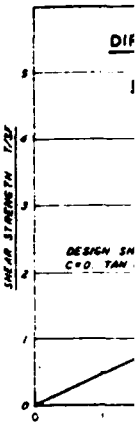
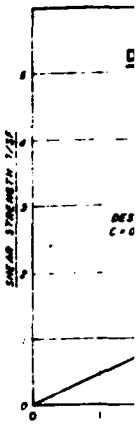
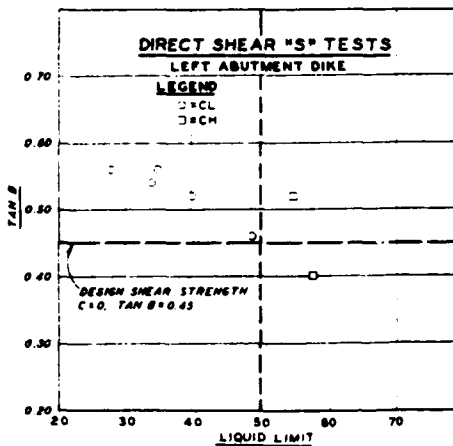
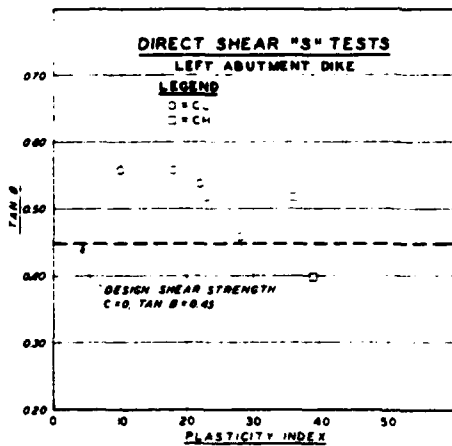
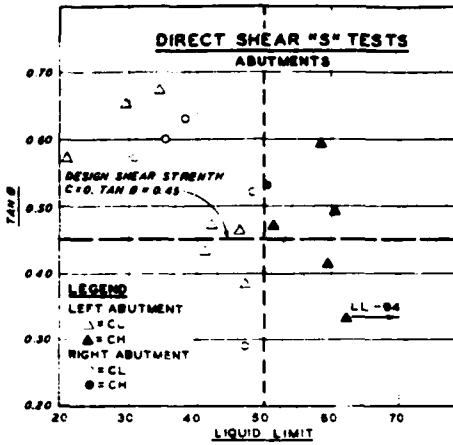
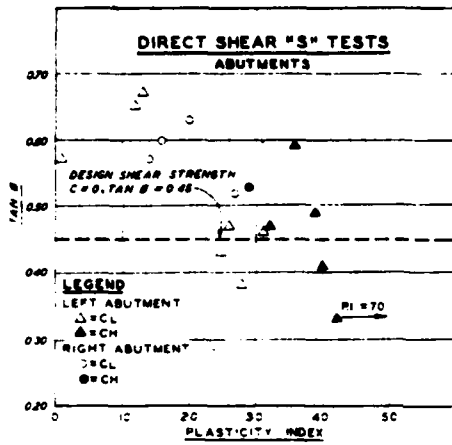
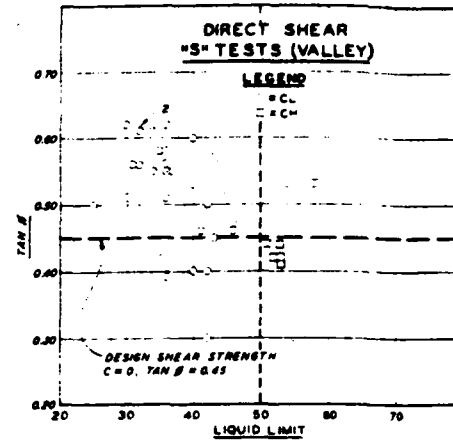
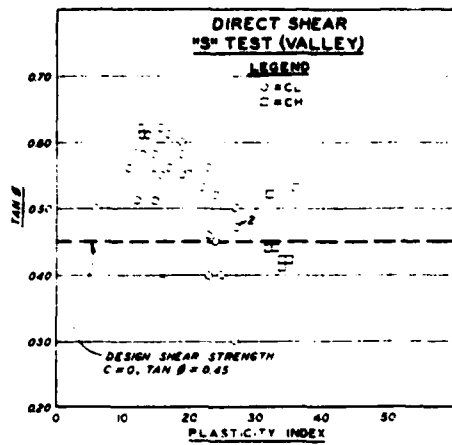
1



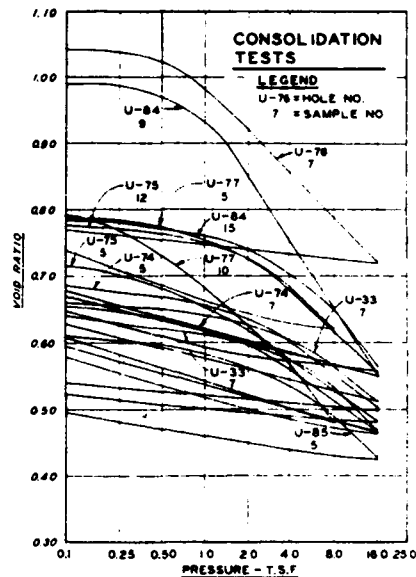
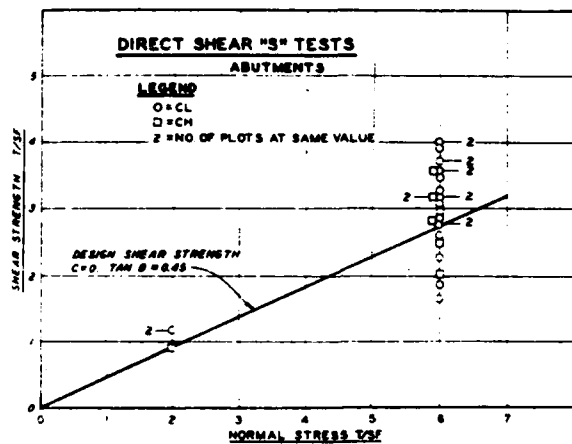
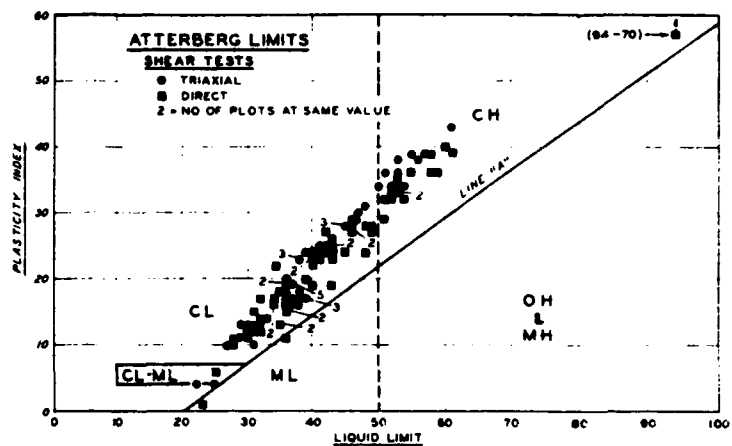
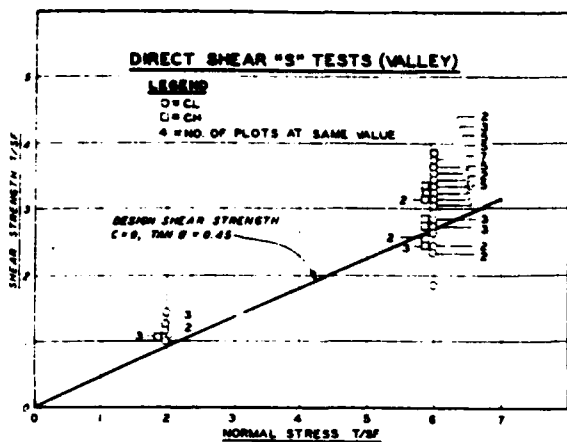
NOTE:
See Plate 48 for combined identification

SYN	DESCRIPTION REVISIONS	DATE	APP'D
	PLATTE RIVER BASIN		
SMITHVILLE RESERVOIR			
LITTLE PLATTE RIVER			
TEST DATA SUMMARY			
COMPACTED EMBANKMENT MATERIALS			
In 2 sheets	Sheet No 2	Scale as shown	
CORPS OF ENGINEERS		U. S. ARMY	
KANSAS CITY DISTRICT		OCTOBER 1970	
Submitted	Recommended	Approved	
COMPILED BY: RMB	CHECKED BY: RCF	FILE NO.	
LEM	DM-10		

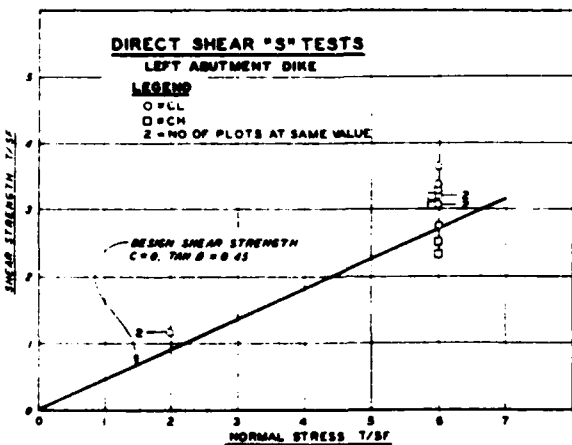
RP-3-1707 APPENDIX A PLATE NO. 33



15-1-7



CONSOLIDATION TEST SUMMARY											
HOLE NO.	DEPTH	LL	P ₀	P ₁	P ₂	C _c	C _u (IN ² /MIN)				
			%	T.S.F.	T.S.F.		2 T.S.F.	4 T.S.F.	8 T.S.F.	16 T.S.F.	
U-33	14.6-16.6	43	10	3.5	18	0.049	0.041	0.035	0.035		
U-33	14.6-16.6	43	10	3.1	19	—	0.098	0.041	0.035		
U-74	8.0-9.6	32	55	2.7	20	—	—	—	—		
U-74	12.2-13.9	34	80	2.1	19	—	—	0.164	0.131		
U-75	8.0-9.9	49	55	2.8	37	—	—	0.031	0.017		
U-75	8.0-9.9	45	55	1.9	23	—	—	0.035	—		
U-75	22.0-23.9	38	11	2.3	23	—	—	0.189	—		
U-76	12.3-13.9	38	55	0.8	24	—	—	—	—		
U-77	17.9-19.0	28	90	—	172	—	—	—	—		
U-84	16.0-17.9	28	90	1.0	38	—	—	0.390	—		
U-84	26.0-28.9	40	11	4.1	33	—	—	—	0.350		
U-85	8.3-9.0	33	55	3.0	23	—	—	—	0.150		



SYM _____ DESCRIPTION _____ DATE _____ APP. _____

REVISIONS _____

PLATTE RIVER BASIN

SMITHVILLE RESERVOIR

LITTLE PLATTE RIVER

TEST DATA SUMMARY

FOUNDATION OVERBURDEN

In 3 sheets Sheet No. 1 Scale as shown

CORPS OF ENGINEERS OCT. 1970

KANSAS CITY DISTRICT

Submitted _____ Recommended _____

Checked by _____

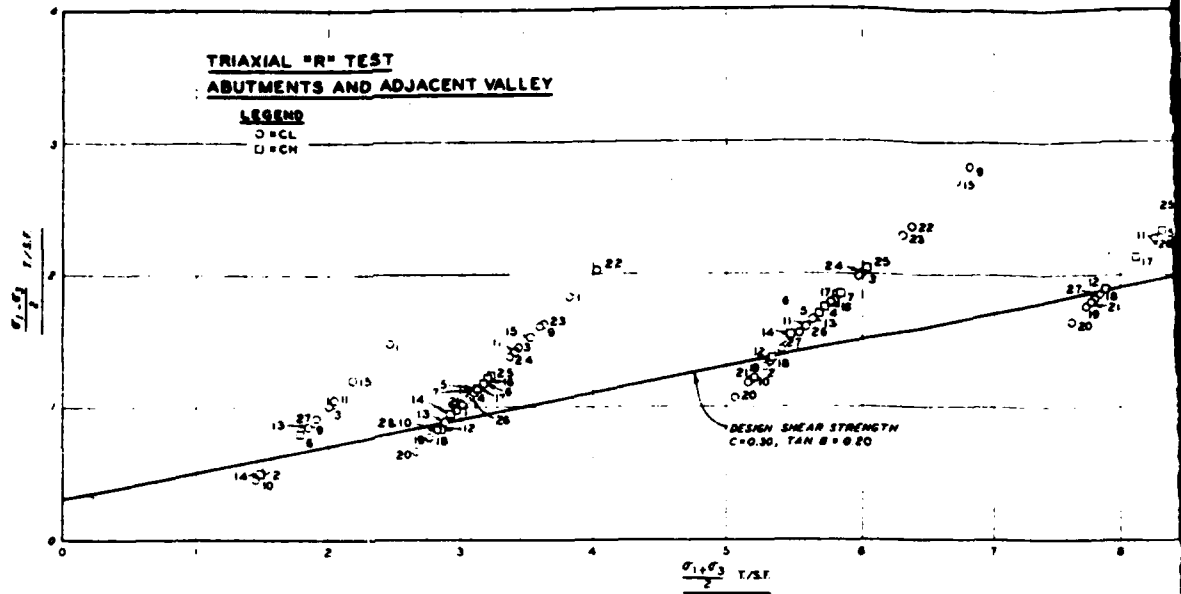
Reviewed by _____

Approved by _____

DATE _____

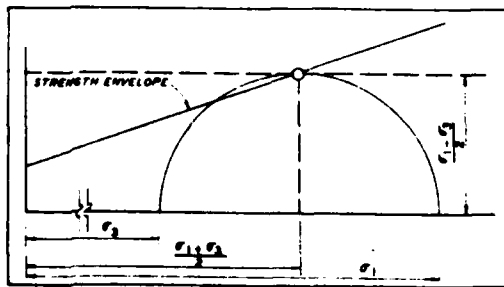
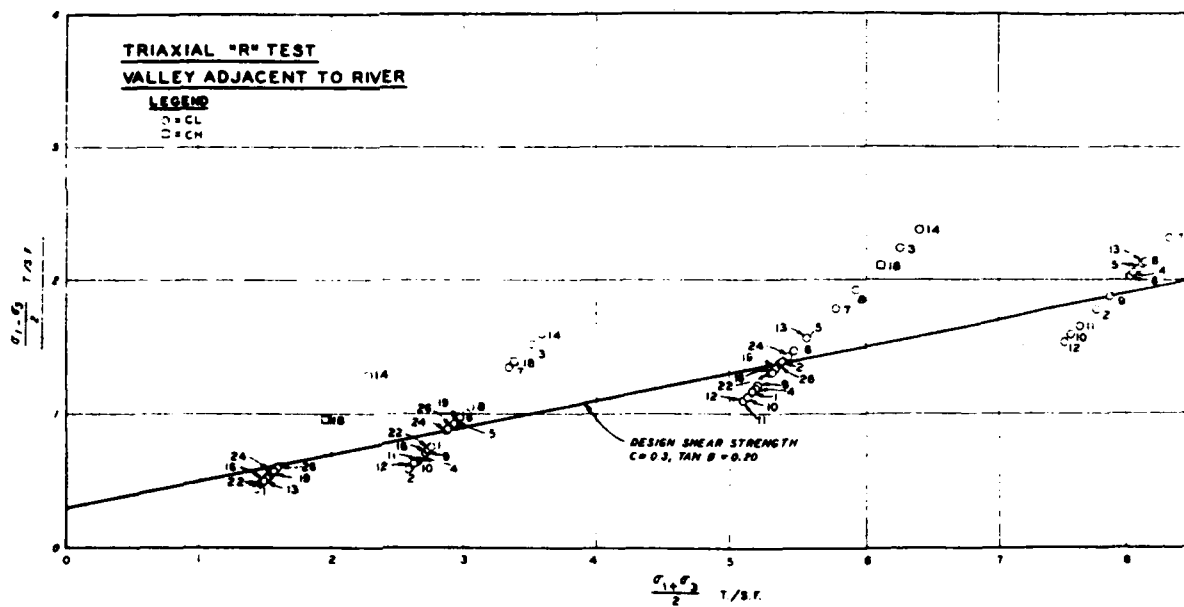
**TRIAXIAL "R" TEST
ABUTMENTS AND ADJACENT VALLEY**

LEGEND
○ = CL
□ = CH



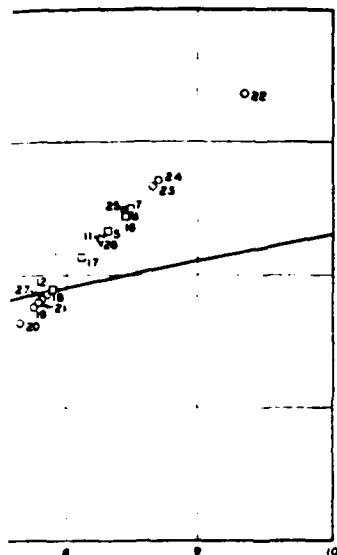
**TRIAXIAL "R" TEST
VALLEY ADJACENT TO RIVER**

LEGEND
○ = CL
□ = CH

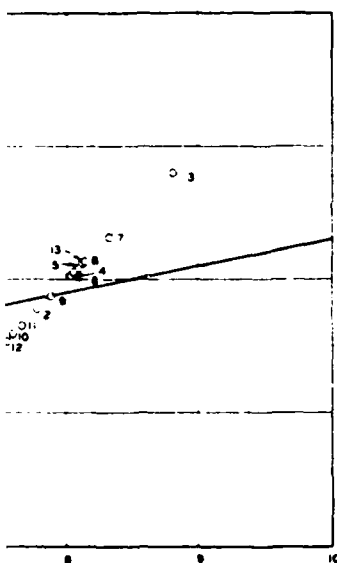


MOHR'S DIAGRAM

15-1-7



"R" TEST DATA						
ABUTMENTS AND ADJACENT VALLEY						
SYMBOL	HOLE NO.	SAMPLE DEPTH	CLASSIFICATION		RANGE	
			SYMBOL	PI	MIN	MAX
1	4C-29	11.0-12.9	CL	58	27.2-106.4	25.1-114.5
2	4C-31	7.1-8.7	CL	41	25.4-114.5	25.1-114.5
3	4C-31	10.4-20.0	CL	56	25.4-114.5	25.1-114.5
4	4C-31	4.2-5.9	CL	57	25.4-114.5	25.1-114.5
5	4C-31	6.1-7.9	CL	58	25.4-114.5	25.1-114.5
6	4C-31	8.7-10.0	CL	57	25.4-114.5	25.1-114.5
7	4C-31	8.2-10.0	CL	56	25.4-114.5	25.1-114.5
8	4C-31	10.0-11.9	CL	57	25.4-114.5	25.1-114.5
9	4C-31	10.0-11.9	CL	57	25.4-114.5	25.1-114.5
10	4C-31	20.0-21.9	CL	57	25.4-114.5	25.1-114.5
11	4C-31	2.0-3.9	CL	57	100.4-104.1	98.7-105.8
12	4C-31	2.0-3.9	CL	61	98.4-104.1	98.7-105.8
13	4C-31	0.0-1.9	CL	25	101.4-104.1	101.4-104.1
14	4C-31	0.0-1.9	CL	25	101.4-104.1	101.4-104.1
15	4C-31	0.0-1.9	CL	25	101.4-104.1	101.4-104.1
16	4C-31	4.1-5.9	CL	43	98.4-104.1	98.7-105.8
17	4C-31	6.0-7.9	CL	43	98.4-104.1	98.7-105.8
18	4C-31	8.0-9.9	CL	43	98.4-104.1	98.7-105.8
19	4C-31	10.0-11.9	CL	43	98.4-104.1	98.7-105.8
20	4C-31	12.0-13.9	CL	43	98.4-104.1	98.7-105.8
21	4C-31	22.0-23.9	CL	43	98.4-104.1	98.7-105.8
22	4C-31	0.0-1.9	CL	25	98.4-104.1	98.7-105.8
23	4C-31	2.0-3.9	CL	25	98.4-104.1	98.7-105.8
24	4C-31	4.0-5.9	CL	25	98.4-104.1	98.7-105.8
25	4C-31	0.0-1.9	CL	25	98.4-104.1	98.7-105.8
26	4C-31	4.2-5.9	CL	25	98.4-104.1	98.7-105.8
27	4C-31	6.0-7.9	CL	25	98.4-104.1	98.7-105.8



"R" TEST DATA						
VALLEY ADJACENT TO RIVER						
SYMBOL	HOLE NO.	SAMPLE DEPTH	CLASSIFICATION		RANGE	
			SYMBOL	PI	MIN	MAX
1	4C-27	14.0-15.9	CL	58	98.4-104.1	98.7-105.8
2	4C-27	2.0-3.9	CL	58	98.4-104.1	98.7-105.8
3	4C-27	4.0-5.9	CL	58	98.4-104.1	98.7-105.8
4	4C-27	6.0-7.9	CL	58	98.4-104.1	98.7-105.8
5	4C-27	8.0-9.9	CL	58	98.4-104.1	98.7-105.8
6	4C-27	10.0-11.9	CL	58	98.4-104.1	98.7-105.8
7	4C-27	12.0-13.9	CL	58	98.4-104.1	98.7-105.8
8	4C-27	2.0-3.9	CL	57	98.4-104.1	98.7-105.8
9	4C-27	4.0-5.9	CL	57	98.4-104.1	98.7-105.8
10	4C-27	6.0-7.9	CL	57	98.4-104.1	98.7-105.8
11	4C-27	8.0-9.9	CL	57	98.4-104.1	98.7-105.8
12	4C-27	10.0-11.9	CL	57	98.4-104.1	98.7-105.8
13	4C-27	12.0-13.9	CL	57	98.4-104.1	98.7-105.8
14	4C-27	0.0-1.9	CL	25	98.4-104.1	98.7-105.8
15	4C-27	2.0-3.9	CL	25	98.4-104.1	98.7-105.8
16	4C-27	4.0-5.9	CL	25	98.4-104.1	98.7-105.8
17	4C-27	6.0-7.9	CL	25	98.4-104.1	98.7-105.8
18	4C-27	8.0-9.9	CL	25	98.4-104.1	98.7-105.8
19	4C-27	10.0-11.9	CL	25	98.4-104.1	98.7-105.8
20	4C-27	12.0-13.9	CL	25	98.4-104.1	98.7-105.8

SYM _____ DESCRIPTION _____ DATE _____ APP'D _____

REVISIONS _____

PLATTE RIVER BASIN

SMITHVILLE RESERVOIR

LITTLE PLATTE RIVER

TEST DATA SUMMARY

FOUNDATION OVERBURDEN

In 3 sheets Sheet No 3 Scale as shown

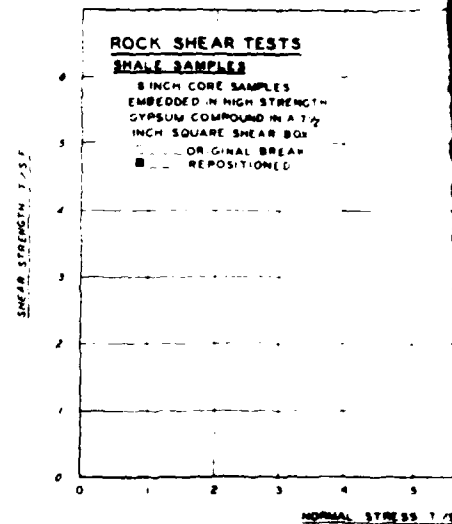
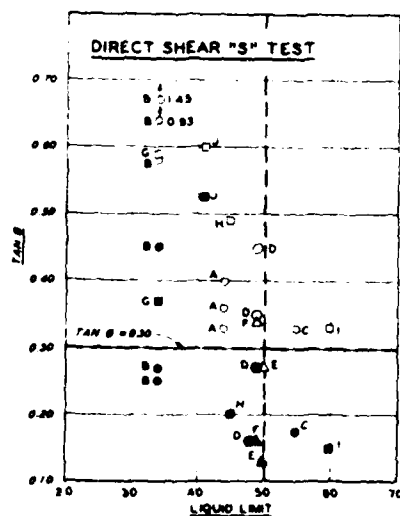
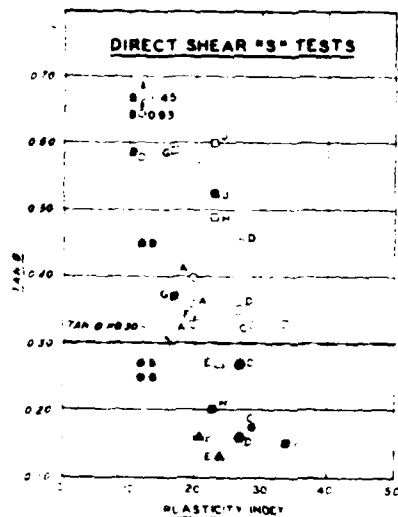
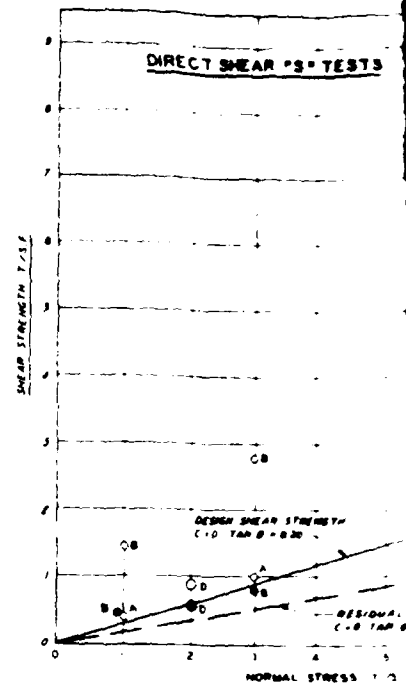
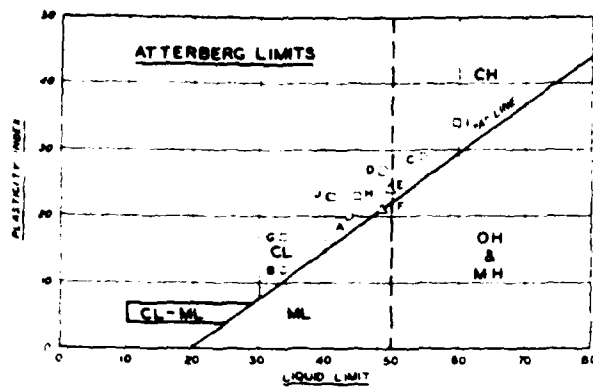
CORPS OF ENGINEERS OCT 5 1970

KANSAS CITY DISTRICT

Submitted by _____ Recommended by _____

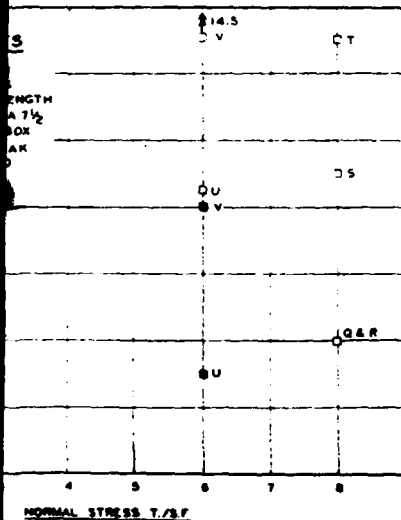
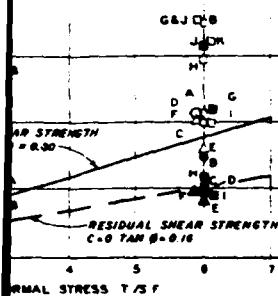
Checked by _____

W.W.B. C.F.L. R.K.B. DM 10



15-1-7

LEO



DIRECT SHEAR "S" TEST DATA									
SYMBOL	SAMPLE NO.	SAMPLE DEPTH	CLASSIFICATION		RANGE			GEOL.	
			LETTER	PI	INITIAL DRY DENSITY	INITIAL MOISTURE	PERCENT SATURATED		
			VALUES						
A	UC-27-26	52.4-52.8	CL	44	20	106.0-107.5	17.5-19.5	8-18	CL
B	UC-27-35	54.5-55.5	CL	38	12	139.5	8.5	9	CL
C	U-102-23	46.0-47.4	CL	55	29	104.0	23.0	6	CL
			SPILLAGE						
D	UUC-52-1	5.0- 6.9	CL	45	27	102.5-106.0	19.5-21.5	88-95	BONNER SPRINGS
			ST. ABUTMENT						
E	U-86-8	14.0-15.9	CM	50	24	103.0	23.5	98	ISLAND CREEK
F	U-86-9	16.0-17.7	CL	49	21	109.0	20.0	94	ISLAND CREEK
			OUTLET WINGS						
G	CC-103-1	28.7-30.0	CL	34	17	113.0	18.5	90	CHANNING
H	CC-103-2	30.0-31.7	C	41	25	101.0	24.0	97	CHANNING
I	CC-104-1	17.7-18.5	CM	60	38	108.0	23.5	94	CHANNING
J	D-105-1	22.0-23.7	CL	41	23	108.5	20.0	94	CHANNING
K	C-146-5	26.4-28.0	C	50	32	113.0	12.5	95	CHANNING CREEK
L	C-146-8	30.5-32.1	C	57	34	117.0	12.5	95	CHANNING CREEK
M	C-146-9	32.5-34.1	C	58	34	132.0	12.5	95	CHANNING CREEK
N	C-146-11	35.5-36.5	C	56	35	126.5	12.5	95	CHANNING
O	C-146-12	36.5-38.0	C	56	35	126.5	12.5	95	CHANNING
P	C-146-13	38.4-40.8	C	56	35	134.0	12.5	95	CHANNING

ROCK SHEAR TEST DATA						
OUTLET WORKS - SHALES						
SYMBOL	SAMPLE NO.	SAMPLE DEPTH	Dry DENSITY	MOISTURE	WATER CONTENT BY GRAVITY (%)	GEOLOG. NUMBER
Q	DC-101-2	30-31	132		0.0	CHANDLER
R	DC-101-3	46-47	83		0.0	UNIVERSITY
S	DC-102-2	33-34	131	10.0	0.0	CHANDLER
T	DC-102-3	32-33	119	7.0	0.0	CHANDLER
U	C-100-1A	40-41-2	124	11.0	0.0	CHANDLER

<u>MAXIMUM SHEAR STRESS</u>	<u>RESIDUAL SHEAR STRESS</u>	<u>LOCATION</u>
○	●	VALLEY
○	●	SPILLWAY
△	▲	RIGHT ABUTMENT
□	■	OUTLET WORKS

SYM	DESCRIPTION	DATE	APP D
	REVISIONS		

PLATTE RIVER BASIN
SMITHVILLE RESERVOIR
LITTLE PLATTE RIVER

TEST DATA SUMMARY FOUNDATION SHALES

In 1 sheet
CORPS OF ENGINEERS
KANSAS CITY DISTRICT

Sheet No. 1

Scale: as shown
U S ARMY
OCTOBER 1970

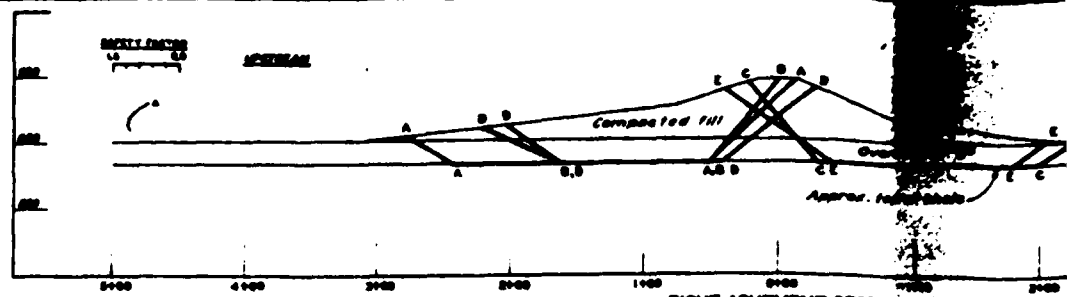
ASPP

Recommended
GP Gordon
DATE FORWARDED TO UNIT: 20
DISCLOSURE TO:
R.G.F. 0

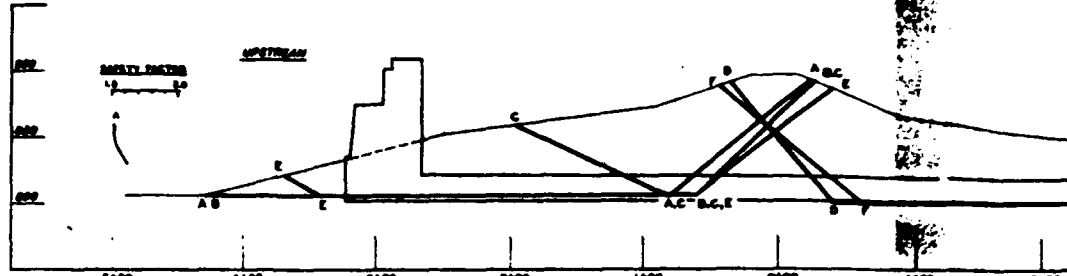
Approved: _____
Special Agent in Charge

R. K. B. C. F. L. R. G. F. DM-10

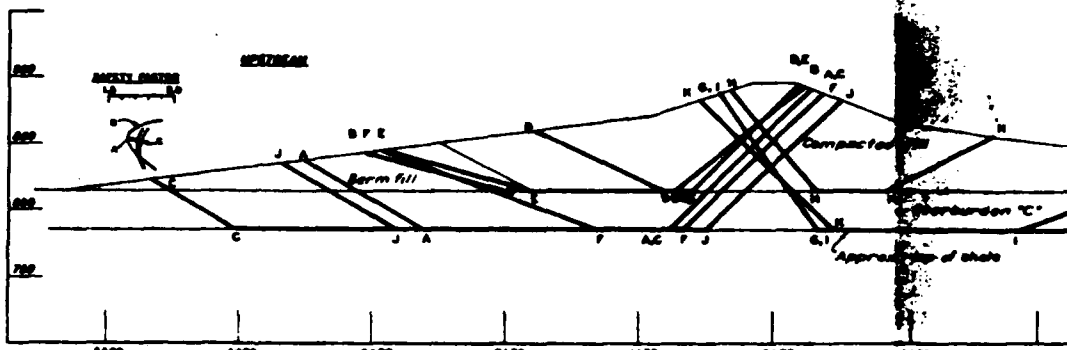
RP-3-1710 APPENDIX A PLATE NO 36



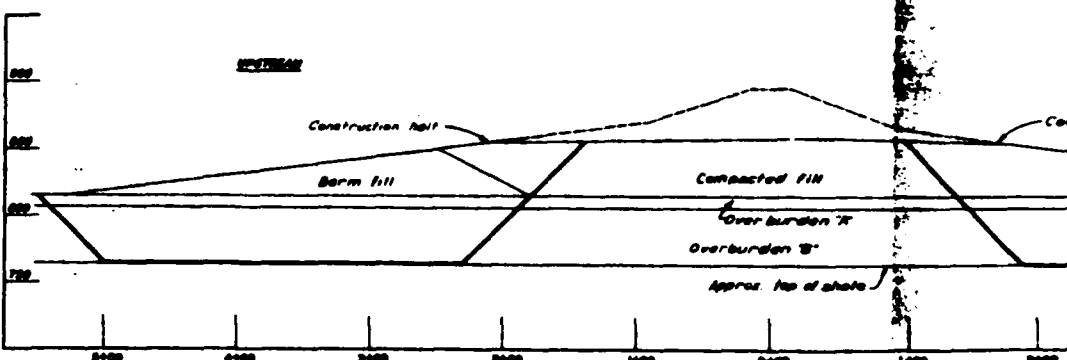
RIGHT ABUTMENT SECTION



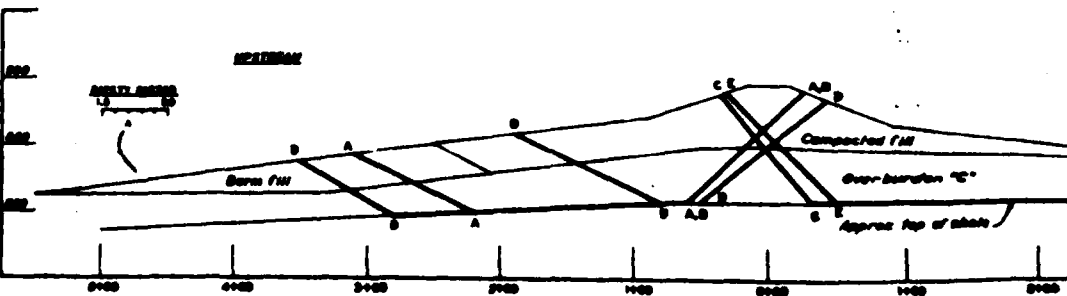
CONDUIT SECTION



VALLEY SECTION - RIGHT SIDE OF VALLEY

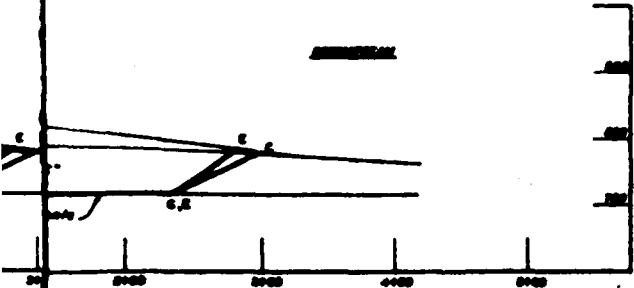
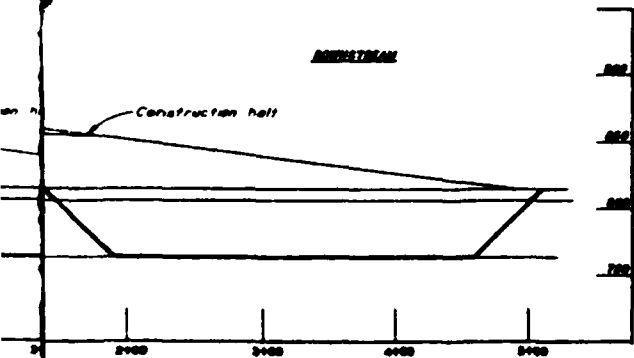
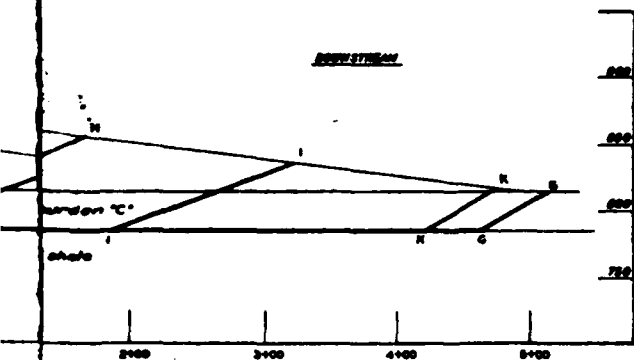
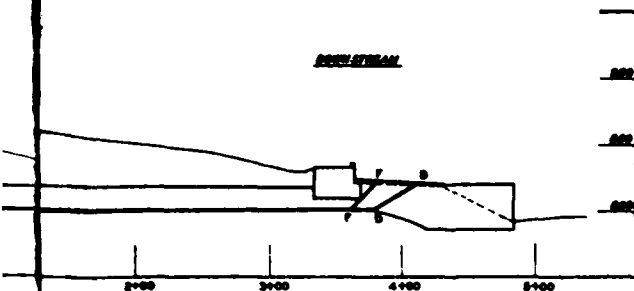
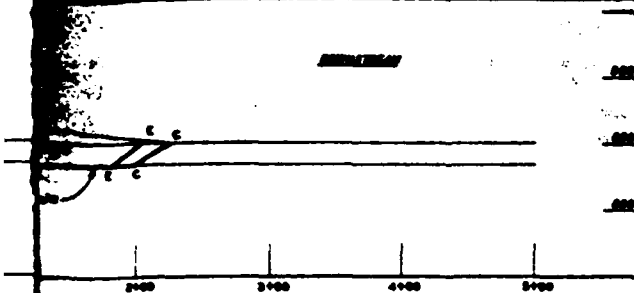


VALLEY SECTION - LEFT SIDE OF VALLEY



LEFT ABUTMENT SECTION

15-1-7



RIGHT ABUTMENT SECTION			
CASE	SURFACE	POOL	S.F.
Partial Pool	A	865	1.62 (1)
Sudden Drawdown	B	865.2	1.31
"	B	864.2	1.19
Steady seepage	C	860.2	1.53 (1)
Construction	D		2.00
"	E		1.67

CONDUIT SECTION			
CASE	SURFACE	POOL	S.F.
Partial Pool	A	850	1.51
Sudden Drawdown	B	850.2	1.50
"	B	849.2	1.25
"	C		1.25
Steady seepage	D	840.2	1.57
Construction	E		1.70
"	F		1.67

VALLEY SECTION - RIGHT SIDE OF VALLEY			
CASE	SURFACE	POOL	S.F.
Partial Pool	A	850	1.70 (1)(2)
"	B	850	1.66
"	C	845	1.74 (1)
Sudden Drawdown	D	845.2	1.24 (2)
"	E		1.27
"	F		1.44
"	G	844.2	1.15
"	H		1.20
"	I		1.53
Steady seepage	J	840.2	1.62 (1)(2)
"	K		1.70
"	L		1.64 (1)
Construction	M		1.24
"	N		1.24

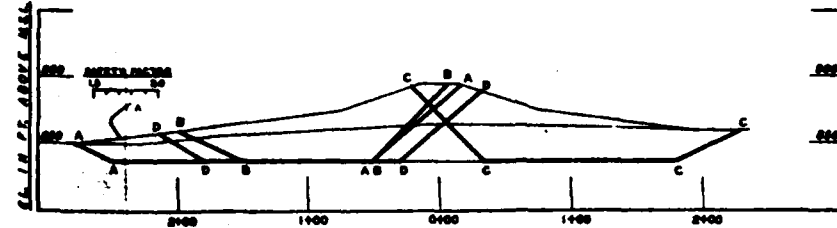
PHYSICAL SOIL CONSTANTS									
MATERIAL	UNIT WEIGHT		SOIL SHEAR STRENGTH						
	LB/FT ³		COHESION	ANGLE OF INTERNAL FRICTION	COHESION	ANGLE OF INTERNAL FRICTION	COHESION	ANGLE OF INTERNAL FRICTION	COHESION
Compacted fill	125	120	0.7	0.0	0.2	0.2	0.0	0.45	
Berm fill	115	110	0.3	0.0	0.2	0.2	0.0	0.3	
Overburden "A"	120	115	0.6	0.0	0.3	0.2	0.0	0.45	
Overburden "B"	120	115	0.3	0.0	0.3	0.2	0.0	0.45	
Overburden "C"	120	115	0.5	0.0	0.3	0.2	0.0	0.45	
Shale contact					0.4	0.3	0.0	0.30	
Shale contact/Residual					0.4	0.3	0.0	0.16	

REQUIRED SAFETY FACTORS	
CASE	S.F.
Construction	1.3
Sudden Drawdown from Maximum Surge	1.0
Sudden Drawdown from Spillway Crest	1.2
Partial Pool	1.3*
Steady Seepage	1.5*

* A higher safety factor was used when the shale-overburden contact formed part of the failure surface.

Pool Elevations	
Multipurpose Pool	864.2
Flood Control Pool	876.2
Spillway Crest	880.2
Maximum Surge	889.3

DIKE SECTION			
CASE	SURFACE	POOL	S.F.
Partial Pool	A	865	1.64 (1)
Sudden Drawdown	B	865.2	1.23
"	B	864.2	1.14
Steady seepage	C	860.2	1.53 (1)
Construction	D		1.55



DIKE SECTION

VALLEY SECTION - LEFT SIDE OF VALLEY			
CASE	SURFACE	POOL	S.F.
Partial Pool	A	850	1.70 (1)(2)
"	B	850	1.66
"	C	845	1.74 (1)
Sudden Drawdown	D	845.2	1.24 (2)
"	E		1.27
"	F		1.44
"	G	844.2	1.15
"	H		1.20
"	I		1.53
Steady seepage	J	840.2	1.62 (1)(2)
"	K		1.70
"	L		1.64 (1)
Construction	M		1.24
"	N		1.24

NOTES:

- (1) For these cases, a higher safety factor is required because the shale-overburden contact forms part of the failure surface.
- (2) These studies are the typical studies shown on Plate 1.

DATE	DESCRIPTION	DATE	APP'D.
	REVISIONS		

SMITHVILLE RESERVOIR

LITTLE PLATE RIVER

EMBANKMENT STABILITY ANALYSIS SUMMARY

In 1 sheet
 Corps of Engineers
 Kansas City District
 Drawn by
 Checked by
 Approved by
 Date
 Scale: as shown
 U. S. ARMY
 OCTOBER 1970

RP-3-1711

APPENDIX A PLATE NO. 39

APPENDIX B

APPENDIX B

OPERATION AND MAINTENANCE MANUAL
SMITHVILLE LAKE
LITTLE PLATTE RIVER, MISSOURI
APPENDIX V

EMBANKMENT CRITERIA AND
PERFORMANCE REPORT
SUPPLEMENT NO. 1

APPENDIX B

SMITHVILLE DAM
LEFT ABUTMENT SEEPAGE REPORT

DECEMBER 1984

DEPARTMENT OF THE ARMY
Kansas City District, Corps of Engineers
Kansas City, Missouri

OPERATION AND MAINTENANCE MANUAL
SMITHVILLE LAKE

APPENDIX V

EMBANKMENT CRITERIA AND
PERFORMANCE REPORT

SUPPLEMENT NO. 1

LEFT ABUTMENT REMEDIAL MEASURES

APPENDIX B

CONTENTS

<u>Section</u>	<u>Title</u>	<u>Page</u>
1.	Introduction	V-1-B-1
2.	Geology	V-1-B-1
	a. Glacial History	V-1-B-2
	b. Overburden	V-1-B-2
	c. Bedrock	V-1-B-3
3.	Field Investigations	V-1-B-3
	a. Field Reconnaissance	V-1-B-3
	b. Drilling and Instrumentation	V-1-B-4
	c. Findings	V-1-B-5
4.	Hydrogeologic Considerations	V-1-B-5
5.	Stability Consideration	V-1-B-6
6.	Present Conditions	V-1-B-7
	a. Downstream Seep Area	V-1-B-7
	b. Dike Seep Area	V-1-B-7
	c. Bowers Seep Area	V-1-B-7
7.	Conclusions	V-1-B-7
8.	Recommendations	V-1-B-8

Tables

<u>Table No.</u>	<u>Title</u>	
1	Instrumentation Schedule	V-1-B-9
2	Relief Drains; Schedule and Performance	V-1-B-11
3	Effect of Relief Drains on Piezometric Levels	V-1-B-12

CONTENTS--con.

Drawings

<u>Drawings</u>	<u>Title</u>	<u>File</u>
B-1	General Plan	RP-3-1721
B-2	Bowers Seep Area	RP-3-1722
B-3	Profiles - Left Abutment; Bowers Seep Area	RP-3-1723
B-4	Left Abutment Dam Axis Profile; Embankment Section - Station 110+00	RP-3-1724
B-5	Main Dike Section - Station 20+00	RP-3-1725
B-6	Geologic Column and Legend, Detached Borings, and Typical Installation Diagrams	RP-3-1726
B-7	Gradation Curves - Left Abutment Borings	RP-3-1727
B-8	Piezometric Level D-513(OW)	RP-3-1728
B-9	Piezometric Level D-514(PZ)	RP-3-1729
B-10	Piezometric Level DC-514A(PZ)	RP-3-1730
B-11	Piezometric Level D-515(OW)	RP-3-1731
B-12	Piezometric Level D-516(OW)	RP-3-1732
B-13	Piezometric Level DC-517(OW)	RP-3-1733
B-14	Piezometric Level D-518(OW)	RP-3-1734
B-15	Piezometric Level D-519A(PZ)	RP-3-1735
B-16	Piezometric Level A-520(PZ)	RP-3-1736
B-17	Piezometric Level A-521(PZ)	RP-3-1737
B-18	Piezometric Level D-521A(PZ)	RP-3-1738
B-19	Piezometric Level D-522(PZ)	RP-3-1739
B-20	Piezometric Level D-523(PZ)	RP-3-1740
B-21	Piezometric Level A-524(PZ)	RP-3-1741
B-22	Piezometric Level P-106-4	RP-3-1742
B-23	Piezometric Level P-110-6	RP-3-1743
B-24	Piezometric Level P-110-7	RP-3-1744
B-25	Piezometric Level P-110-8	RP-3-1745
B-26	Piezometric Level P-118-1	RP-3-1746
B-27	Piezometric Level P-20-1, P-20-2	RP-3-1747
B-28	Piezometric Level P-20-3, P-20-4	RP-3-1748

Smithville Seepage Report

1. Introduction.--Smithville Dam is located on the Little Platte River about 1 mile northeast of Smithville, Missouri. The project was authorized by the Flood Control Act of 1965 with construction starting in February 1974. The dam is a rolled earthfill embankment with two dike sections located in low areas high on the left abutment. Foundation material consists of valley alluvium, loess residual soils and glacial drift. Impoundment began in October 1979 but lake filling was delayed because of real estate acquisition problems. Multipurpose pool, elevation 864.2, was first reached in June 1982. The record high pool to date, elevation 869.4, occurred in April 1983 and again in April 1984. Flood control pool is elevation 876.2, while spillway crest pool is elevation 880.2.

Seepage was first observed at the project during a site visit in April 1983 when the record high pool to date was reached. A seep area had developed at the downstream toe of the main embankment at Station 110+00 near the base of the left abutment. The area was spongy and wet but flowing conditions were not observed. Piezometric levels recorded in the foundation overburden in the area were 3 feet above ground. In August of the same year Mr. Roy Bowers, the owner of a tract of farmland adjacent to the left dam abutment, reported a large wet area in his pasture about 3,000 feet downstream (west) of the main dike. According to Mr. Bowers, the area had been wet during spring and early summer but he had attributed this to heavy precipitation during that period. However, during the extended dry period of the summer of 1983 the area remained wet. Consequently, Mr. Bowers reported the condition to the project office at the lake. A subsequent field reconnaissance revealed three general areas of groundwater seepage: (1) the downstream seep area; (2) the dike seep area; and (3) the Bowers seep area (as shown on Plate B-1). An investigation of the seep areas was initiated to determine the causes, potential consequences, and possible remedial measures. It included 14 piezometers and observation wells installed in a line between the lake and the Bowers seep area and in the seep area itself. Five piezometers were installed near the downstream seep area. In addition, a series of ten relief drains were installed in April 1984 on both the Bowers property and adjacent Government property to provide a measure of seepage control and to determine the effect on the piezometer surface.

2. Geology.--Smithville Lake is located near the southern limit of the Dissected Till Plains Section of the Central Lowlands Physiographic Province. Major topographic features are the maturely to submaturely developed valleys of the Little Platte River, Crows Creek, and Camp Branch. Drainage patterns typical of northern Missouri are developed on thick glacial deposits resulting in gently rolling topography. Bedrock exposures are not common but can occasionally be found along the bases of valley walls of major streams. Maximum relief in the area is about 160 feet.

a. Glacial history.--Pleistocene glaciers extended into the northern region of Missouri approximately 750,000 years ago during the Kansas glacial episode and persisted for approximately 100,000 years. Glaciers may have also advanced into the area during the earlier Nebraskan episode. Both the Nebraskan and Kansan advances were from the north-northwest and are attributed to the Iowa ice lobe from the Keewatin ice center in Canada. Since the same general regions were traversed during both episodes, the content of resultant drift materials is similar and difficult to distinguish. The southern limit of glaciation is generally recognized as being slightly south of, and approximately parallel to, the present course of the Missouri River.

Pleistocene ice sheets have been compared in size and extent to those of the Antarctic which have an average central thickness of about 6,500 feet. Estimated thicknesses of marginal masses are of the order of 1,600 feet. Glacial erosion was primarily by abrasion and quarrying whereby slabs of frozen ground were sheared from and dragged forward over nonfrozen ground. Magnitudes of erosion were dependent upon the thickness and velocity of the ice mass, the nature of materials incorporated into the basal ice, and the character of surfaces overridden. Glacial sediments include nonstratified till and, less frequently, fluvio-glacial deposits of stratified silts, sands, and gravels. Drift of variable thickness has been deposited upon essentially flat lying Pennsylvanian bedrock and is the thickest in pre-Pleistocene topographic lows.

b. Overburden.--Overburden in the vicinity of the dam is of three principal types; alluvium, glacial drift, and loess. Alluvium occupies the valleys of the Little Platte River and its tributaries and generally consists of lean and fat clays overlying clayey sands and sandy clays with minor amounts of basal gravel. Thicknesses range from 25 to 50 feet. Upland areas are deposits of glacial drift thinly mantled with loess. In the left abutment area, the drift ranges in thickness up to 85 feet and generally consists of 20 to 60 feet of till overlying 5 to 25 feet of coarser outwash sediments. Till, in general, is composed of unsorted, unconsolidated (geologically), nonstratified sediments deposited directly by and underneath glacial ice masses and consists of heterogeneous, random mixtures of clay, silt, sand, gravel, cobbles, and boulders. The overburden above approximately elevation 845 in the left abutment is predominantly lean clay glacial till with scattered gravel and cobbles and occasional isolated silty sand lenses. Below elevation 845, the material is much more heterogeneous with considerable lateral and vertical variation. Throughout most of the abutment area, the upper 11 to 20 feet of this lower unit is generally silt, however, silty clay or lean clay was encountered in some borings at this horizon. Below the silt zone, the material is coarser and consists of sand, gravel, and cobbles generally with a significant amount of silt and clay. The coarser materials underlying the till are meltwater sediments deposited from advancing or retreating ice sheets. Loess overlying the till reaches a thickness up to 20 feet in the area. The maximum thicknesses occur on broad, gently sloping, interstream divides where erosion has been minimal.

c. Bedrock.--Near surface bedrock strata are of the Pennsylvanian System, Lansing and Kansas City Groups and consist of alternating beds of shale and limestone. A geologic column for the left abutment is shown on Plate B-6. The essentially horizontal configuration of the left abutment bedrock surface is the result of a pre-Pleistocene stream channel trending generally east-west through the abutment. It is one of two major channels mapped in the reservoir area which are part of the ancestral Missouri River drainage system prior to the advance of Pleistocene glaciers. The other is located several miles upstream of the dam in the reservoir area. As ice masses traversed the area, existing sediments were scoured away and near-surface bedrock strata subjected to shear forces induced by ice thrusts.

3. Field investigations.

a. Field reconnaissance.--The initial phase of the field investigations involved a field reconnaissance of the entire left abutment region downstream of the dam embankment and main dike. Three general areas of seepage were found (see Plate B-1). The first area, labeled the Bovers seep area, covers about 20 acres of land both on Government and private property, about 3,000 feet downstream of the dike and adjacent to the valley alluvium of Wilkerson Creek. The private property includes two parcels of land, one owned by Roy Bovers and the other by Roger Burnett/Helen Cutting. The area was characterized by numerous seeps: (areas are like numbered on Plate B-2) (1) in a large draw on Government property where natural springs are located; (2) at the northeast corner of Bovers' property and extending south along the fence line; (3) above the waterline of pond A, on the Burnett/Cutting property; and (4) in the small draw to the east of pond B, the larger pond at the south boundary of the seep area. Much of the northeast corner of Bovers' pasture bounded by the draw below pond A and the draw to the west was wet and extremely soft from ponding of seepage. Flow from the area was less than 10 gallons per minute. The second area, known as the dike seep area, is located immediately downstream of Highway DD at the downstream toe of the main dike. The quantity of flowing water from this area was quite small, less than 1 gallon per minute. The third area, the downstream seep area, had previously been identified downstream of the main embankment near Station 110+00. The area had remained soft and spongy but flowing water was not evident in the area adjacent to the embankment. However, flowing water was observed in the area downstream of the toe road.

During the extended dry period in the summer of 1983, farm ponds outside the outlined seep areas were either dry or very low. Those within the seep areas were at, or very near storage capacity, being constantly fed by groundwater seepage. Also, several mature trees located in the large draw on the north side of the Bovers seep area died, apparently as a result of intolerance to the elevated zone of saturation.

In connection with the field reconnaissance, several persons familiar with the area either previous to or during construction of the dam were contacted. They related that before impoundment some of the current seep areas contained natural springs or became unusually wet during periods of wet weather. Construction personnel recalled that during the 1974 flood during which the Little Platte River valley was inundated behind the partially

completed embankment, drinking water from an old well at the construction office located on the left abutment became very dirty. It apparently became dirty as a result of the valley flooding causing a significant increase in the groundwater level.

b. Drilling and instrumentation.--A drilling program was initiated to determine if the Bovers seep area was related to the impoundment of water in Smithville Lake. The program sought information to determine the type of materials present, to determine the existing pressure gradient from the lake to the seep area, to monitor uplift pressures at the seep areas, to monitor piezometric responses to pool fluctuations over an extended period of time, and to determine the top of bedrock surface contours in the left abutment.

Initially, a series of borings were completed in the fall of 1983 along a line between the lake and the Bovers seep area (D-513 through D-518 and DC-514A) (see Plate B-1 for location and Plate B-3 for profile). Borings were advanced by continuous churn drill sampling of overburden materials with a 6-inch diameter drive barrel. Representative samples were retained for laboratory soil classification and mechanical grain size analyses. In most borings, difficult drilling conditions resulted when cobbles and gravel were encountered near the bedrock surface. In order to positively identify the bedrock surface, some of the borings were continued into bedrock short distances with NX core barrels. Borings were completed either as observation wells monitoring the more permeable basal layer of the glacial drift (D-513(OW), DC-515(OW) through D-518(OW)) or as piezometers monitoring isolated zones in the basal layers to determine if a pressure gradient existed within the basal layer itself (D-514(PZ), DC-514A(PZ)). Monitoring devices were constructed of 2-inch PVC pipe and slotted well screen (see Plate B-6 for typical installation diagrams). Table B-1 provides a listing for all the instrumentation installed during the investigations.

Five additional borings were completed downstream of the main embankment centerline to supplement and verify existing data for use in determining the extent of the more permeable basal layer (P-106-4, P-110-6, P-110-7, P-110-8, P-118-1) (see Plate B-1 and Table B-1). Drilling and sampling methods were essentially the same as the initial series. NX core samples were obtained from three holes to accurately determine top of rock. Borings were completed as piezometers with tips isolated just above top of bedrock.

A final series of eight borings were completed in February and March 1984 in the Bovers seep area to determine the thickness and material types of the alluvial confining layer and to install piezometers to measure the uplift pressures present. Three were drilled with continuous, hollow stem flight augers (A-519, A-520, A-521). Two were advanced by continuous churn drill sampling with 6-inch/4-inch diameter drive barrels (D-519A, D-521A). (These two holes were completed in the same location as the auger holes because of the need to obtain samples from the hole after the water table was encountered and to install reliable devices.) Two were drilled with a 6-inch rockbit and sampled with either a 3-inch or 1-3/8-inch split-spoon (D-522, D-523). One was drilled with a hand auger (A-524). Piezometers were installed just above bedrock in all borings but two. In A-524(PZ), the tip was isolated 8 feet below ground surface in the low permeability confining layer. A piezometer could not be successfully installed in A-519 because suspended solids left from the drilling action in the hole plugged the tip.

c. Findings.--The basal layer of the glacial drift consists of up to 40 feet of a heterogeneous mixture of silt, and silty or clayey sands and gravels, which form a natural pervious zone. This zone contains between 10 to 30 percent fines passing the No. 200 sieve (see Plate B-7). The thickness of the zone ranges from 40 feet under the upland portion of the abutment to less than 10 feet towards the valleys (see profile on Plate B-3). The zone terminates or becomes very thin in the valleys having been eroded away and replaced by alluvium. The material is more gravelly and cobbly where it is thickest, and becomes more sandy as it thins. Similar materials were found to crop out in the bluffs along Crows Creek upstream of the dam prior to lake filling. At that time it was believed the more pervious zones of glacial materials were too discontinuous and erratic to become seepage problems after impoundment. However, now it appears the more pervious zones are continuous under the entire left abutment area. The natural pervious in combination with the overlying lean clays and silts and underlying tight bedrock form a confined aquifer system which is recharged from the lake. After initial entrance pressure losses, the piezometric pressure gradient is relatively flat. The pressure gradient steepens near the seeps (see Plate B-3). The seep areas are characterized by decreasing thickness of low permeability material and pockets or lenses of more pervious material extending to near the ground surface. Pressure levels remain above ground downstream from the seeps.

4. Hydrogeologic considerations.---Prior to impoundment, the basal layer of the left abutment glacial drift was a mostly saturated unconfined aquifer system whose main discharge points were the exposures along the banks of Crows Creek along with the valley alluvium and naturally occurring springs. In 1971, three borings (D-147, D-148, and D-149) had piezometers installed in them and were monitored to investigate the possibility of abutment seepage. Two piezometers were installed in each boring, one was isolated in the basal sands and gravels and the other was isolated in higher sand lenses in the drift. The borings, however, never fully penetrated the basal layer since refusal was reached 20 feet above top of rock. Thus, when dry readings for the lower piezometers were obtained, they were misleading. As stated earlier, during the 1974 flood the construction office noticed dirty drinking water which was most likely caused by a significant rise in the water level of their water supply well. The first link was established between the influence of the river stage and the water level in the basal layer.

When impoundment began, the exposed basal material along the banks of Crows Creek became submerged. The discharge area became the recharge area initially, completing saturation of the aquifer and then subjecting the confined aquifer system to a hydrostatic pressure head corresponding to the lake level. The pressure levels increased as the lake elevation rose to multipurpose pool. Pressure levels in the basal layer near the downstream base of the abutment increased to above ground level, providing enough vertical gradient to force seepage from the basal layer through the thinning confining layer. However, since the relatively impervious alluvium blocks the aquifer as it emerges from the abutment, seepage quantities are relatively low. Flow from natural springs increased. Since the aquifer has reached a saturated condition, changes in pool level produce rapid pressure changes in the confined aquifer system. Data obtained during pool rises in the spring of 1983 indicates up to a 50 percent response in piezometric levels to changes in pool level with higher responses closer to the lake entrance point. When

pressure levels change in the confined system, the quantity or seepage will change and the extent or seep areas will tend to change. The extent or seep areas is also dependent on the length of time the pool is at a given level as well as the climatic (temperature and rainfall) conditions at the time.

5. Stability considerations.--Using the information from boring logs and piezometric data, minimum safety factors against uplift were calculated with projected piezometric levels corresponding to flood control pool and a spillway crest pool. Projections were based upon a 50 percent increase in piezometric levels corresponding to pool increases. Safety factors were computed by dividing weight of the soil mass by the uplift pressures exerted on the mass. In the Bowers seep area, the minimum safety factor against uplift was calculated as 1.1, while the area immediately downstream of the dam at Station 110+00 the minimum safety factor approached 1.0. It was concluded that the uplift stability of both seep areas was questionable when the pool reached or exceeded flood control pool.

Actions taken in the downstream area are discussed in a subsequent paragraph. In the Bowers seep area, a series of ten relief drains were installed in April 1984 to relieve and control the seepage pressures and to determine their effect on piezometer levels. Drains were initially installed on 50-foot centers and constructed with 2-inch PVC slotted well screen (.020-inch slots), (RD-1 to RD-7). The last two drains installed, RD-5A and RD-5B, were located 25 feet on either side of the RD-5 which had the largest discharge and the greatest effect on the piezometric level of the initial drains installed. These last two drains were constructed with 4-inch PVC and .030-inch slotted well screen to allow for greater discharges while reducing the entrance velocity. Each screen was surrounded by a commercial filter pack to prevent migration of the natural material into the well and to increase the effective well radius. Typical installation diagrams for both the 2-inch and 4-inch drain are shown on Plate B-6. A schedule of the relief drains are shown in Table B-2. Relief drains installed on Government property, RD-1 through RD-3, were completed with a churn rig. An all terrain vehicle mounted CME 750 was used for installation of the relief drains on Bowers' property because of extremely wet field conditions. Table B-2 includes a schedule of relief drains along with the performance of individual drains.

The effect of the relief drains on the piezometric surface was significant. Drops in pressure levels in the piezometers located down-gradient from the drains varied from in excess of 7.5 feet in D-522(PZ) to over 5.5 feet in A-521(PZ) and D-521A(PZ). Pressure drops up-gradient were measured at over 2.2 feet in D-514(PZ) and DC-514A(PZ) with the amount of pressure reductions decreasing with increasing distance from the relief drains. Table B-3 summarizes the effect of relief drains on the actual and projected piezometric levels. Since the relief drains have been installed, pool fluctuations have caused little change in the piezometric surface in the general vicinity of the wells. Piezometers located near the lake remain unaffected by the relief drains.

A temporary collector system was installed in October 1984 to contain flows from the seven relief drains on Bowers' property. It consists of 4-inch PVC pipe and connections assembled together on top of the ground. The area is fenced off to prevent livestock damage. Discharge is directed into the nearest drainage ditch to the south (below pond A). The temporary collector system has significantly dried up the area.

6. Present conditions.

a. Downstream seep area.--Seepage in the downstream area was further investigated during late spring 1984. The investigation included a stability investigation along with measures to control seepage and reduce uplift pressures. A series of four test pressure relief wells were installed in the downstream left abutment area near Station 110+00. The test wells were successful in reducing the piezometric level in the foundation. The Left Abutment Stability Report, dated July 1983, summarized the field investigations, laboratory testing, stability studies, and set forth an interim solution of three pumped wells to reduce uplift pressures during high pools. It also proposed 12 additional pressure relief wells in the downstream toe area and through the downstream embankment slope as a permanent solution to control seepage and reduce uplift pressures. The relief wells and a buried collector system were scheduled to be installed during the fall or winter 1984.

b. Dike seep area.--The draw downstream of the main dike has been soft and wet since before construction of the project. The size of willows growing in the seep areas, show that some seepage was occurring well before the lake reached multipurpose pool. However, the two seep areas are located in the two low areas where the horizontal pervious blanket of the dike extends underneath the road embankment. Instrumentation installed in the dike foundation during construction responds to pool fluctuations, but the pressures at multipurpose pool are not high enough to cause seepage to exit at the ground surface at the toe. (See Plates B-5, B-27, and B-28.) Projected pressure levels at higher pools are above ground, but the thickness of the confining layer should prevent excessive seepage and uplift pressure problems.

c. Bowers seep area.--The relief drains have significantly reduced pressure levels and maintained the lowered levels during pool rises, but piezometric levels are still slightly above the ground surface (1.5 to 2.0 feet). Some surface seepage is still occurring. The groundwater table is still at the ground surface in several places. The amount of seepage is still great enough to cause flowing conditions in the following areas: (1) in the large draw on Government property where the influence of the lake has increased the flow of natural springs; (2) on Government property near the northeast corner of Bowers property; (3) above the waterline of pond A; and (4) in the small draw to the east of pond B (areas marked on Plate B-2). All of the flow crosses Bowers' property before entering Wilkerson Creek. The hillside remains damp in several places (labeled A, B, C, and D on Plate B-2). Erosion and headcutting in the draws leading to Wilkerson Creek are being aggravated by the constant flow from the relief drains, seeps, and natural springs, (labeled I, II, III on Plate B-2). Short term leases are being acquired on both properties.

7. Conclusions.

a. Downstream seep area.--The downstream seep area is directly affected and fed by the lake. Due to the potential embankment stability problems caused by this seepage it was the subject to the Left Abutment Stability Report, dated July 1984 which provided an analysis of the problem and proposed permanent remedial action. The remedial action which includes 12

pressure relief wells and a buried collector system is presently under construction. When completed, it will provide a comprehensive and permanent solution to the seepage conditions in this area.

b. Dike seep area.--There is no apparent direct connection between lake levels and the seepage in the dike area. The present seepage is believed to be originating from isolated sand lenses in the glacial drift in the dike foundation. This seep area was present prior to construction and has not changed significantly with the filling of the lake. However, because of piezometric response to the lake in the lower overburden, this area will continue to be monitored and possibly have some additional instrumentation installed. No remedial work is warranted.

c. Bowers seep area.--The existing seepage conditions on the Bowers property, the Burnett/Cutting property, and the adjoining Government lands are caused by and affected by the lake levels. The relief drains previously installed help to reduce excessive seepage pressures. However, the associated problems with large saturated areas and headcutting still exist. Although areas of instability during high lake levels would probably not have the potentially catastrophic consequences that similar areas immediately downstream of the dam would have, they could be significant, particularly on private property. Access and remedial actions in an emergency could be severely restricted. It is unfortunate the affected Bowers and Burnett/Cutting properties were not acquired during the original land acquisition for the Smithville Lake. Due to the larger and more complex seepage conditions in this area, an engineered total remedial solution is not cost effective since any boils and/or slides caused by high lake levels would not likely be catastrophic. Remedial work consisting of a buried collector system for the existing relief drains, an emergency access trail, and rock ditch construction to halt head cutting will be an adequate engineered solution to the seepage problems, provided the affected tracts of land are acquired by the Government. Government acquisition will insure the necessary access for monitoring, instrumentation, additional remedial work, and emergencies. The cost estimate for the minimal remedial work necessary consisting of the buried collector system, an emergency access trail, and the rock ditch protection is \$70,000.

8. Recommendation.--The 20 acres of Bowers property and 20 acres of the Burnett/Cutting property should be acquired by the Government. The minimal remedial work consisting of the buried collector system, an emergency access trail and rock ditch protection should be constructed. This area should be closely monitored, especially during high pools, and additional remedial work be performed as needed.

TABLE 1 Instrumentation Schedule

Number	Station	Range	Tip Elev.	Grd. Elev.	Tip Matl.	Installation Date	Type	Remarks
<u>EMBARKMENT</u> P-106-4	106+02	0+15 DS	807.9	894.9	Sly Grv Sd*	12-09-83	3/4" / 1.5' - .010"	
P-110-6	110+00	3+48 DS	808.1	838.1	Sly Sdy Grv	10-26-83	2" / 1' - .020"	Flowing hole
P-110-7	110+00	4+23 DS	809.5	832.5	Clay Grv Sd*	11-22-83	2" / 1' - .020"	Flowing hole
P-110-8	110+00	1+09 DS	811.0	863.0	Clay Sdy Grv*	11-17-83	3/4" / 1.5' - .010"	
P-118-1	118+00	1+00 DS	807.0	868.5	Sly Grv Sd	11-08-83	3/4" / 1.5' - .010"	
<u>ABUTMENT</u> D-513(OW)			810.0	851.8	Sly Grv Sd/ Sly Sdy Grv	09-21-83	2" / 10' - .020"	
D-514(PZ)			817.7	841.0	Clay Grv Sd/ Sly Sd	10-13-83	2" / 2' - .020"	Flowing hole
DC-514A(PZ)			809.9	841.4	Clay Grv Sd*	10-14-83	2" / 2' - .020"	Flowing hole
D-515(OW)			807.6	877.1	Sly Grv Sd/Sly Sd Grv/Clay Grv Sd	10-28-83	2" / 10' - .020"	
D-516(OW)			812.7	892.9	Sly Grv Sd/Sly Sdy Grv/Clay Grv Sd	10-03-83	2" / 10' - .020"	
DC-517(OW)			812.9	880.0	Sly Grv Sd	10-07-83	2" / 5' - .020"	
D-518 (OW)			823.3	896.3	Sly Grv Sd/Clay Grv Sd	10-20-83	2" / 5' - .020"	

TABLE 1 (Cont'd)

Number	Station	Range	Tip Elev.	Grd Elev.	Tip Matl.	Installation Date	Type riser / tip	Remarks
<u>BOWERS SEEP AREA</u>								
A-519			-	839.5	-	-	-	PZ not installed, suspended solids in hole
D-519A(PZ)			820.9	838.9	Sdy Cl*	04-02-84	3/4" / 1.6' - .020"	
A-520(PZ)			812.1	828.6	Sdy Cl*	02-09-84	3/4" / 1.5' - .010"	
A-521(PZ)			810.2	830.2	Sly Sd*	02-10-84	3/4" / 1.5' - .010"	Flowing hole
D-521A(PZ)			808.6	830.0	Sly Sd*	04-03-84	3/4" / 1.6' - .020"	Flowing hole
D-522(PZ)			812.2	834.1	Clly Sd*/Sd*	03-28-84	2" / 2' - .020"	Flowing hole
D-523(PZ)			793.4	821.6	Sdy Cl	03-27-84	2" / 2' - .020"	Flowing hole
A-524(PZ)			819.0	827.0	Lean Cl	03-29-84	2" / 2' - .020"	

* Field Classification
 OW - Observation Well
 P, PZ - Piezometer

TABLE 2 - Relief Drains; Schedule and Performance

Number	Grd. El.	Installation Date	Order of Installation	Type Riser/Tip	Feet of Sand Around Tip	Initial Discharge, Est.	Imed. Affect Adj. on P2's	Discharge: After Drains Completed 10 April 84	Discharge Pool El. 863.4 4 Oct 84
RD-1	830.84	4-9-84	8	2" / 4' - .020"	9.6'	1/2-1 gpm	-	.1 gpm	.1 gpm
RD-2	831.24	4-8-84	6	2" / 2' - .020"	8.3'	5 gpm	>-3' (D-521A, A-521)	1.0 gpm	1.2 gpm
RD-3	832.62	4-7-84	4	2" / 2' - .020"	7.0'	1-2 gpm	-.3', -.5' (D-521A, A-521)	.1 gpm	.1 gpm
RD-4	835.39	4-7-84	1	2" / 2' - .020"	5.2'	4 gpm	-.2' (D-522)	.5 gpm	.3 gpm
RD-5	834.92	4-7-84	2	2" / 5' - .020"	9.8'	25 gpm	-4.2' (D-522)	7.1 gpm	1.0 gpm
RD-5A	835.60	4-10-84	10	4" / 5' - .030"	7.5'	4 gpm	-.2' (D-522)	6.2 gpm	3.75 gpm
RD-5B	834.33	4-9-84	9	4" / 6.5' - .030"	9.2'	18 gpm	-1.2' (D-522)	7.9 gpm	6.6 gpm
RD-6	833.28	4-7-84	3	2" / 5' - .020"	8.2'	20 gpm	-1.4' (D-522)	10 gpm	6.6 gpm
RD-7	832.25	4-8-84	5	2" / 3' - .020"	6.6'	2 gpm	-	1.5 gpm	1.0 gpm
RD-8	830.82	4-8-84	7	2" / 2.5' - .020"	7.4'	-	-	.1 gpm	.1 gpm

Relief drains were installed between 7 April 1984 and 10 April 1984. Pool level during this time rose from El. 867.7 to El. 868.4.

Table 3 Effect of Relief Drains on Piezometric Levels

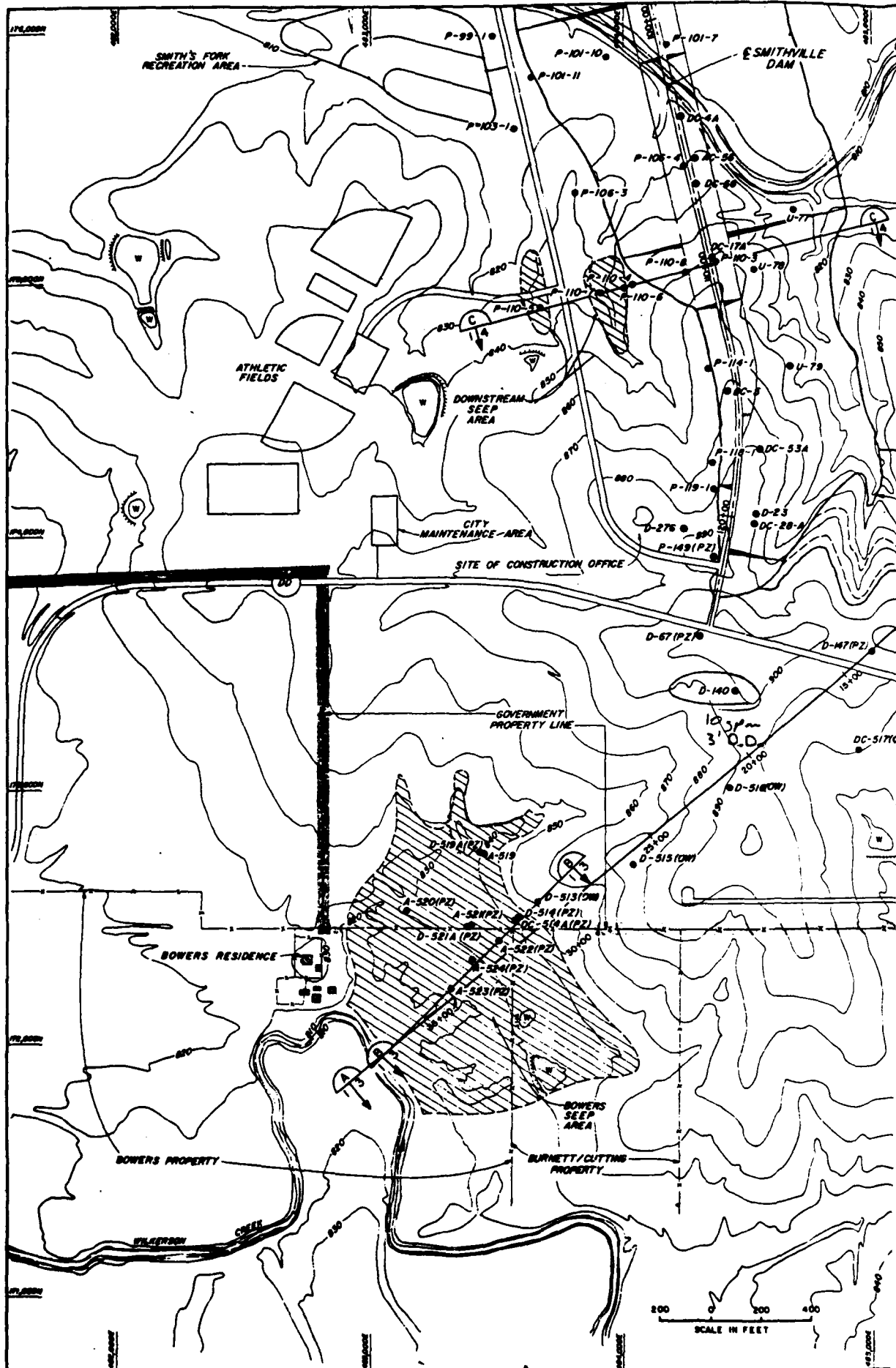
Device	Piezometric Level Before Relief Drains Installed (Friday 6 April 1984)	Piezometric Level During Installation (9 Drains Installed) (Monday 9 April 1984)	Piezometric Level After Relief Drains Installed (Tuesday 10 April 1984)	Projected Piezometric Levels Tuesday 10 April 1984 (No drains)	Comparison of Pz Levels Friday Readings w/ Tuesday	
					Actual Readings	Projected Readings
D-513(O4)	847.8	846.85	845.65	847.9	-2.15	-2.2
D-514(PZ)	845.6	843.55	843.20	845.8	-2.4	-2.6
DC-514a(PZ)	845.9	843.95	843.65	846.1	-2.25	-2.4
D-515(O4)	850.65	850.45	850.35	850.8	-3	-4
D-516(O4)	853.7	853.85	853.75	853.9	+05	-2
D-518 (O4)	857.3	No Reading	857.7	857.6	+4	+1
D-519a(PZ)	838.87	838.92	838.82	838.9	-05	-1
A-520(PZ)	822.4	822.9	822.5	* 822.6	+35	+2
A-521(PZ)	838.3	832.5	832.3	838.6	-5.98	-6.2
D-521a(PZ)	837.64	832.35	832.15	**	-5.49	-
D-522(PZ)	843.78	836.62	835.93	843.9	-7.85	-7.9
D-523(PZ)	818.31	818.96	818.86	-	+35	-
A-524(PZ)	826.76	826.91 +	826.86	826.8	+1	+06

* Used 25% response
 ** Could not project since 6 April 1984 is only
 reading available before beginning drains

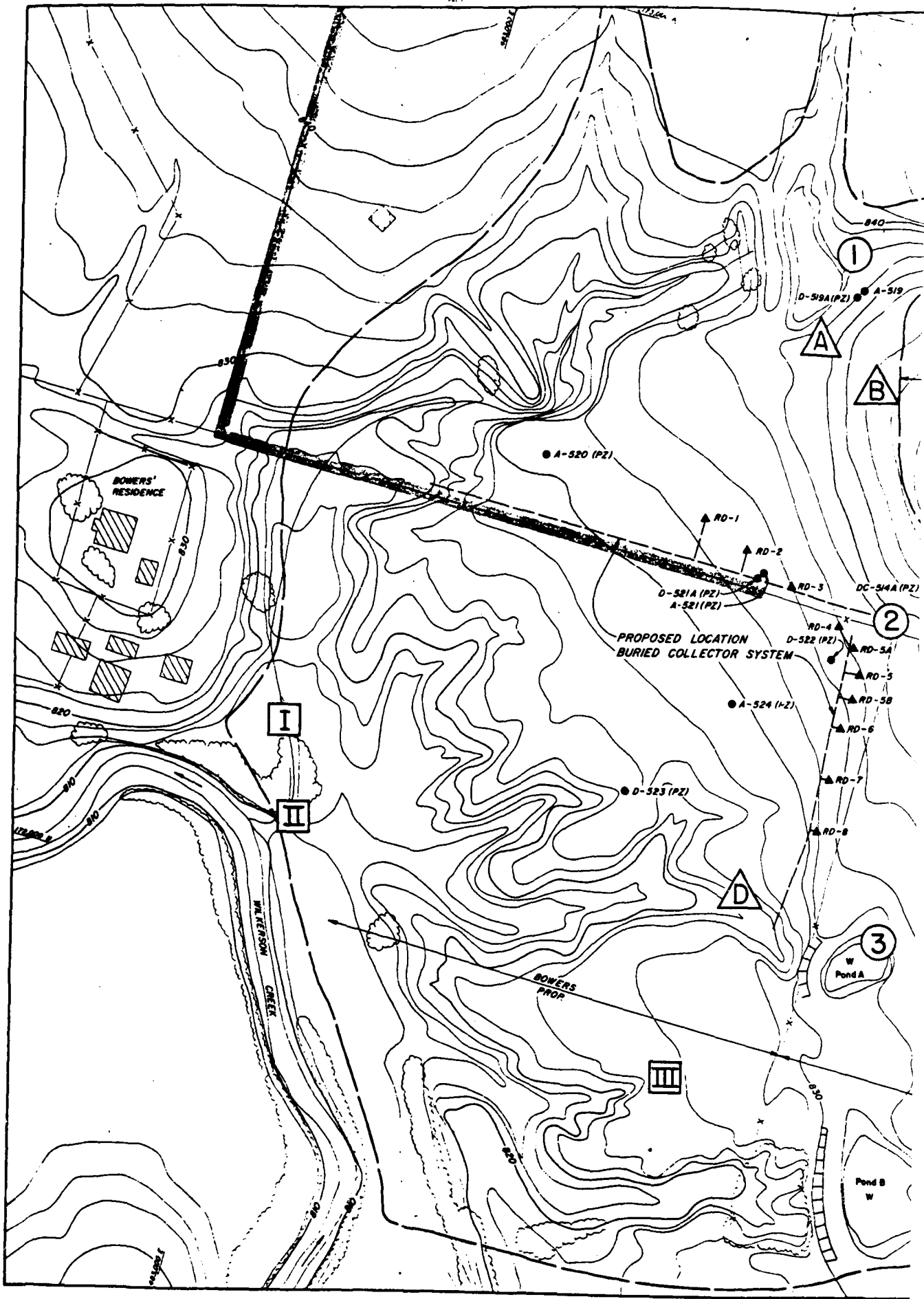
Pool Elevations: 6 April 1984 867.7
 9 April 1984 868.3
 10 April 1984 868.4

DRAWINGS

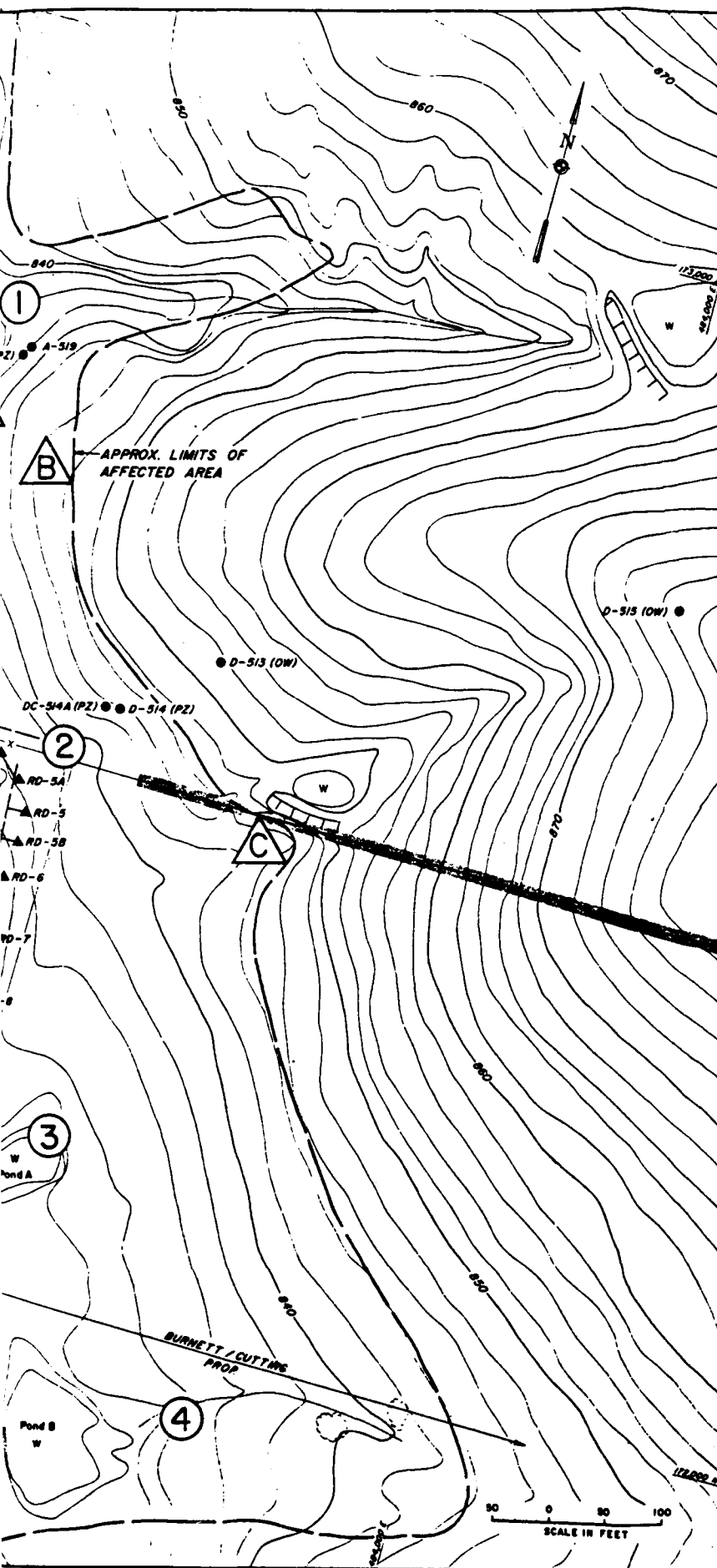
DRAWINGS



15-1-7



15-1-11



LEGEND

- RD RELIEF DRAIN
- (OW) OBSERVATION WELL
- (PZ) PIEZOMETER
- GOVT. PROP. LINE
- SEEPAGE - FLOWING
- SEEPAGE - DAMP
- EROSION - SEVERE HEADCUTTING

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J. M. M.

Drawn by: J. W. P.

Checked by: H. L. F.

Submitted by: K. D. W.

Little Platte River, Missouri
Smithville Lake
LEFT ABUTMENT SEEPAGE REPORT

BOWERS SEEP AREA

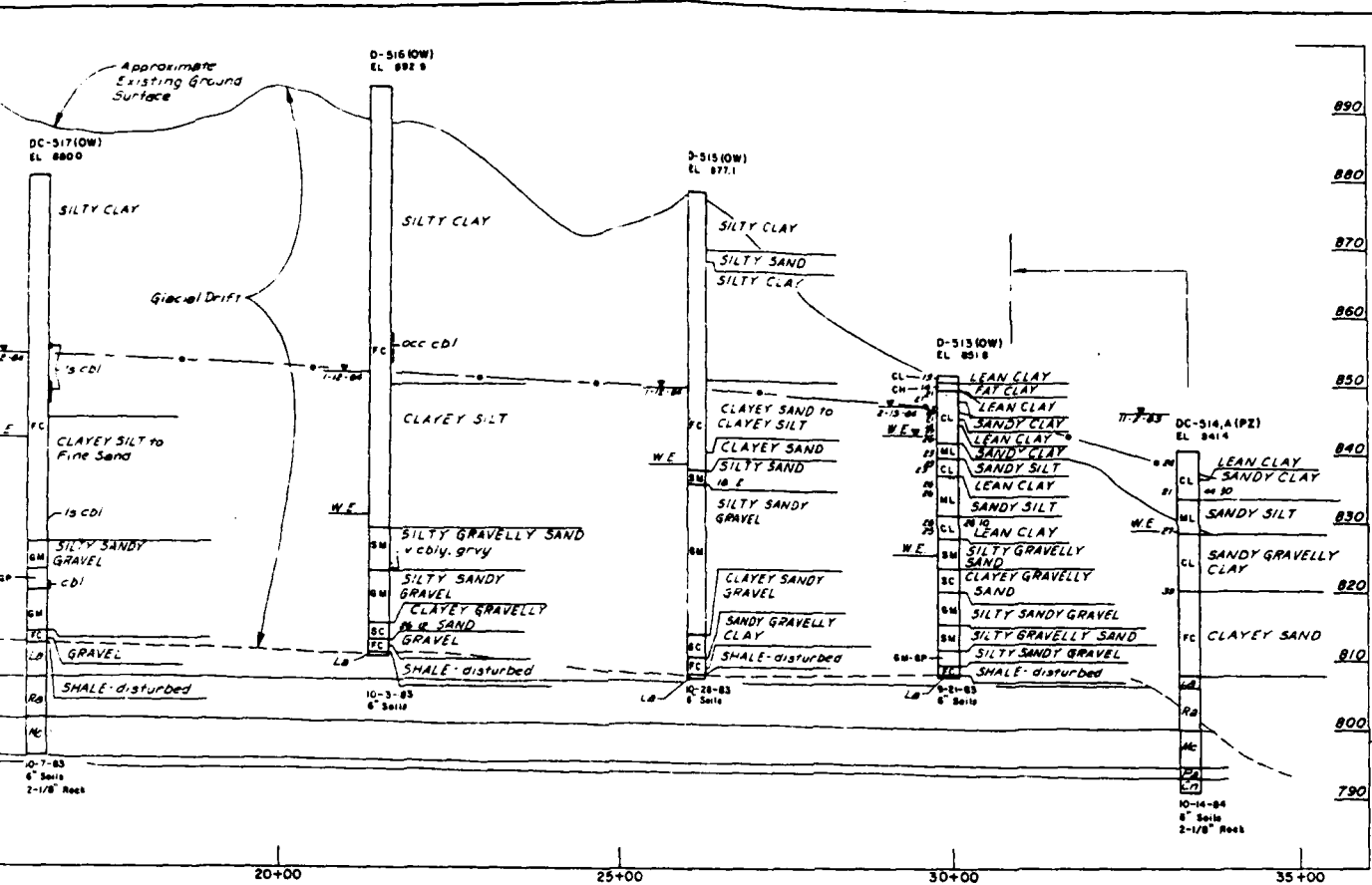
Scale: AS SHOWN

Date: DECEMBER 1994

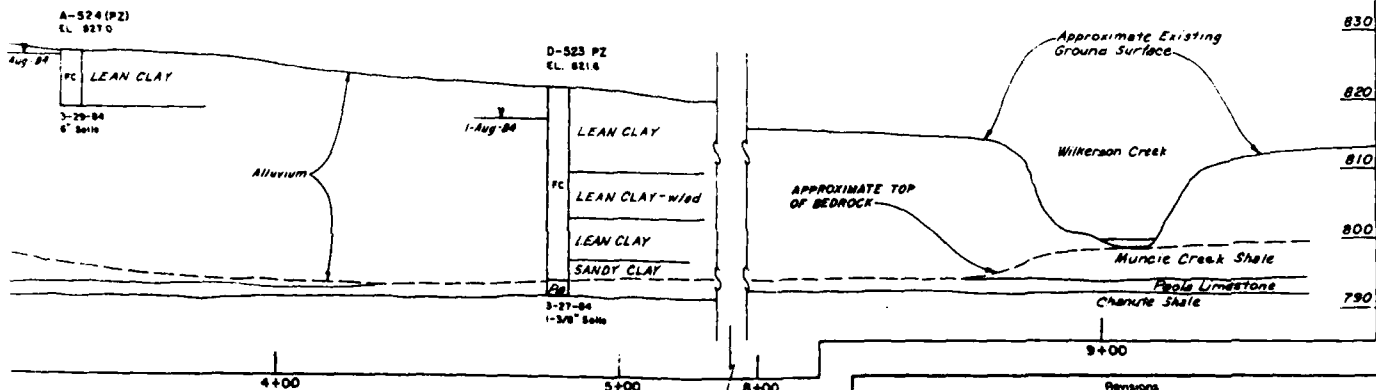
Drawn by: [Signature]

172,000 ft

RP-3-1722



THROUGH LEFT ABUTMENT A



ROUGH BOWERS SEEP AREA B

Revisions			
By	Notes	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: J. M. M.

Drawn by: J. G. S.

Checked by: H. L. F.

Submitted by: K. D. W.

LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE
LEFT ABUTMENT SEEPAGE REPORT

PROFILES-LEFT ABUTMENT;
BOWERS SEEP AREA

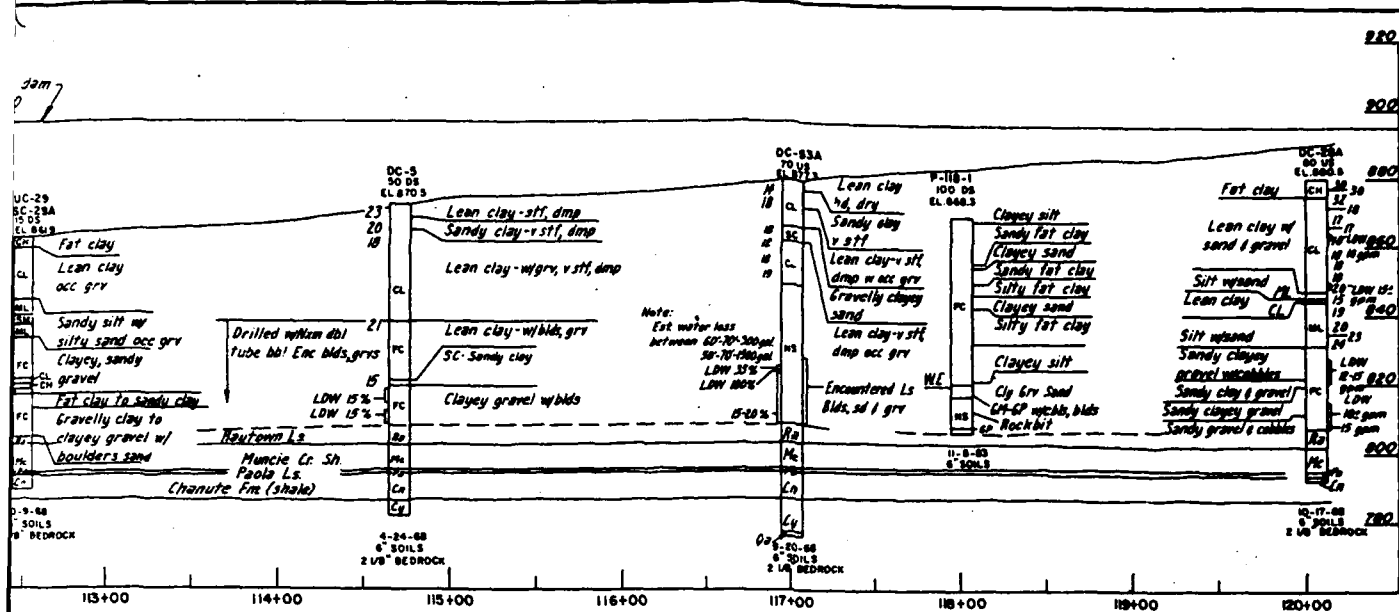
Scale: AS SHOWN

Date: DECEMBER 1984

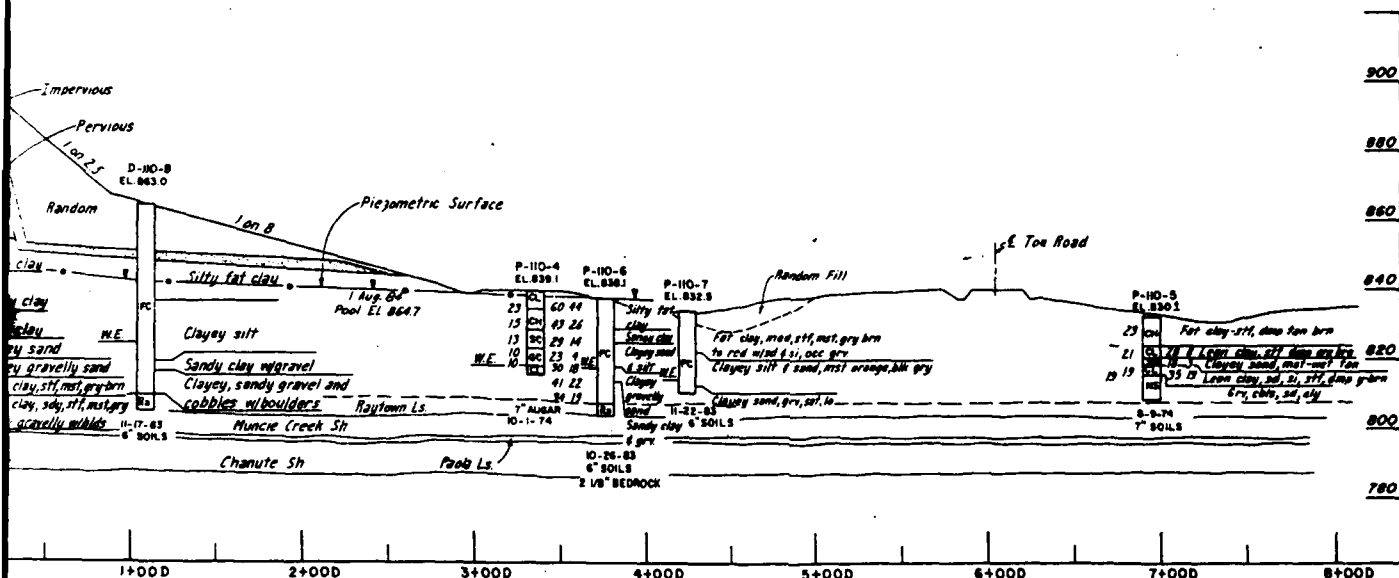
Sheet number: 1

RP-3-1723

FIG. 10-10-84



BUTMENT DAM AXIS PROFILE



TMENT
0+00 C
1 4

Revisions			
Symbol	Description	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: W. F. L.

Drawn by: E. E. R.

Checked by: H. L. F.

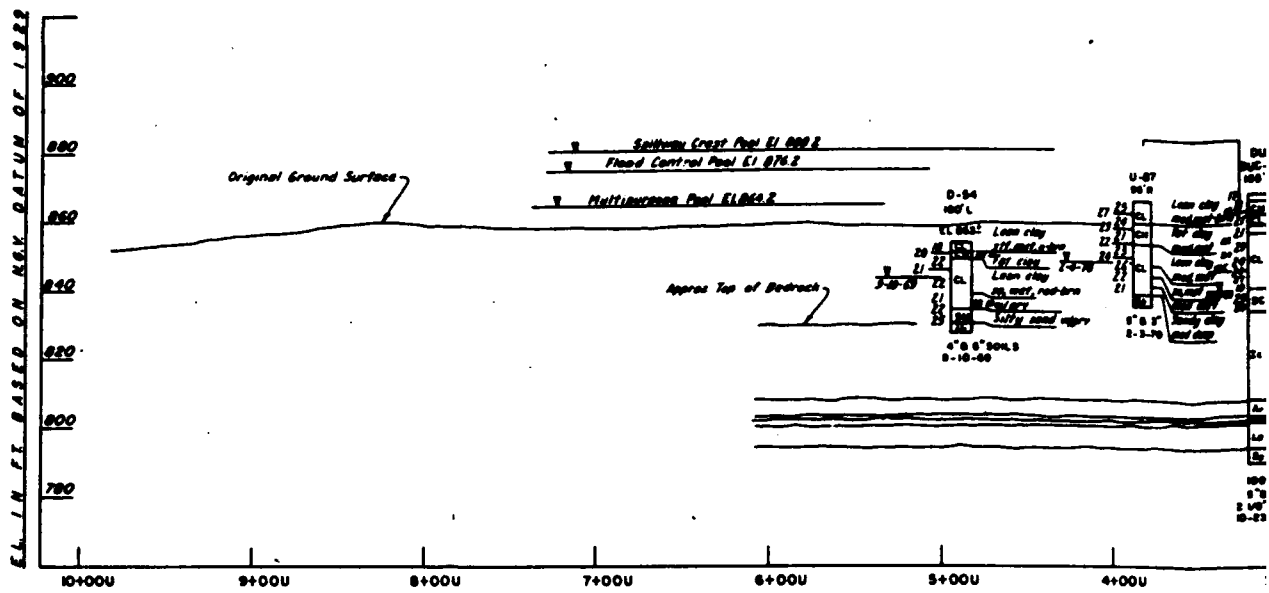
Submitted by: K. D. W.

Scale: AS SHOWN

Date: DECEMBER 1964

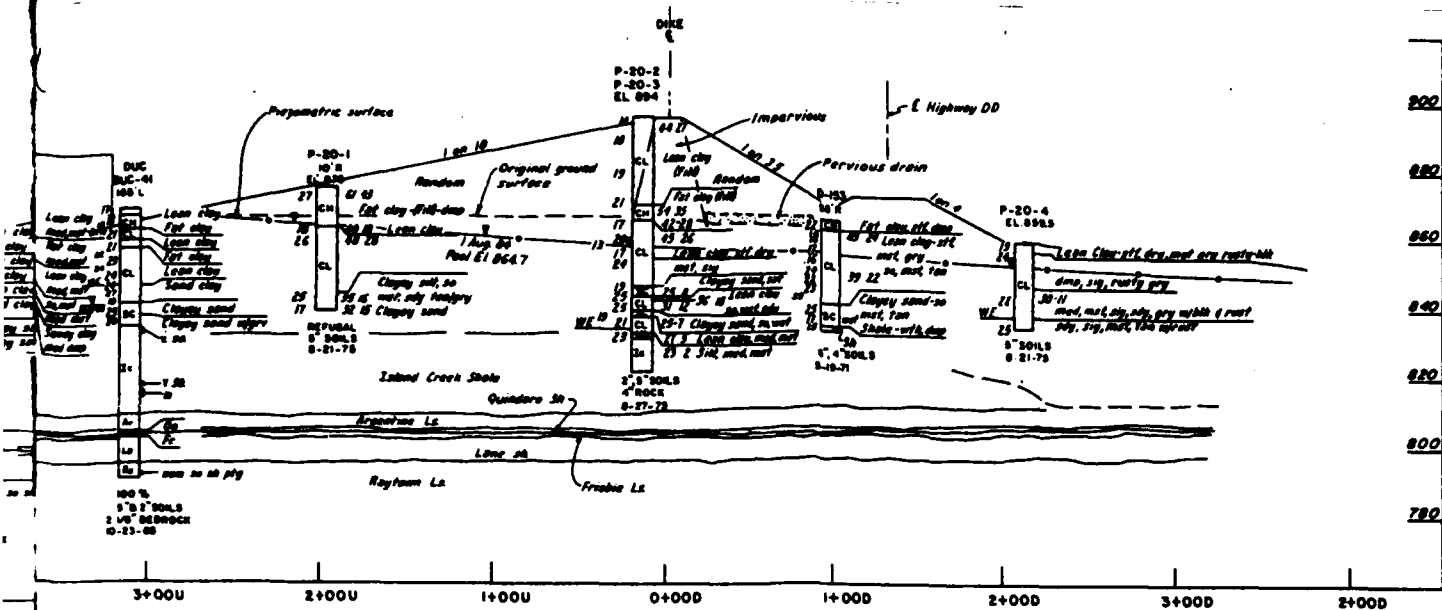
Sheet number: 1

Project number: RP-3-1734



Note:
1. Piezometric surface shown is measured in the pervious basal layer of the overburden.

15-1-7



MAIN DIKE SECTION D
115

Revisions			
Symbol	Description	Date	Approved

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

Designed by: W.F.L.	 LITTLE PLATTE RIVER, MISSOURI BATHVILLE LAKE LEFT ABUTMENT SEEPAGE REPORT MAIN DIKE SECTION-STATION 20+00	Scale: AS SHOWN	 RP-3-1725
Drawn by: E.E.R.		Date: DECEMBER 1964	
Checked by: H.L.F.			
Submitted by: K.D.W.			

GEOLOGIC COLUMN - LEFT ABUTMENT, SMITHVILLE DAM

OVERBURDEN variable thickness, lean clays.
Silty clays w/ occasional lenses of fine sand & occasional fine to coarse gravel, underlain by clayey silts & fine to medium sands w/ clay & occasional gravel. Clayey, sandy gravels w/ limestone chert & quartzite nodules & boulders.

WYANDOTTE FM - ISLAND CREEK SHALE 26'
Soft, platy, w/ clayey, calcareous, few phosphatic nodules in upper 15', H gray to gray. Soft to v soft, clayey, w/ high angle fractures (approximately 15 to 16' above base of Island Creek). Moderately hard to hard, dense, medium to thin bedded, brown-gray limestone w/ few small open vugs & occasional wavy shale partings (approximately 8 to 7' above base of Island Creek).

WYANDOTTE FM - ARGENTINE LIMESTONE 5'
Moderately hard to hard, dense, thin wavy bedded, brown-gray, w/ numerous soft dk gray wavy shale partings & bands.

WYANDOTTE FM - QUINDARO SHALE 0.5'
Soft, very calcareous, gray, fossiliferous.

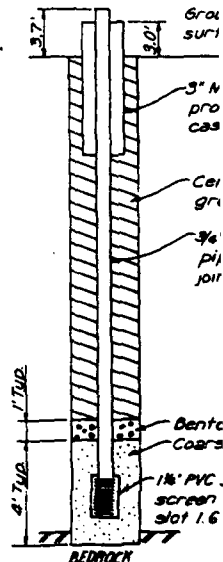
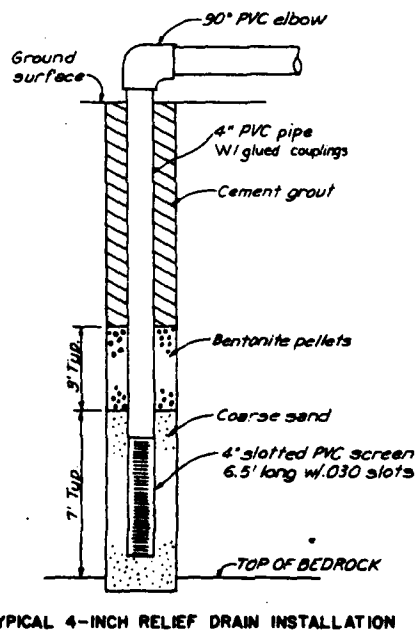
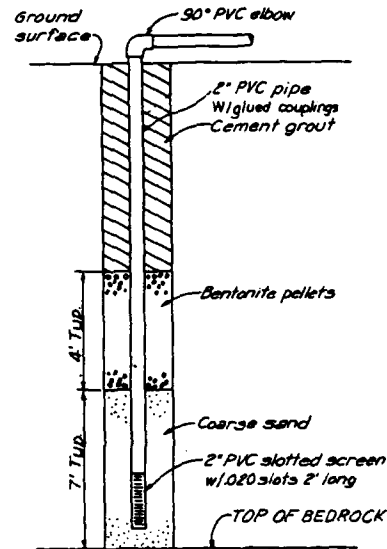
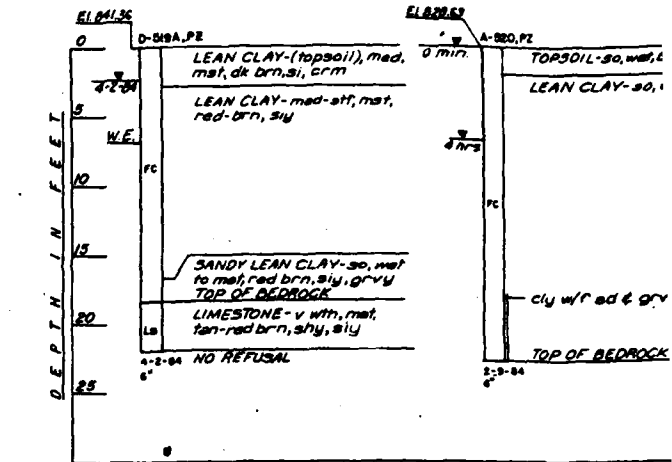
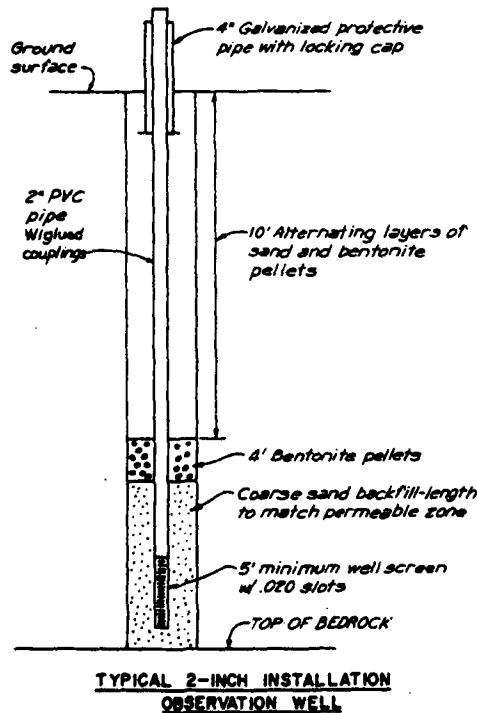
WYANDOTTE FM - FRISBIE LIMESTONE 1'
Moderately hard to hard, dense, thick-bedded, tabular & fossiliferous nodules, gray-brown.

LANE FM - SHALE average thickness 7'
Soft, fissile, slightly calcareous, dark gray.

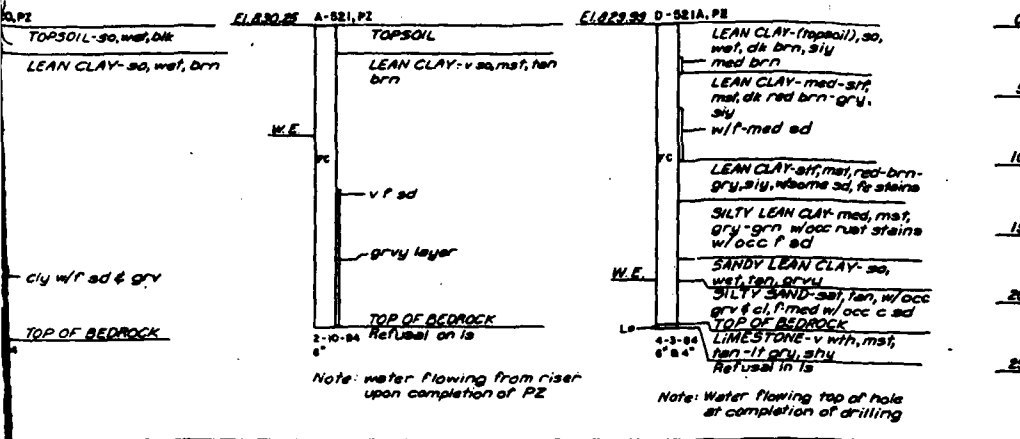
IDA FM - RAYTOWN LIMESTONE 6'
Moderately hard to hard, crystalline to dense fossiliferous & massive in upper part. Thin-bedded w/ intercalated soft, calcareous shale w/ persistent soft to v soft clayey shale (approximately 1 to 1.5' above base of Raytown) overlying moderately hard, dense to finely crystalline fossiliferous limestone w/ cherty partings in lower part.

IDA FM - MUNCIE CREEK SHALE 7'
Soft to moderately hard w/ occasional v soft thin laminae, platy, dk gray to black, calcareous, wavy to block carbonaceous sh near base.

IDA FM - PAOLA LIMESTONE 1'
Moderately hard to hard, finely crystalline, massive w/ occasional argillaceous partings, gray fossiliferous.



alt	alternating	amp	amper
ang	angular	def (s)	defect
an	anhydrite	ent	entrance
ar	argillaceous	f (y)	fine
bdd	bed, bedded, bedding	to	to
bdr	bedrock	th	thickness
bly	blocky	tm	time
bt	blue	tos (s)	topsoil
bld	bedded	trac (s)	trace
blk	black	trng (s)	trough
bnc (d)	breccia (brecciated)	th	thickness
brs	broken	th	thickness
brn	brown	gr	gravel
c	coarsen	grs	gravel
calc	calcareous	grm	gravel
carb	carbonaceous	grv (y)	gravel
car	carly	gr	gravel
cbl	cobble	grp	gravel
cht	chart	he	height
crd	circulation	hd	head
cl (s)	clay, (clayey)	hld	head
cl	closed	hor	horizontal
cncl	cemented	incl	inclined
col	columnar	incl	inclined
conc	concretions	iniam	injection
cong	conglomerate	int	interior
crm	crumbly	f (s)	fine
d	dark	to	to
dk	dark	tem (s)	temperature



UNIFIED SOIL CLASSIFICATION SYSTEM

GW	Well graded gravel, gravel-sand mixtures, little or no fines.	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or silty silts w/ slight plasticity.
GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy loam, silty clay, lean clay.
GM	Silty gravel, gravel-sand-silt mixtures.	OL	Organic silts and organic silty clays of low plasticity.
GC	Clayey gravel, gravel-sand-clay mixtures.	MH	Inorganic silts, silty sands or clayey silts, fine sandy or silty silts, elastic silts.
GM	Well-graded sands, gravelly sands, little or no fines.	CH	Inorganic clays of high plasticity, fat clays.
GP	Poorly graded sands or gravelly sands, little or no fines.	DH	Organic clays of medium to high plasticity, organic silts.
SM	Silty sands, sand-silt mixtures.	PI	Peat and other highly organic silts.
SC	Clayey sands, sand-clay mixtures.		

Classification from actual laboratory tests where LL and PI are shown.

Desl classification, where used, is in accordance with the Unified Soil Classification System.

For details on the Unified Soil Classification System, See Waterways Experiment Station Technical Memorandum No. 3-957 dated March 1953 and revised in 1960.

TERMS FOR CONSISTENCY OF SOIL AND HARDNESS OF BEDROCK

Consistency	Estimated Unconfined Compressive Strength (Tons per square foot)
Very soft	< 0.25
Soft	0.25-0.5
Medium	0.5-1.0
Stiff	1.0-2.0
Very stiff	2.0-4.0
Hard	> 4.0

BEDROCK

SCALE OF HARDNESS

Very soft or plastic	Can be indented easily with thumb.
Soft	Can be scratched with fingernail.
Moderately hard	Can be scratched easily with knife; cannot be scratched with fingernail.
Hard	Difficult to scratch with knife.
Very Hard	Cannot be scratched with knife.

DETACHED BORINGS

ABBREVIATIONS

dmp	damp	lea	leached	rnd (d)	round, (rounded)
dol (c)	dolomite, (dolomitic)	lg	lignite	sat	saturated
ent	entombed	ls	limestone	scat	scattered
f (y)	fine, (finely)	lt	light	sd (y)	sand, (sandy)
fs	fine	lo	loose	sev	several
fd	filled	LC	lost core	sh (y)	shale, (shaly)
fm	firm	L.D.W.	lost drill water	sl (y)	silt, (silty)
fsa (s)	fossil, (fossiliferous)	med	medium	sls	siltstone
fract (d)	fracture, (fractured)	mic	micaceous	slt	slightly
frag (d)	fragments, (fragmented)	min	mineralized	slic	siliceous
fr	frable	mod (y)	moderate, (moderately)	slic	siltstone
fs	fossil	mat	mottled	so	soft
gr	gran	mas	massive	sol (d)	solution, (solutionized)
grn	gradation	mat	mat	ss	sandstone
grm	green	md	material	st (d)	stained, (staining)
grv (y)	gravel, (gravelly)	mb	matrix	stf	stiff
gry	gray	md	material	sty	stylolitic
grym	grum	num	numerous	v	very
ha	high angle	occ (y)	occasional, (occasionally)	vert	vertical
hd	hard	op	open	vib	vibrant
hld	healed	or	orange	w	water
hor	horizontal	org	organic	w/	with
intd	interbedded	par	partially	wh	weathered
incl	inclined	pit	pit, pitted, pitting	wht	white
inlem	interlaminated	pl	plastic	x-bed	cross-bedded
irreg	irregular	pla	plastic	xln	crystalline
j (s)	joint, (joints)	ply	ply	y	yellow
la	low angle	ph	plate		
lrm (d)	laminar, (laminated)	plg (d)	porling, (porlings)		
		qtz (d)	quartz, (quartzite)		

BEDROCK UNIT THICKNESS

Parting	< 0.02'
Bed	0.02' to 0.2'
Thin Bed	0.2' to 0.5'
Medium bed	0.5' to 1.0'
Thick bed	1.0' to 2.0'
Massive	> 2.0'

TYPE OF EXPLORATION

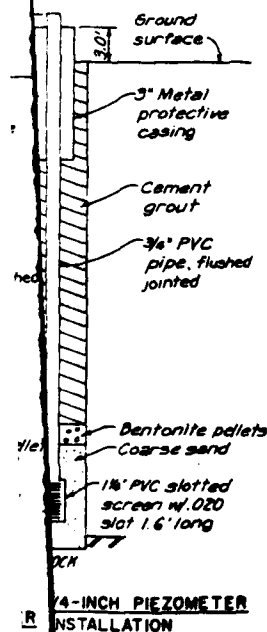
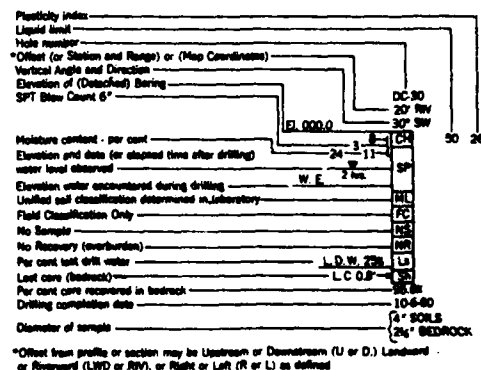
MAP SYMBOL

- Vertical boring
- Inclined boring showing direction and vertical angle

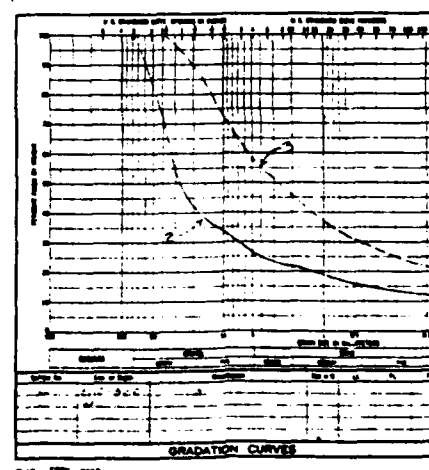
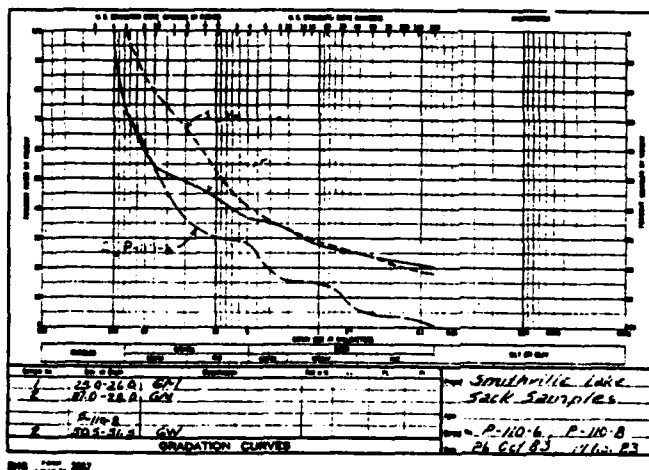
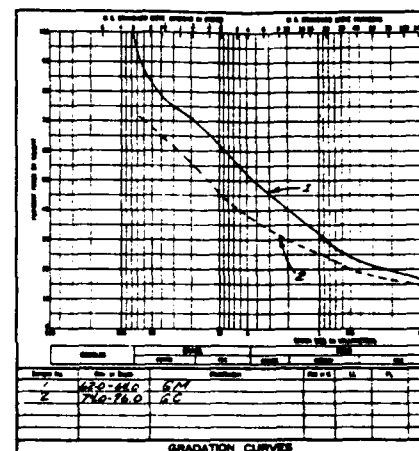
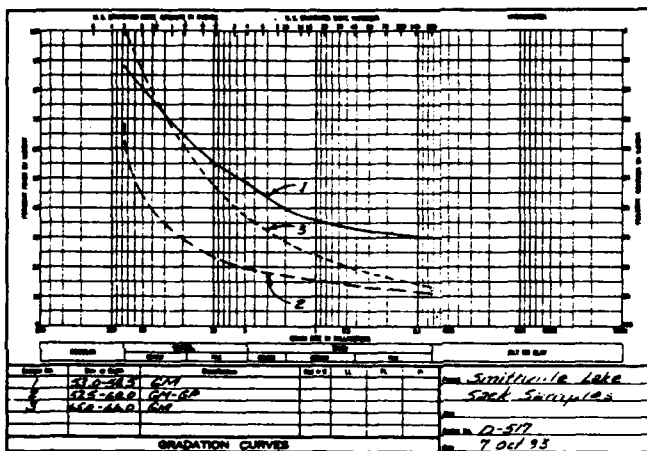
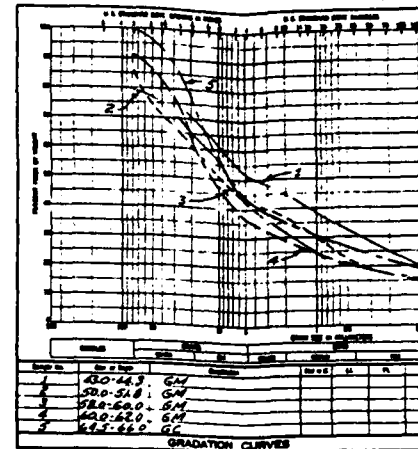
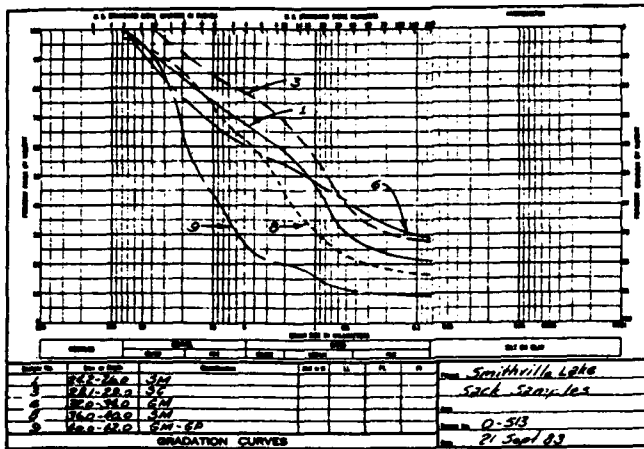
CODE DESIGNATION

- D Drive sample hole
- C Core hole
- TP Test pit (includes power auger 24" or larger diameter)
- U Undisturbed sample hole
- A Auger hole-hand or power auger less than 24" diameter
- NS Not Sampled (Field Classification from cuttings only)
- FS Field Section of outcrop
- RD Observation well
- P Piezometer no.
- PZ Piezometer

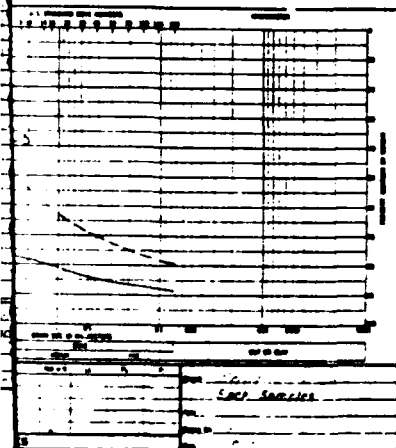
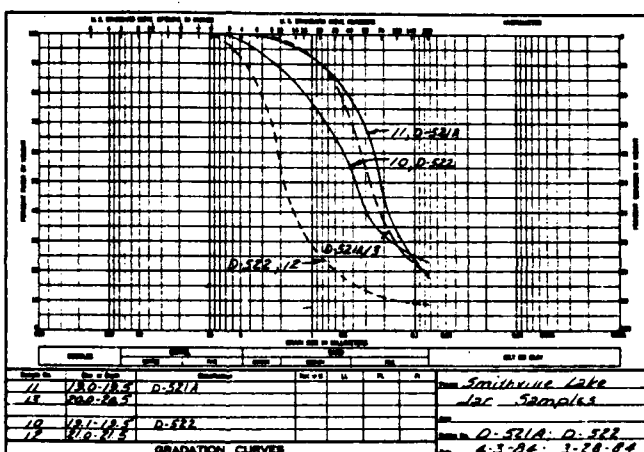
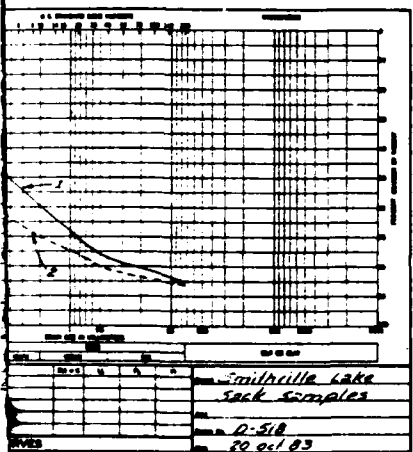
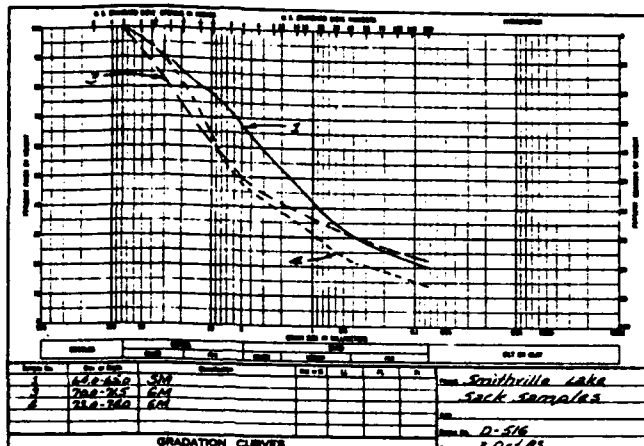
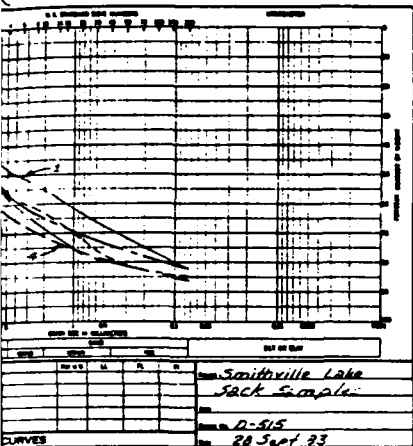
LEGEND FOR LOGS OF BORINGS



Revisions			
Symbol	Descriptions	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	J. M. M.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE LEFT ABUTMENT SEEPAGE REPORT	
Drawn by:	R. L. D.	GEOLOGIC COLUMN AND LEGEND, DETACHED BORINGS, AND TYPICAL INSTALLATION DIAGRAMS	
Checked by:	H. L. F.	Scale:	AS SHOWN
Submitted by:	K. D. W.	Date:	DECEMBER 1984
		Sheet number:	1 of 1
		RP-3-1726	



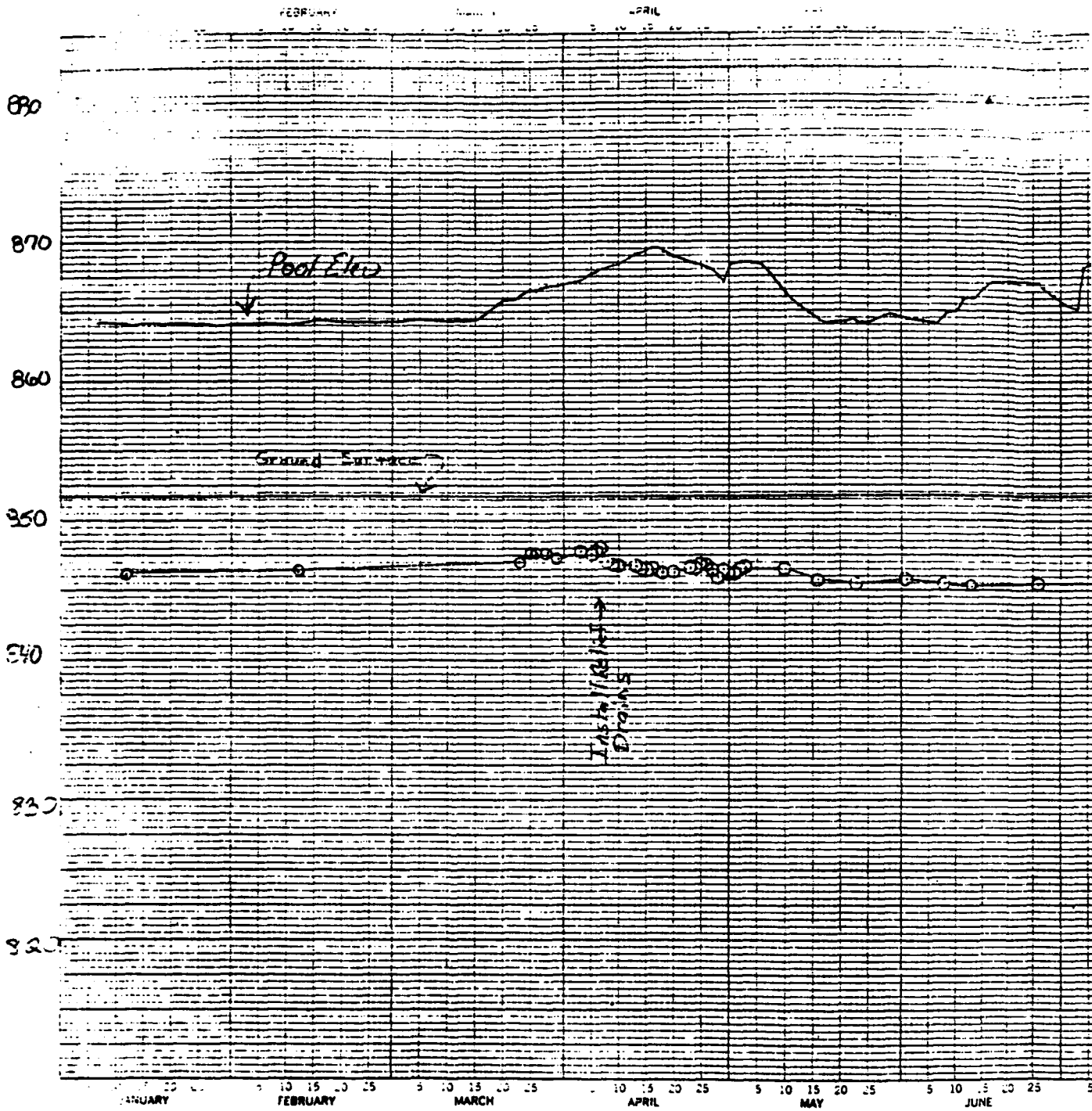
5-1-7



Symbol	Revisions	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	J.M.M.	LITTLE PLATTE RIVER, MISSOURI SMITHVILLE LAKE LEFT ABUTMENT SEEPAGE REPORT	
Drawn by:	P.B.T.	GRADATION CURVES LEFT ABUTMENT BORINGS	
Checked by:	H.L.F.	Scale:	AS SHOWN
Submitted by:	H.O.W.	Date:	DECEMBER 1984
		Sheet number:	RP-3-1727

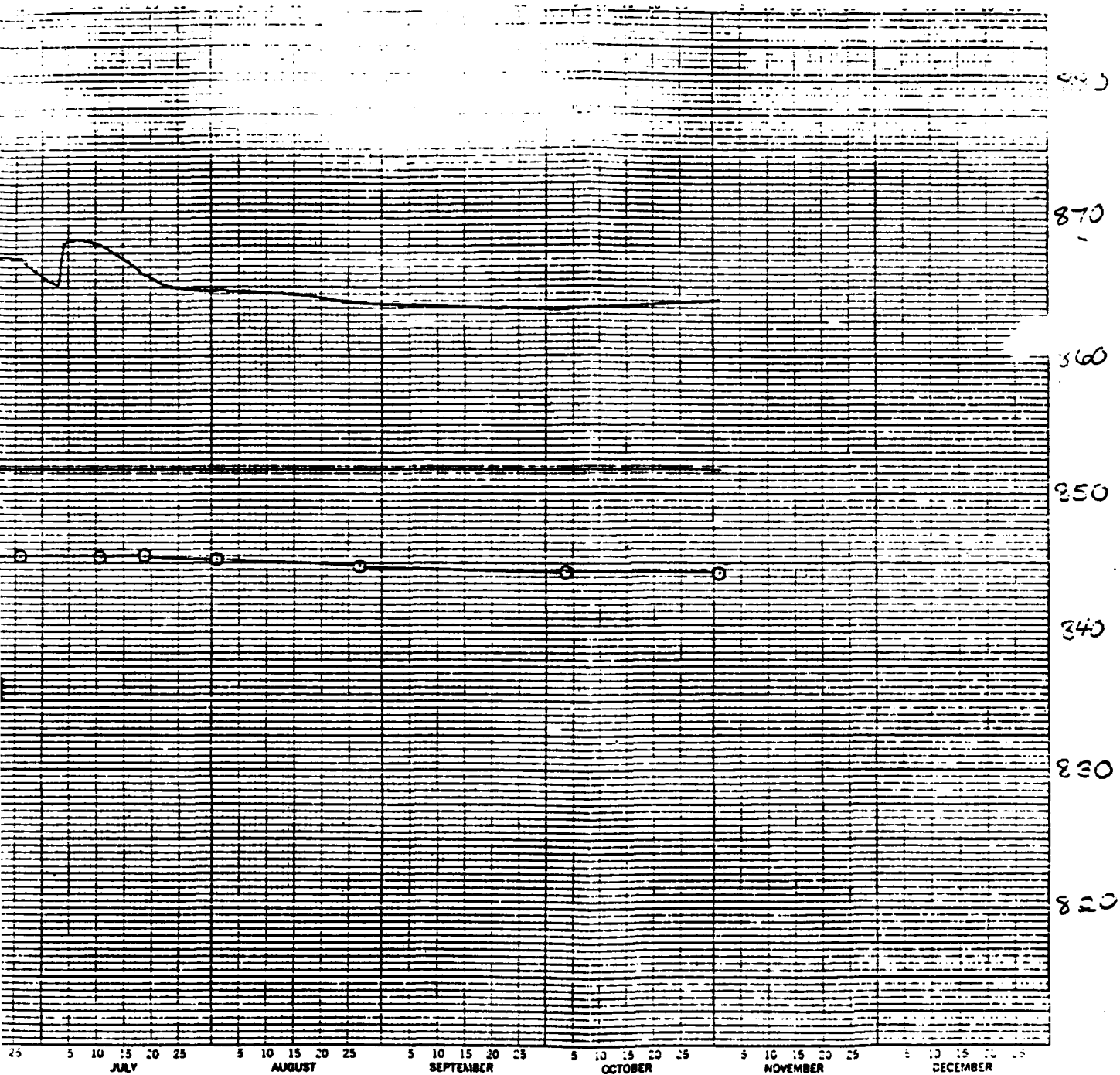
47 2813

ICE FIELD DATA & DIVISIONS
ALBERTA & SASKATCHEWAN



1984

15-1-7



1984

P-513

STA

TIP EL 810.0

RNG

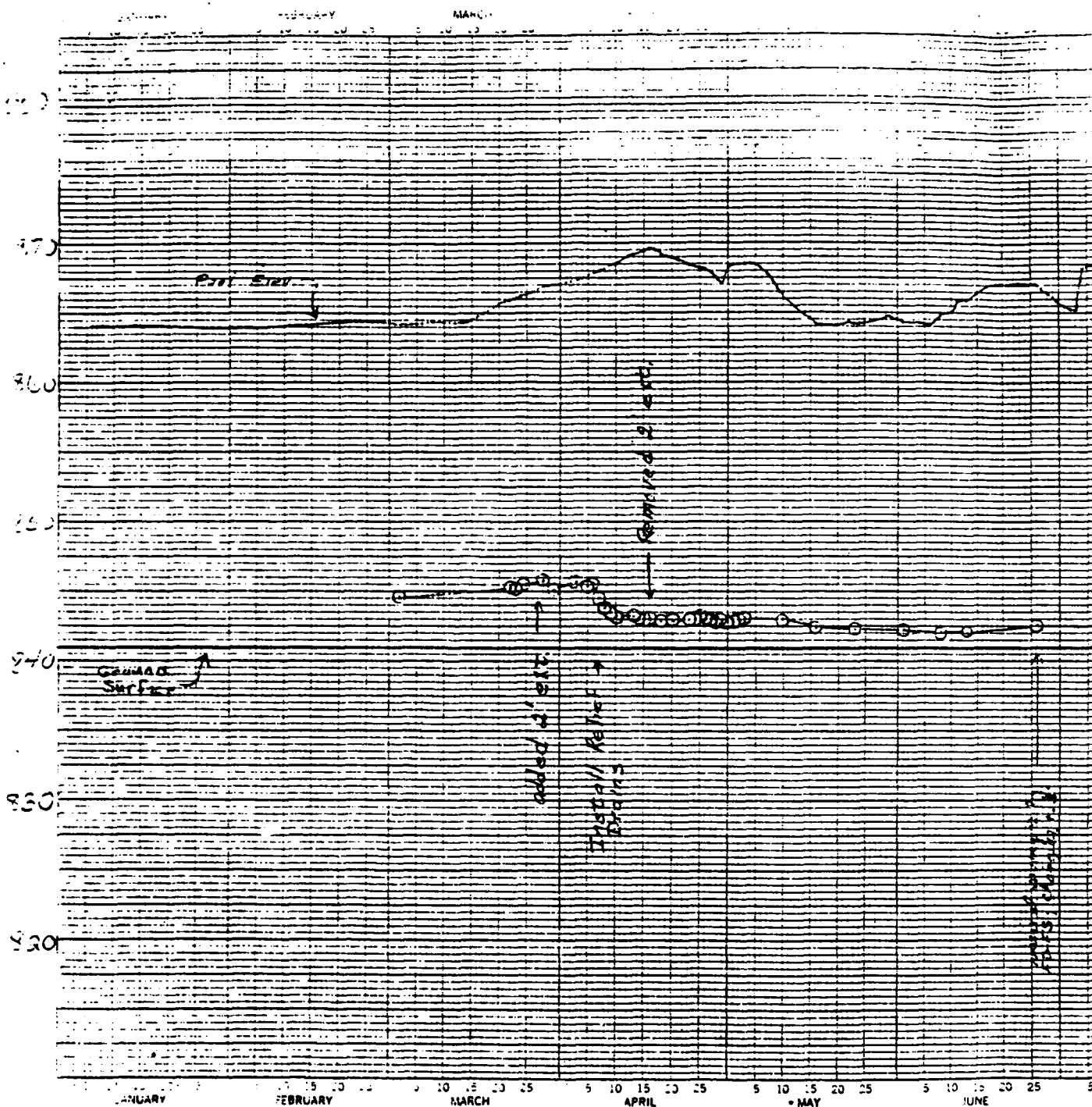
GRND EL 851.3

MAT'L CL SDY GRV

APPENDIX B Plate No. 8 RP-3-1728

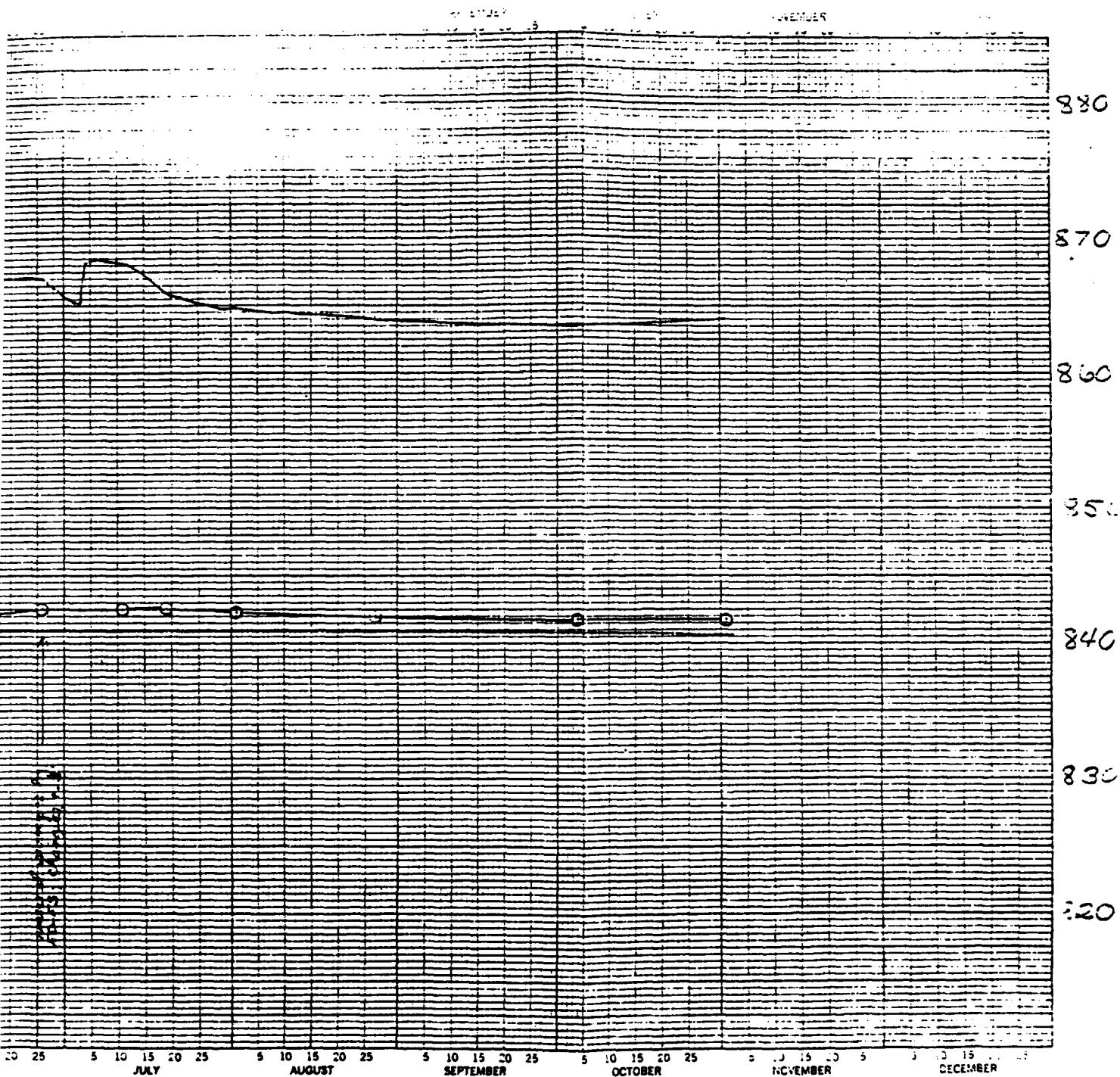
47 2813

ICE 1 YEAR IN L.A.S. & DIVISIONS
MILITARY & MARINE



1984

15-1-7



1984

STA
RNG
MAT'L CLY SAND

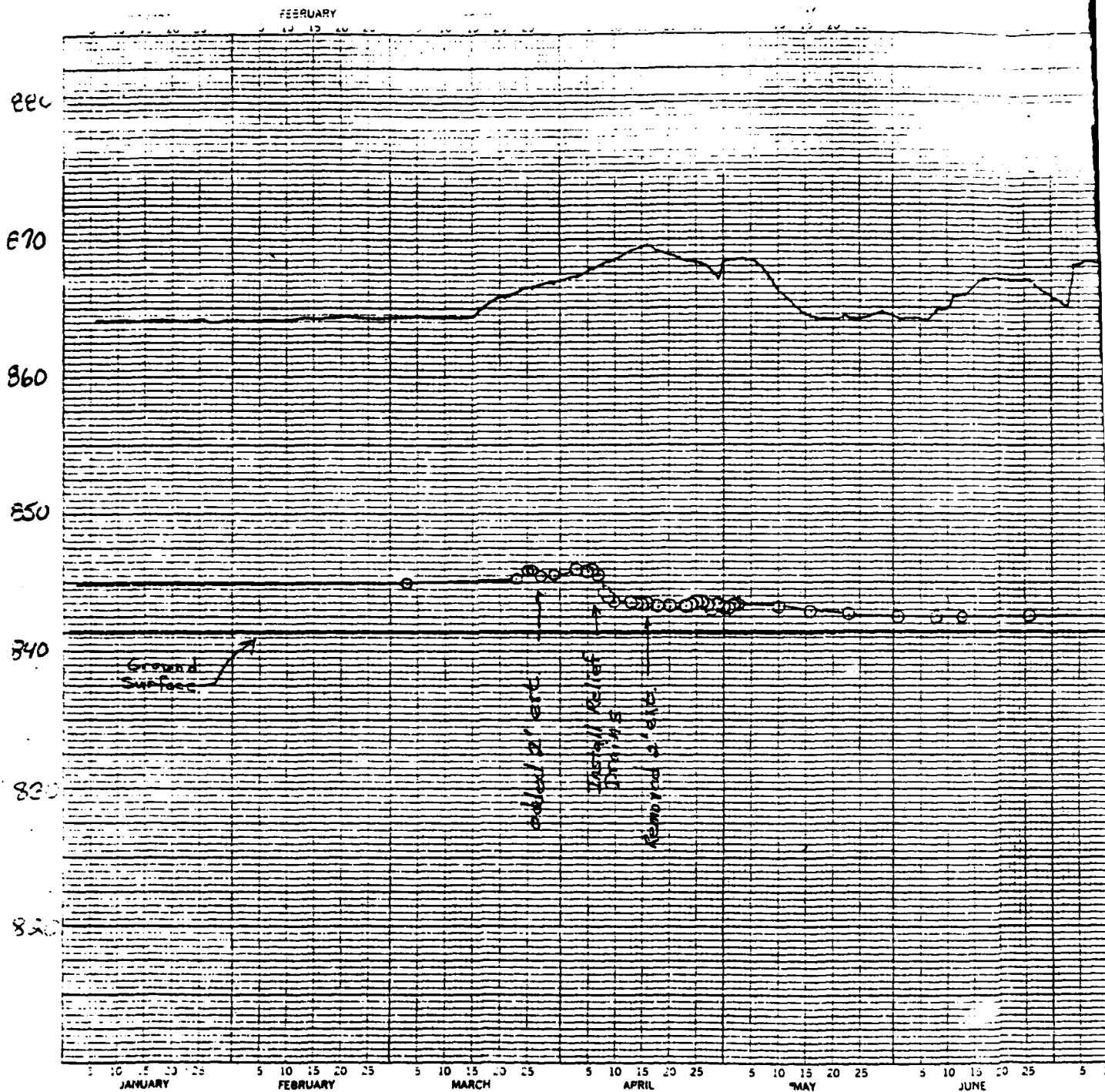
TIP EL 217.7
GRND EL 341.0

Plate No. 9 RP-3-1729

2

47 2813

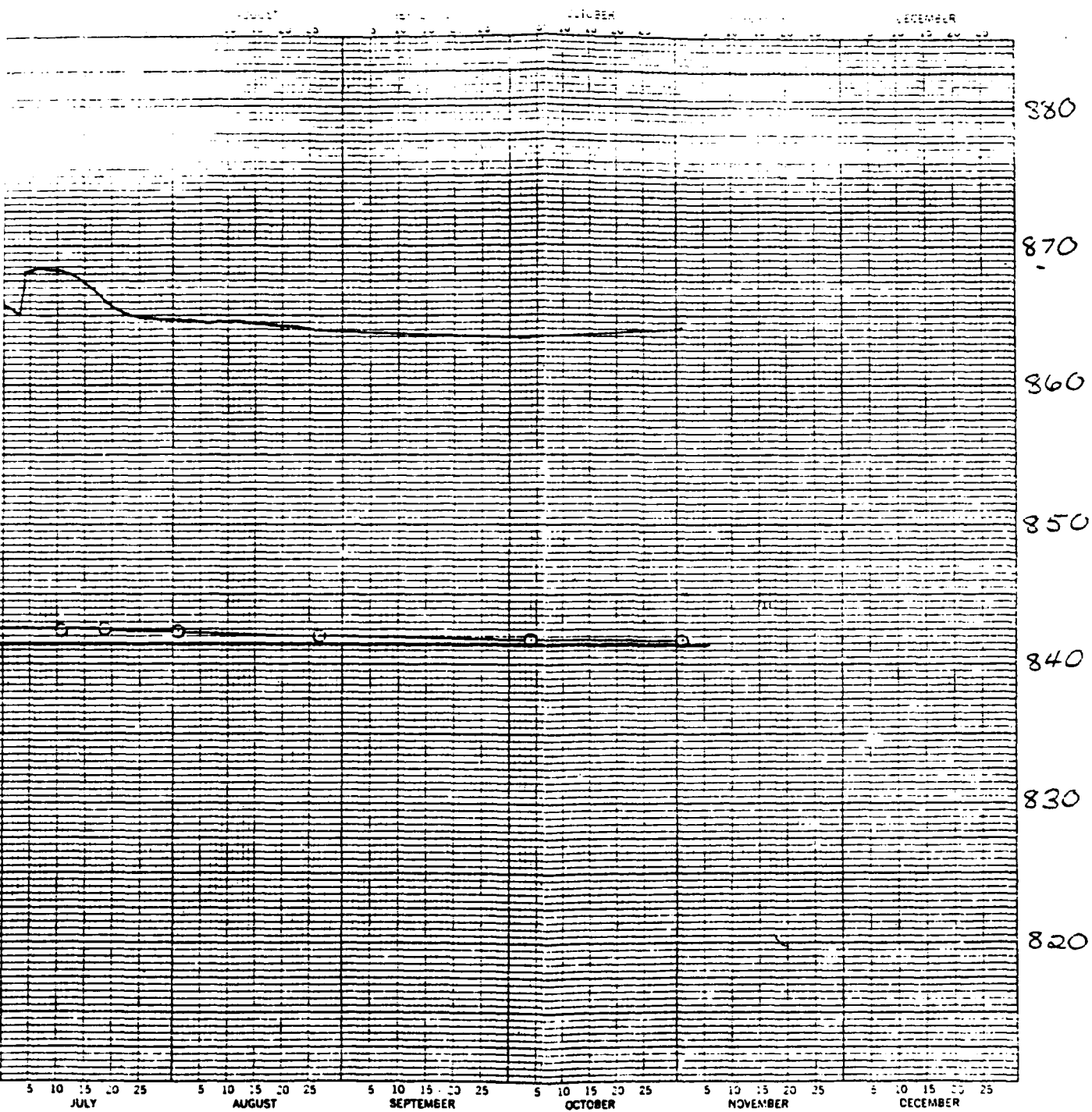
ICE 1 YEAR BY DAY: X 100 DIVISIONS
ALUPPE & SONS CO. NEW YORK



1984

15-1-7





DC-514A

STA

TIPEL 309.9

RNG

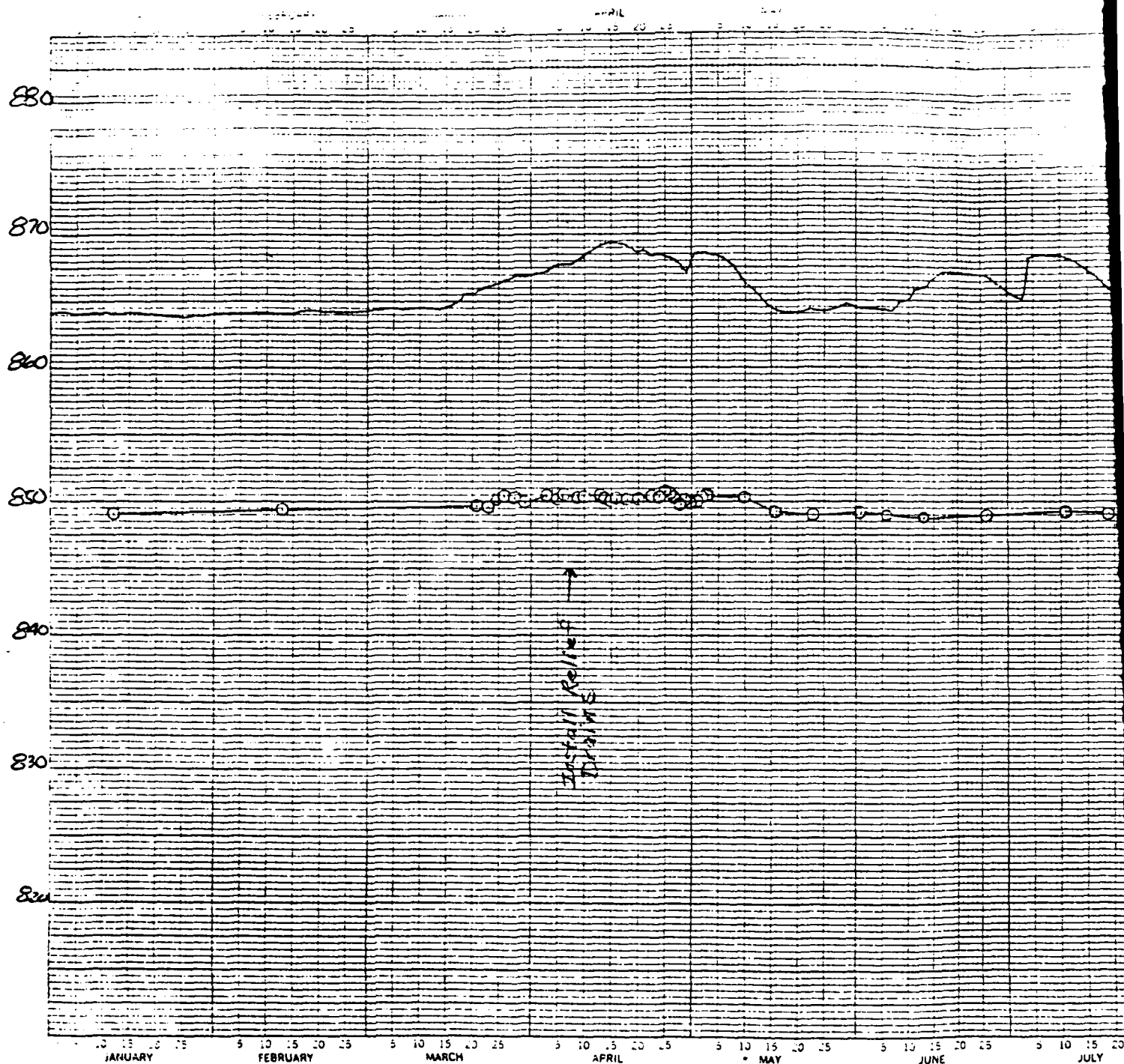
GRND EL 841.4

MAT'L CLY GRVY SD

APPENDIX B Plate No. 10 RP-3-1730

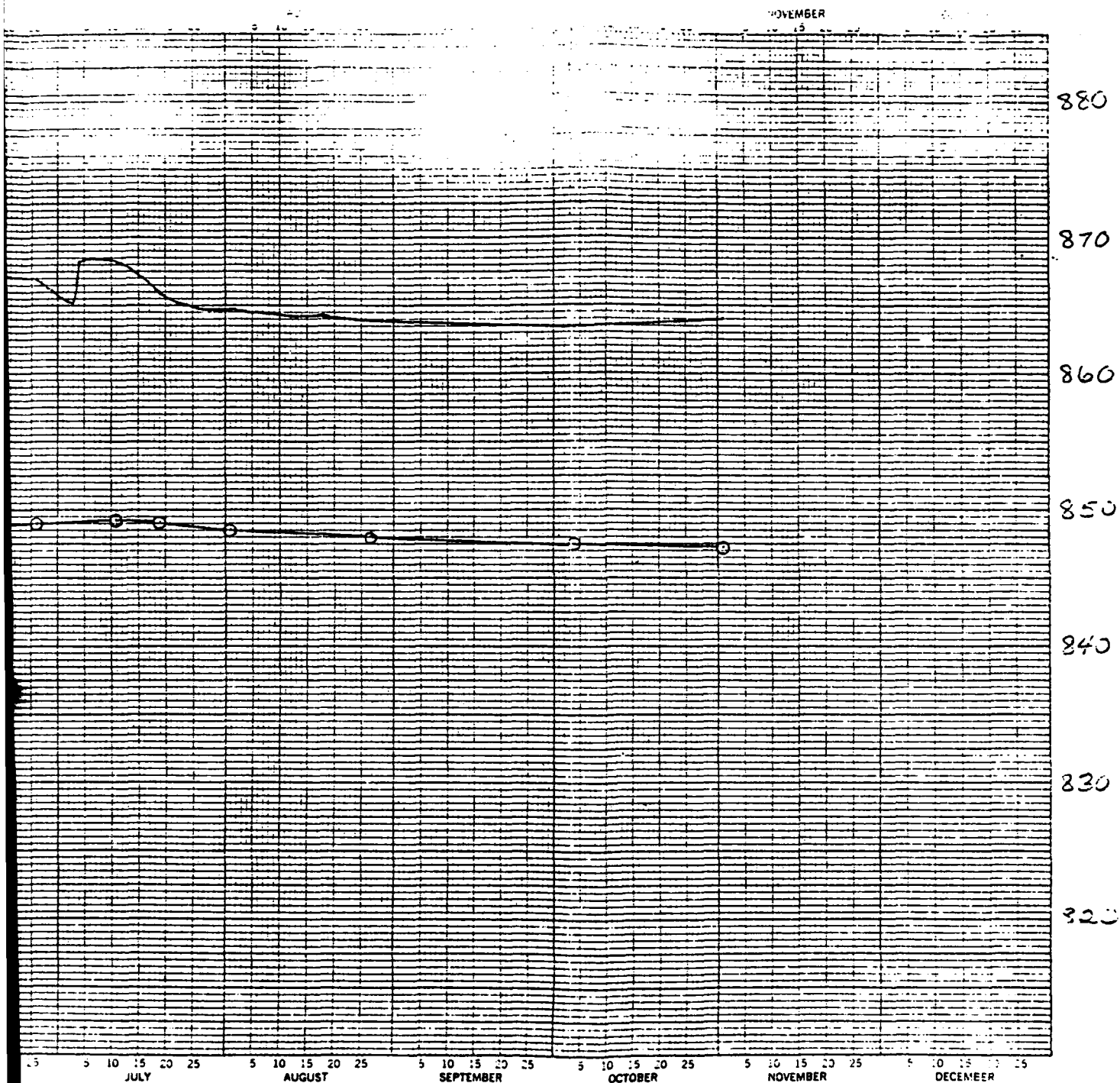
1-2 1 YEAR BY LAYS X 100 DIVISIONS
NEUPILL & L-55M CO. MADE IN U.S.A.

47 2813



1984

15-1-7



984

STA
RNG
MAT'L

SIY GRV SD

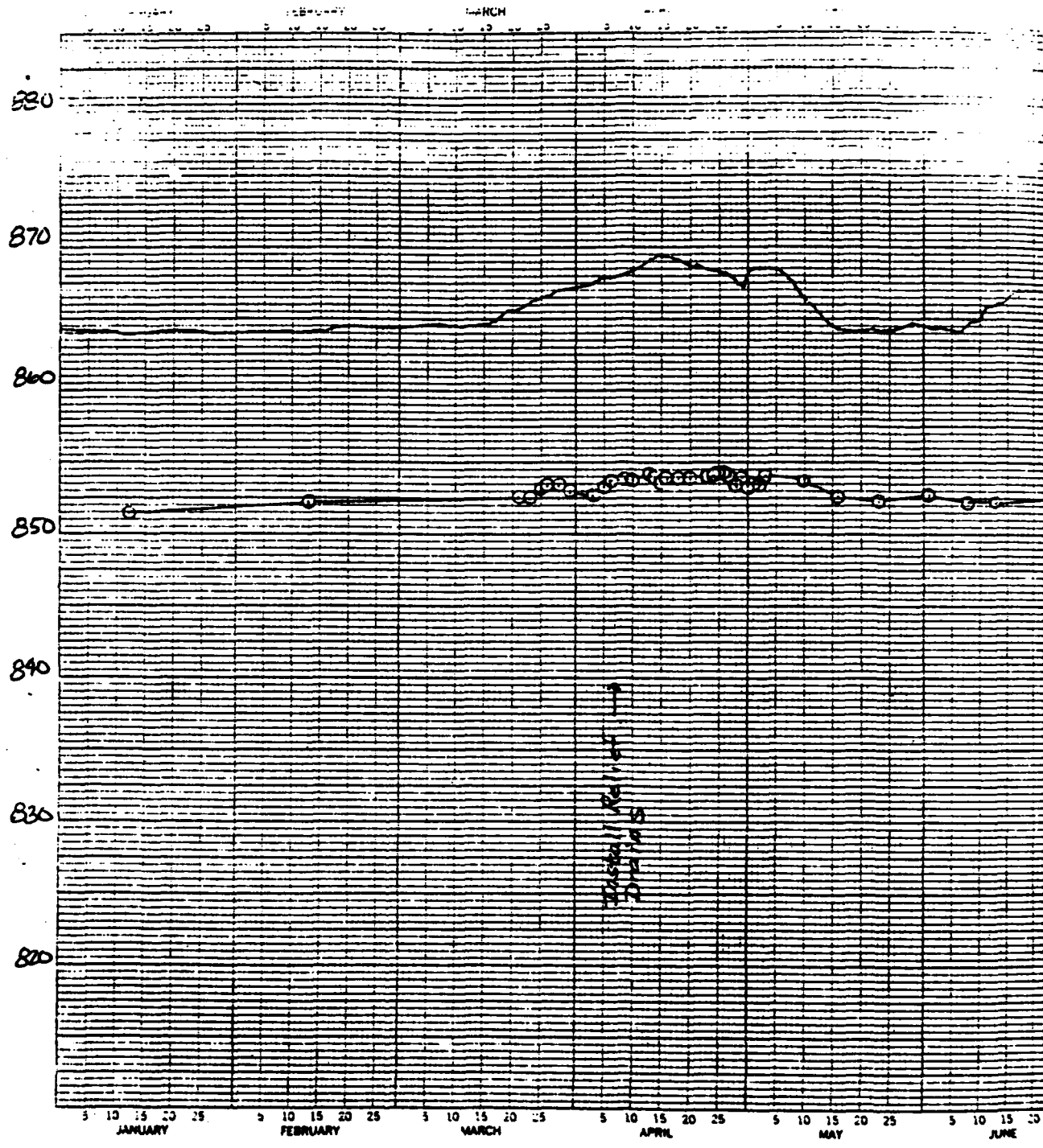
TIP EL 807.6
GRND EL 877.1

APPENDIX B Plate No. 11 RP-3-1731

2

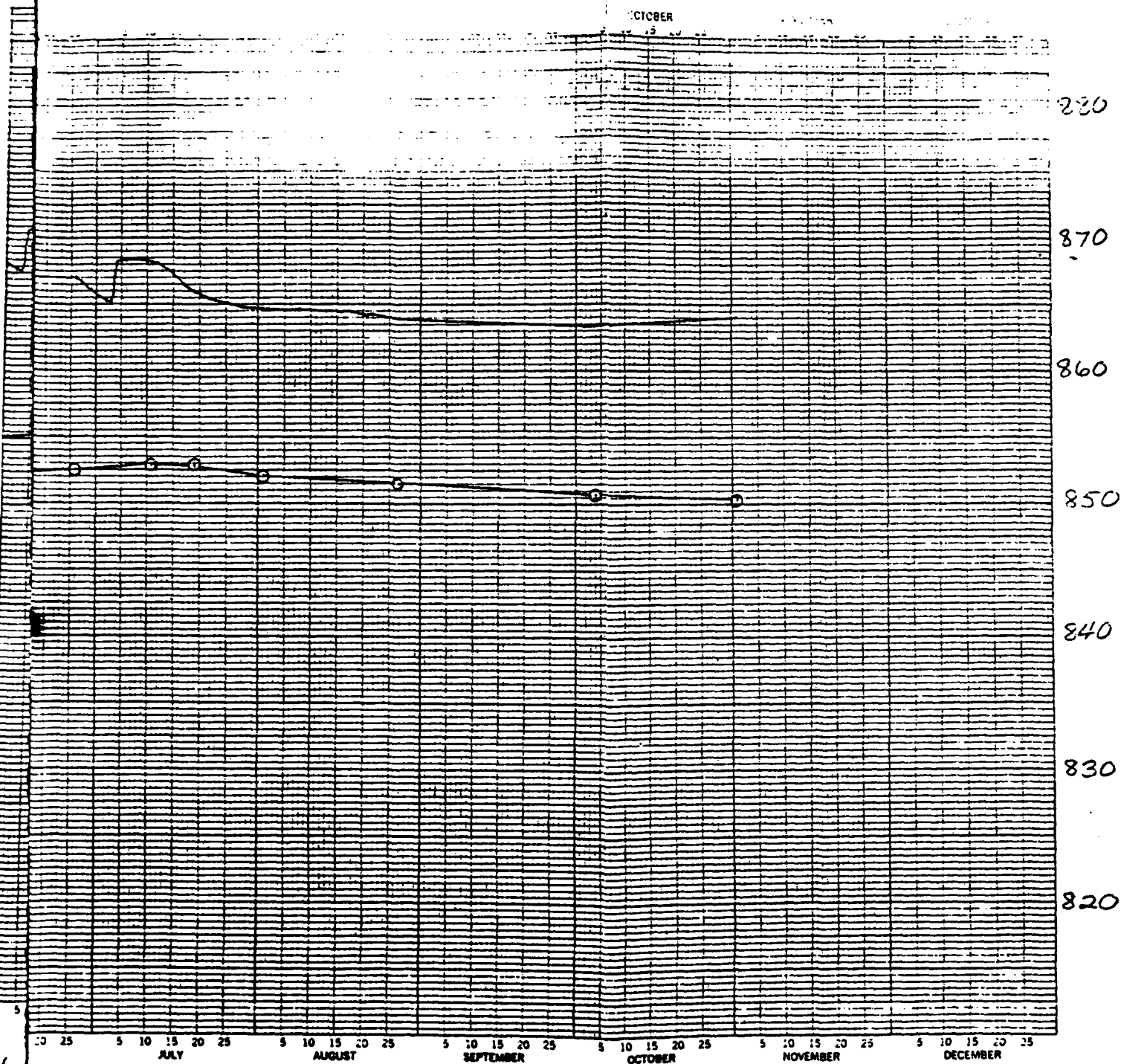
47 2813

1-2E 1 YEAR UV RAYS X 150 DIVISIONS
NEWELL & FISHER CO. MADE IN U.S.A.



Dashed Relief Division

15-1-7



1984

D-516

STA

TIP EL 812.7

RNG

GRND EL 892.9

MAT'L

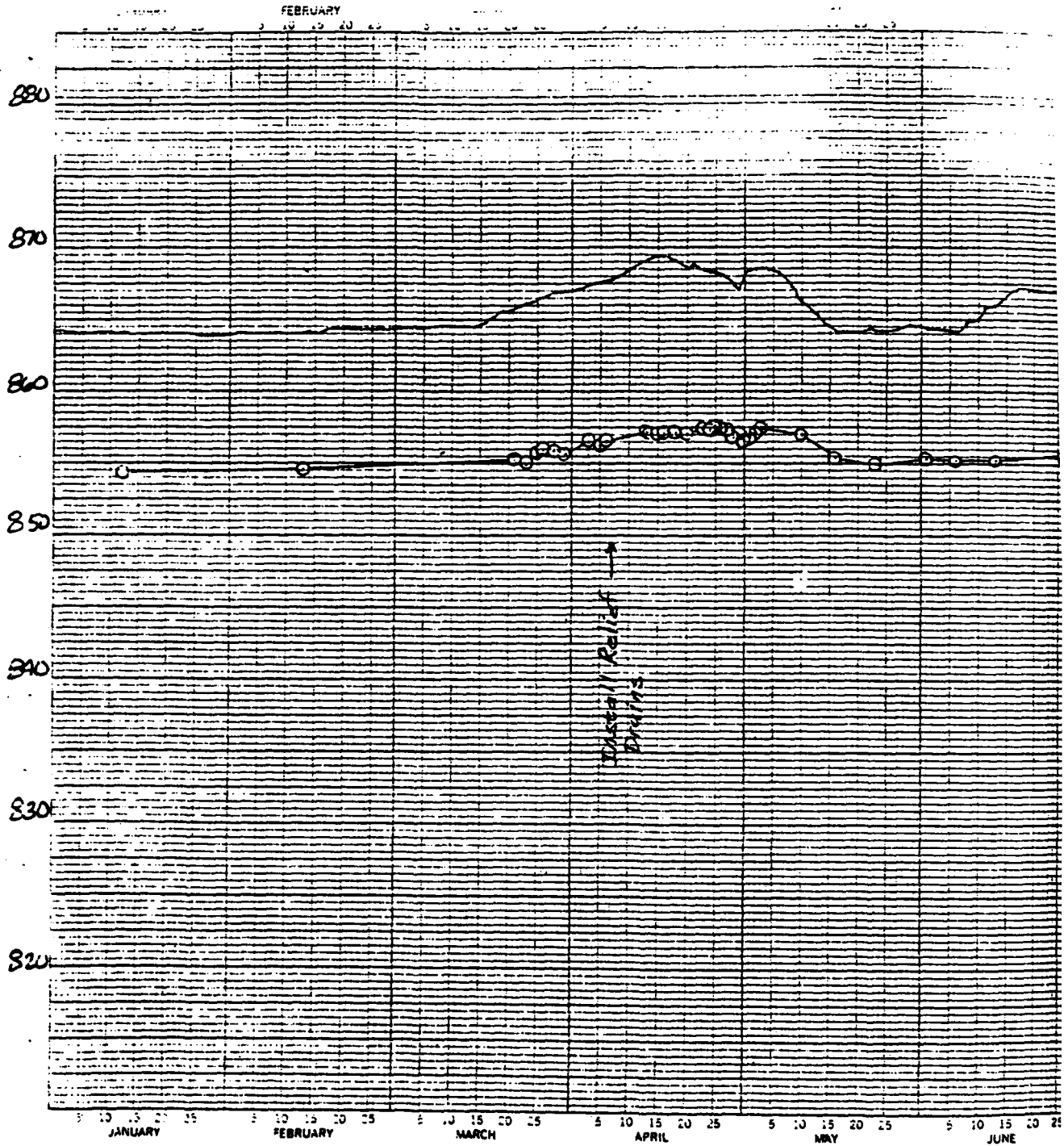
SIY GRV SD

APPENDIX B

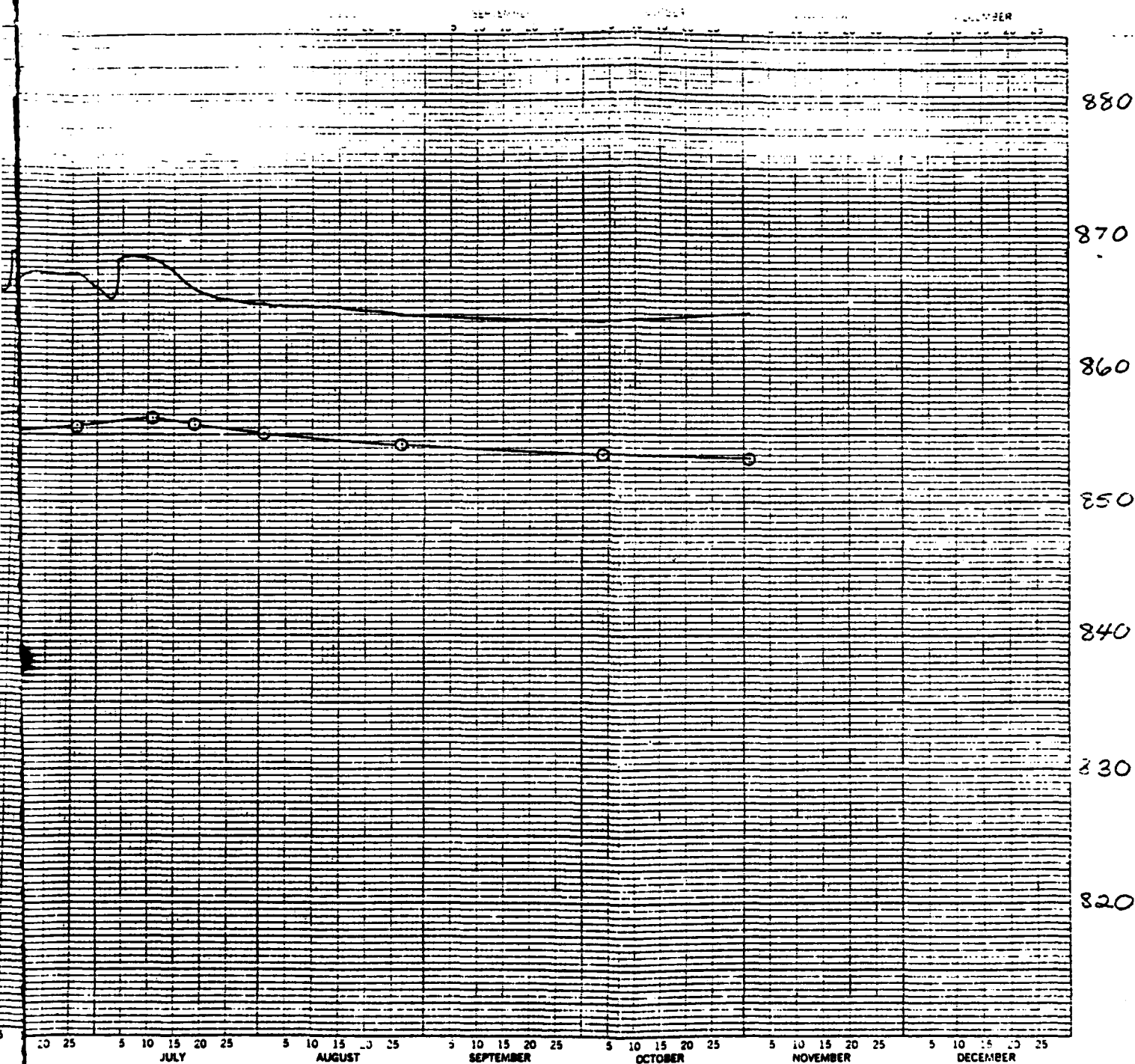
Plate No. 12 RP-3-1732

47 2813

ICE 1 YEAR BY DAYS X 150 DIVISIONS
 AUGUST 8 1950 TO MAY 1951



15-1-7



1984

DC-517

STA

TIP EL 812.9

RNG

GRND EL 880.0

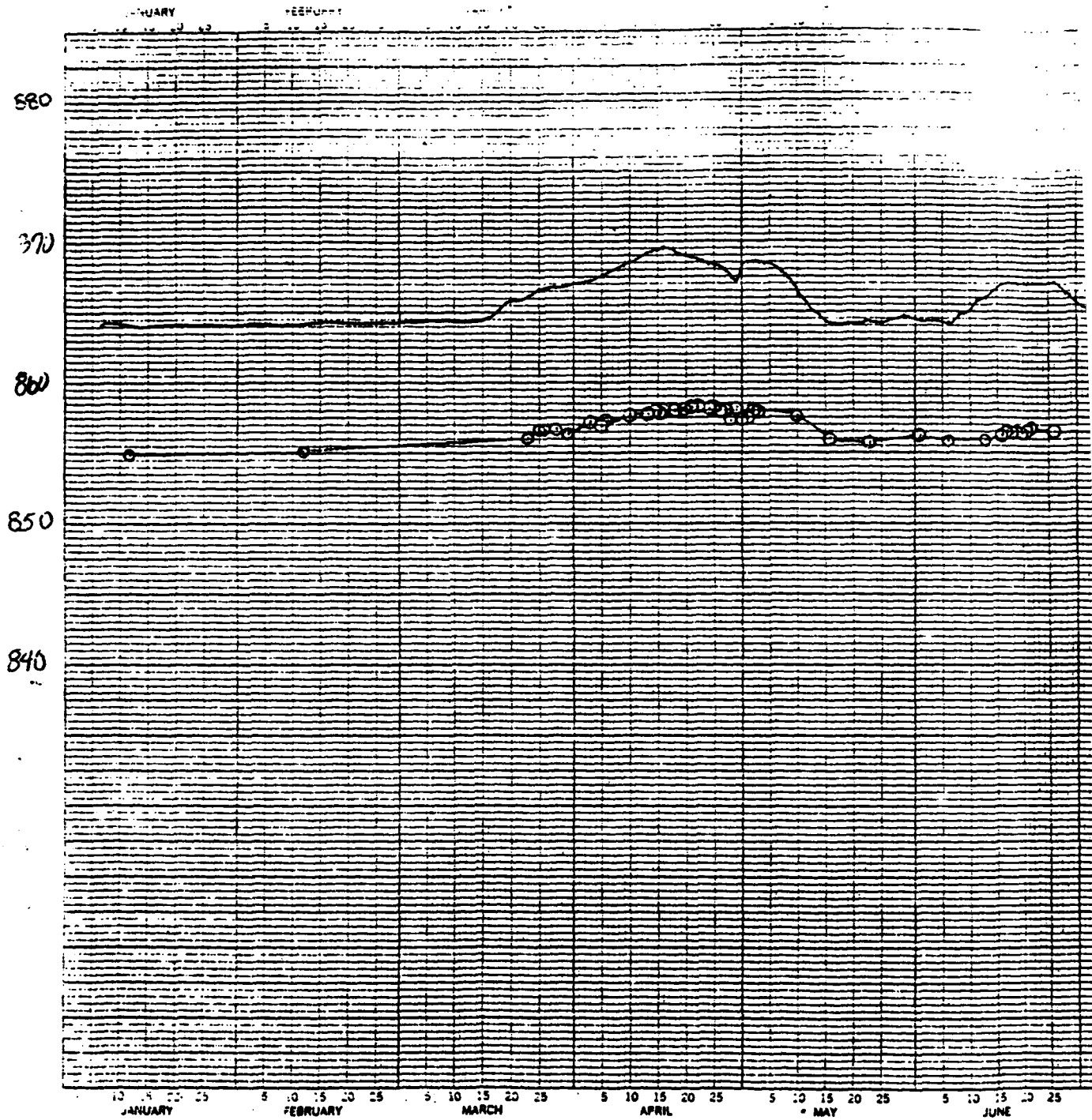
MAT'L SIY GRV SD

APPENDIX B Plate No. 13 RP-3-1733

2

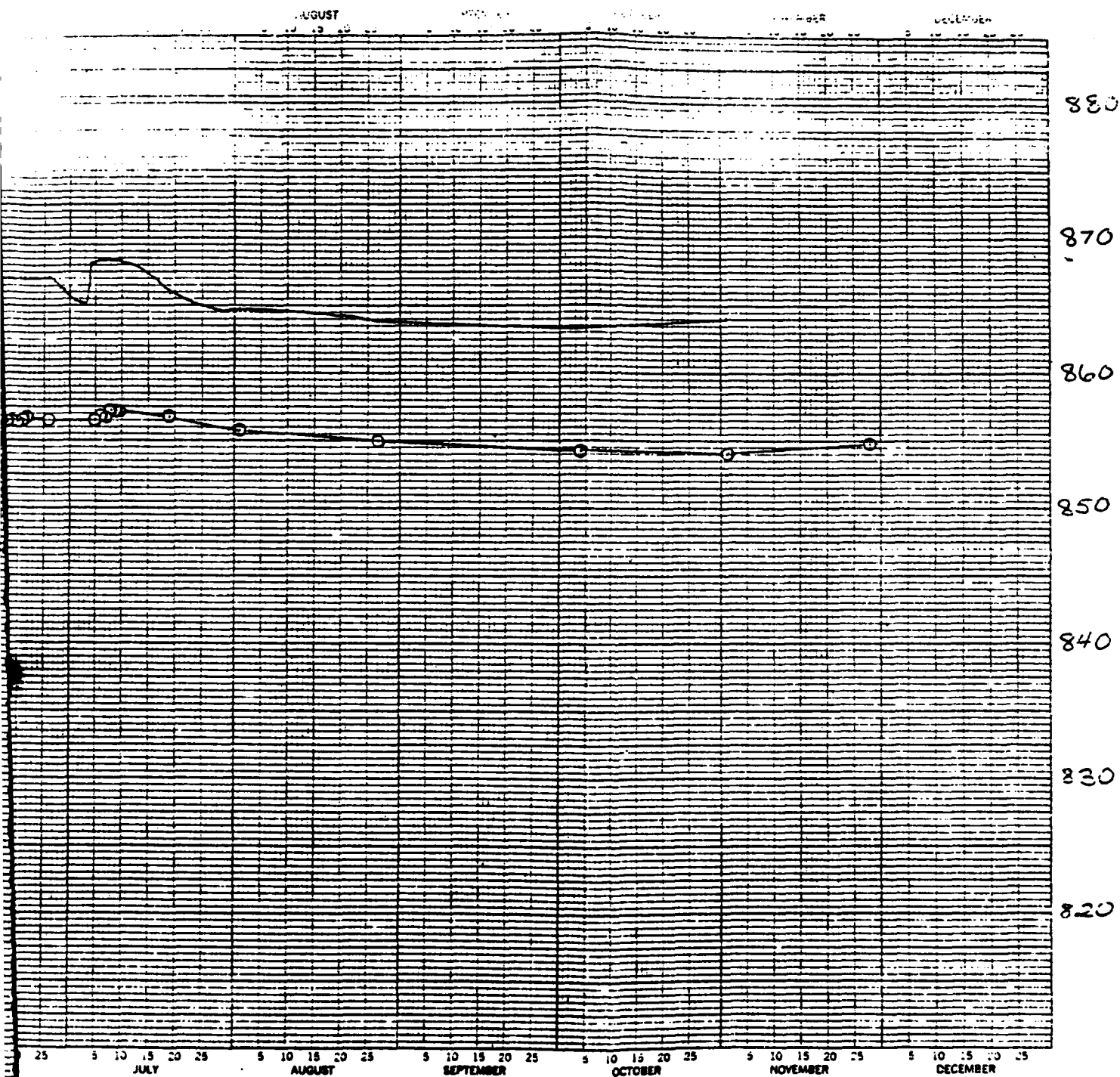
47 2813

12-2 1 YEAR BY 1/2° X 1/2° DIVISIONS
KALIBRE 8 150MM CO. MADE IN USA



1984

15-1-7



1984

STA
RNG
MAT'L

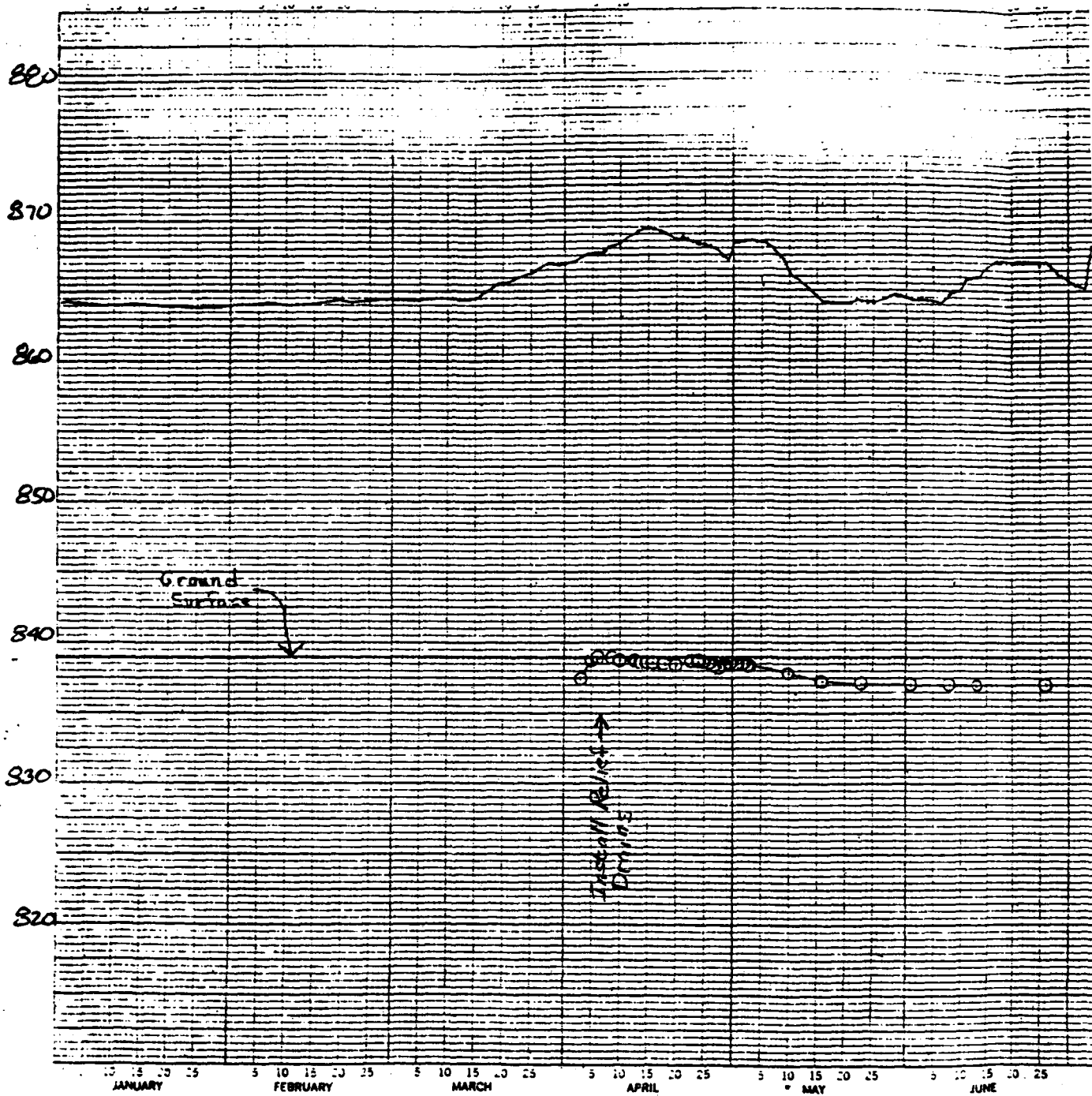
TIP EL 823.3
GRND EL 896.3
SDY CLY GRV

APPENDIX B Plate No. 14 RP-3-1734

7

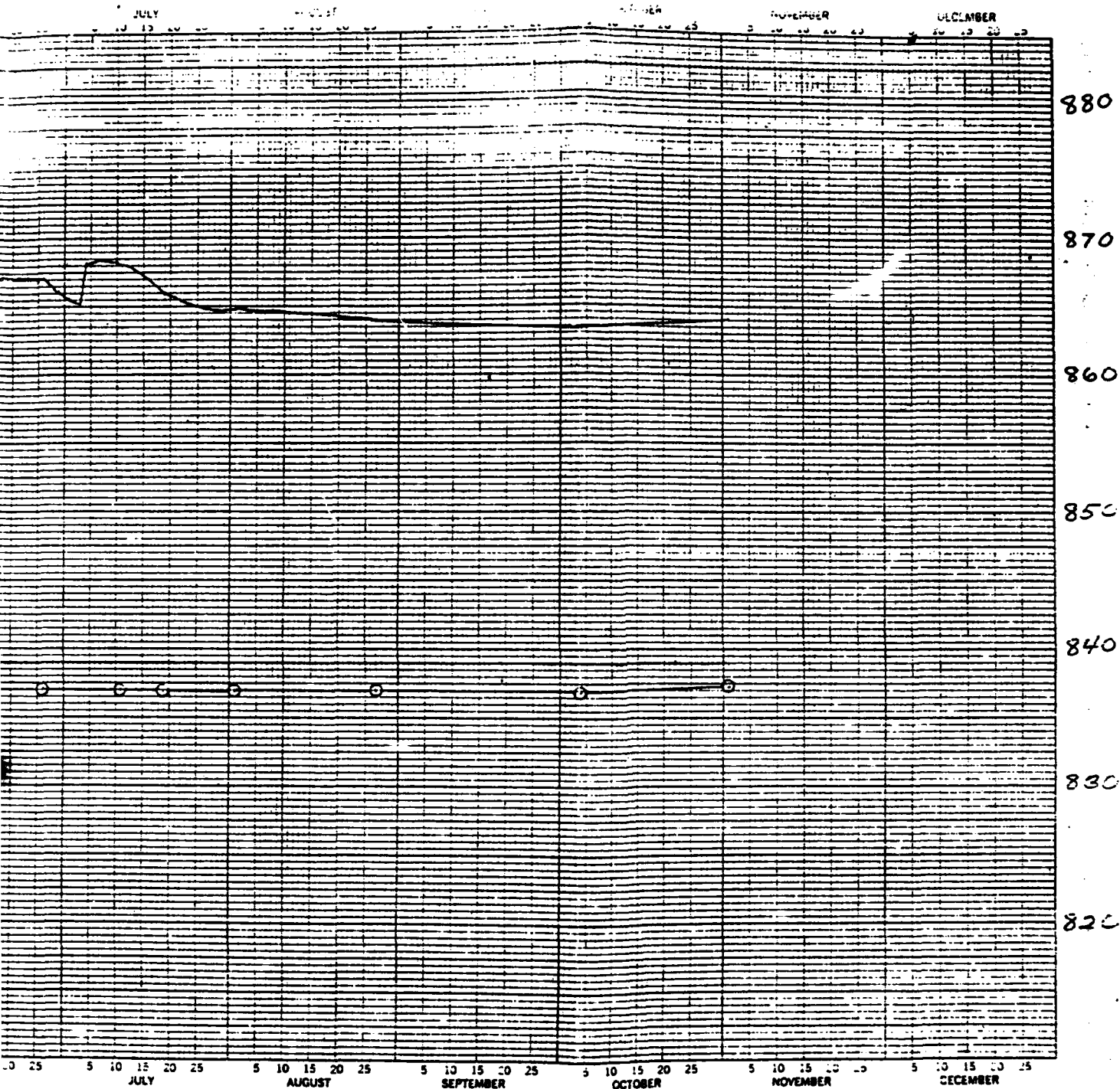
47 2813

16-E 1 YEAR HYDRA X 10 DIVISIONS
REDFIELD & SENSEN CO. MADE IN U.S.A.



1984

15-1-7



1984

D-519A
 STA TIDE L 860.7
 RING GRND EL 872.7
 MAT'L SDY LEAN CL RP-3-1735
 APPENDIX B Plate No. 15

2

47 2813

16.2 FATHOMS, 16.2 DIVISIONS
16.2 FATHOMS, 16.2 DIVISIONS

880

870

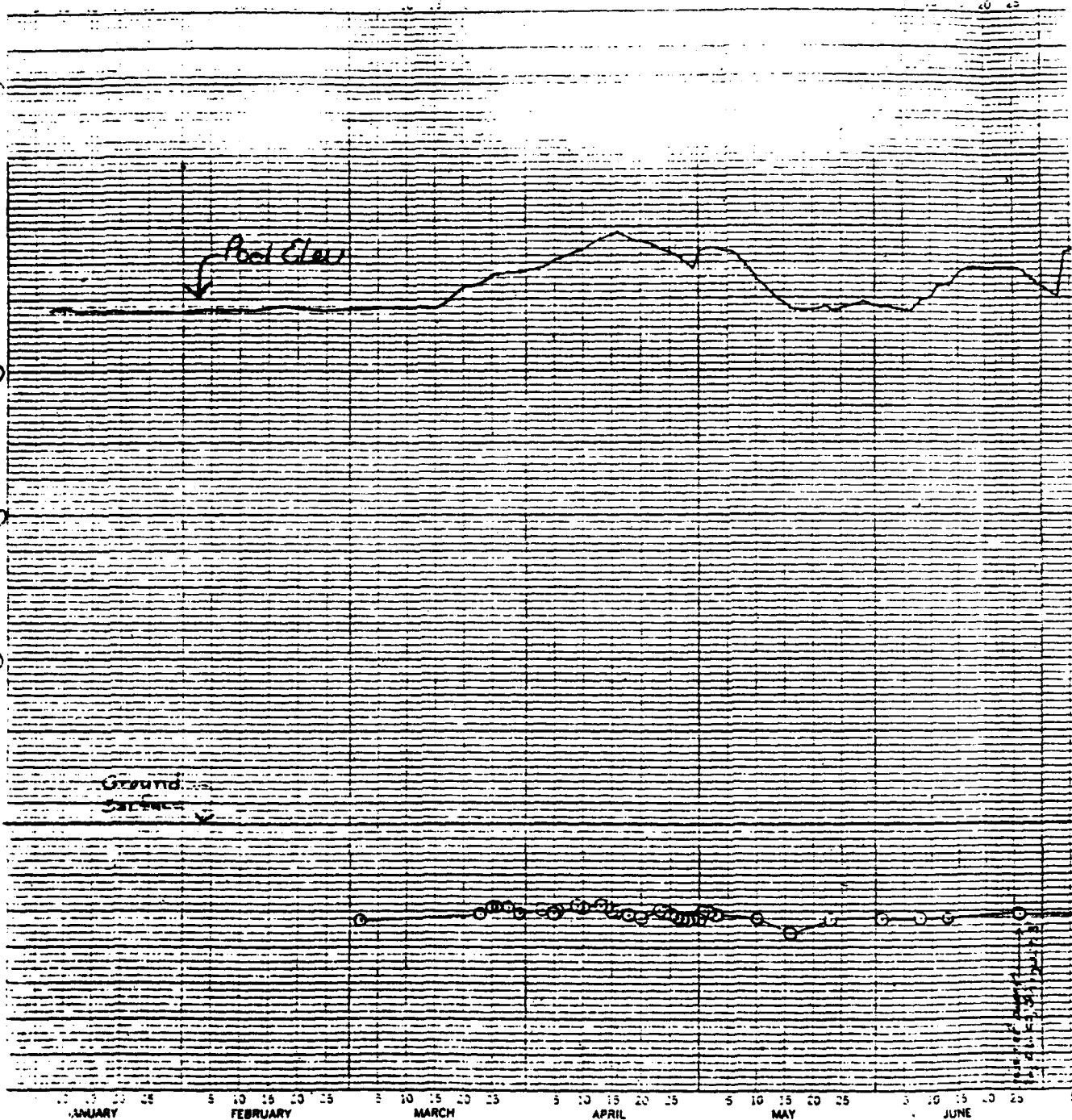
860

850

840

830

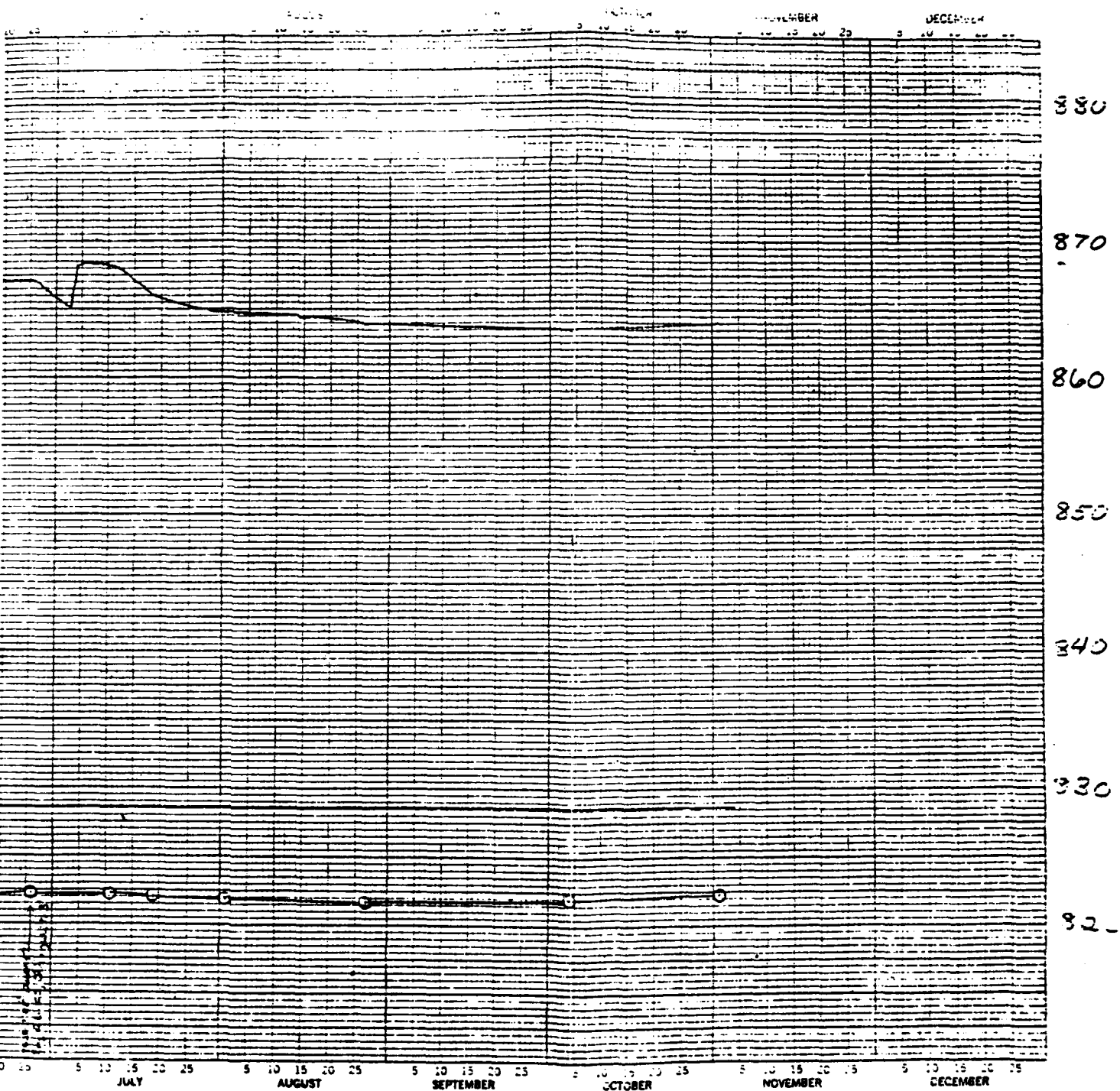
820



1984

15-1-7

1



1984

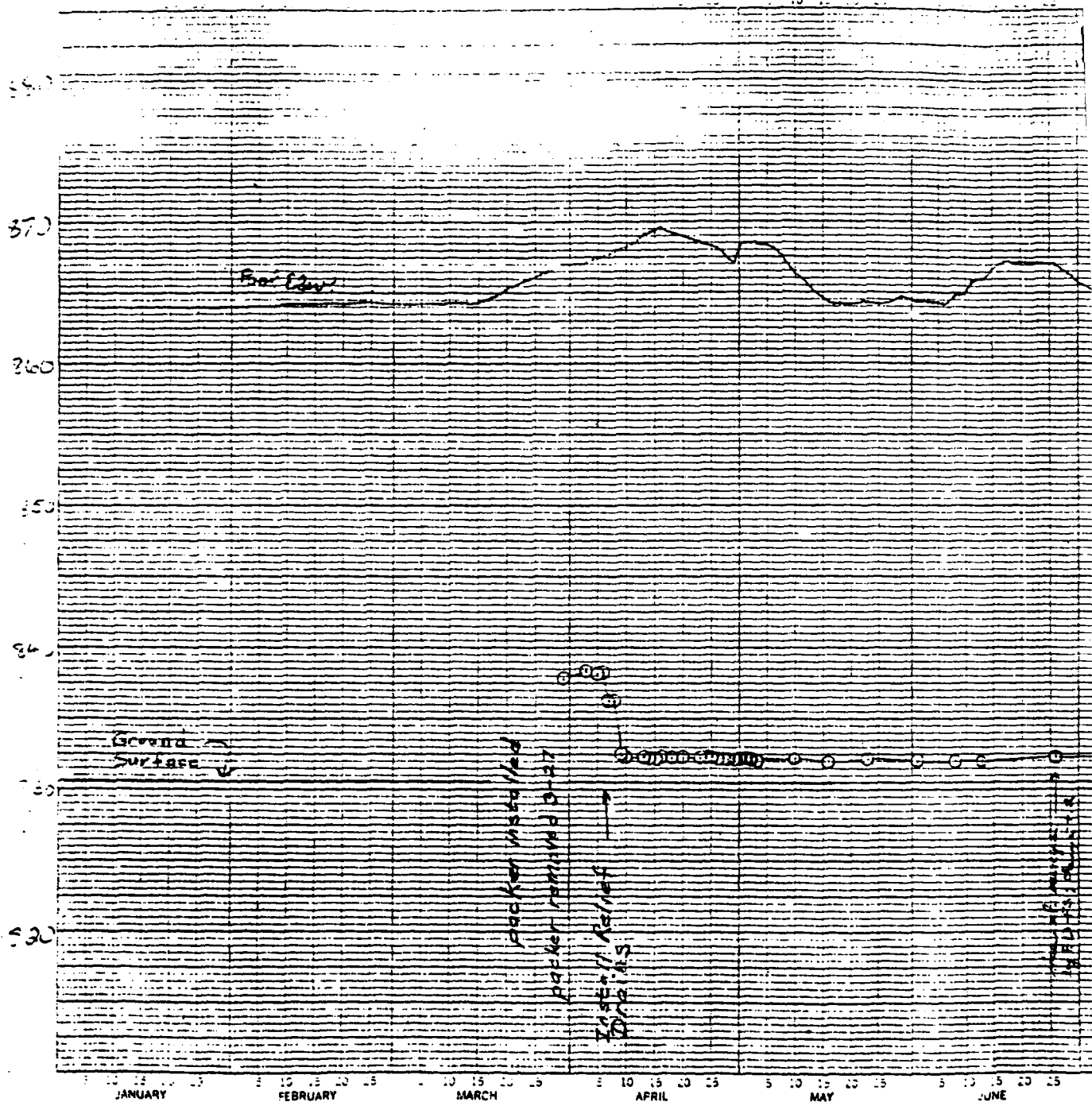
APPENDIX B PLATE NO. 16

STA
RNG
MAT'L

A-520 RP-3-1736
TIP EL 812.1
GRND EL 828.6
SDY CLAY

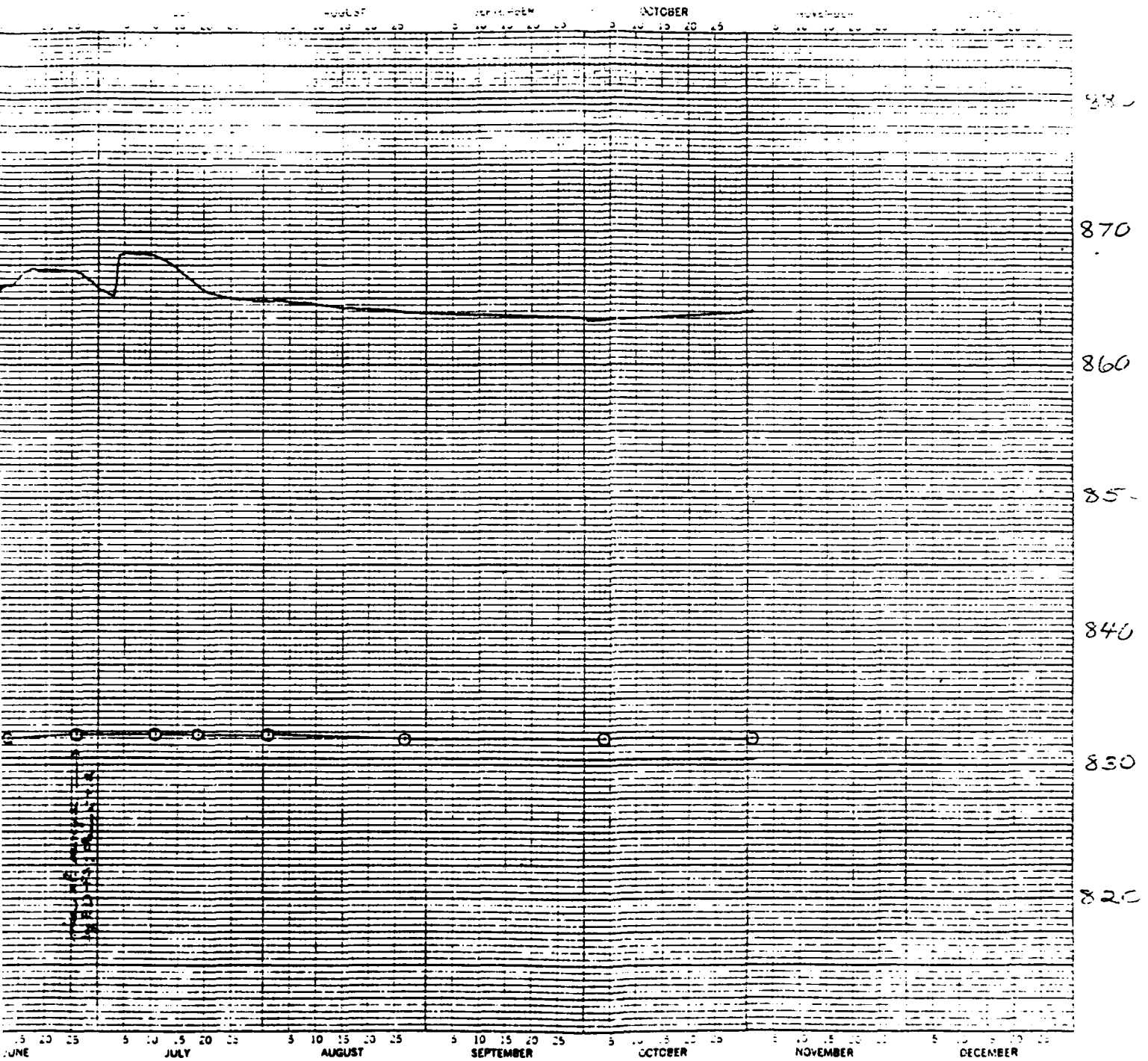
47 2813

ICE DIVISIONS
KASIA A. 120000 100000



15-1-7

1984



1984

APPENDIX B PLATE NO. 17

STA

RNG

MAT'L

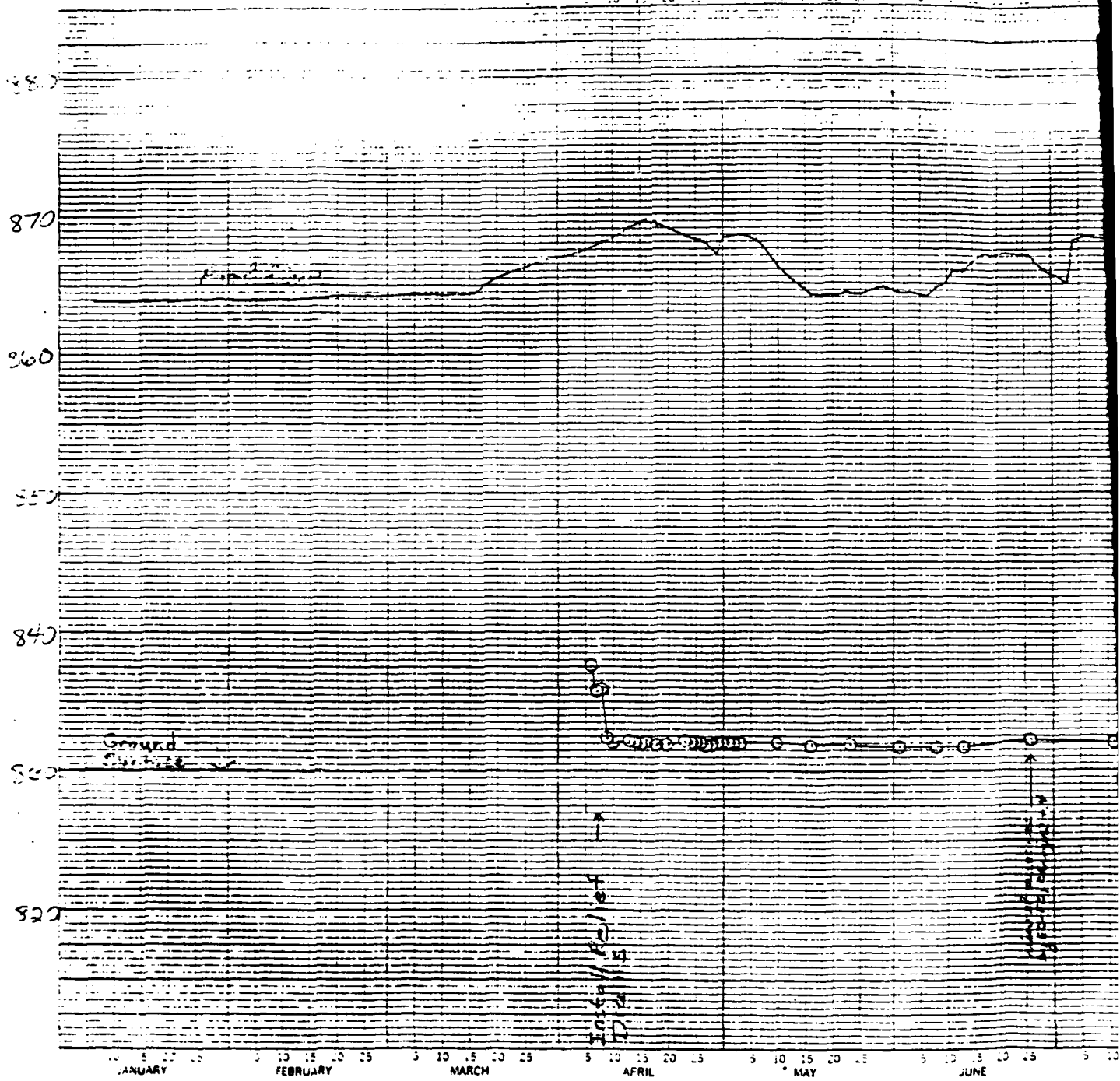
A-521

RP-3-1737

TIP EL 810.2

GRND EL 830.2

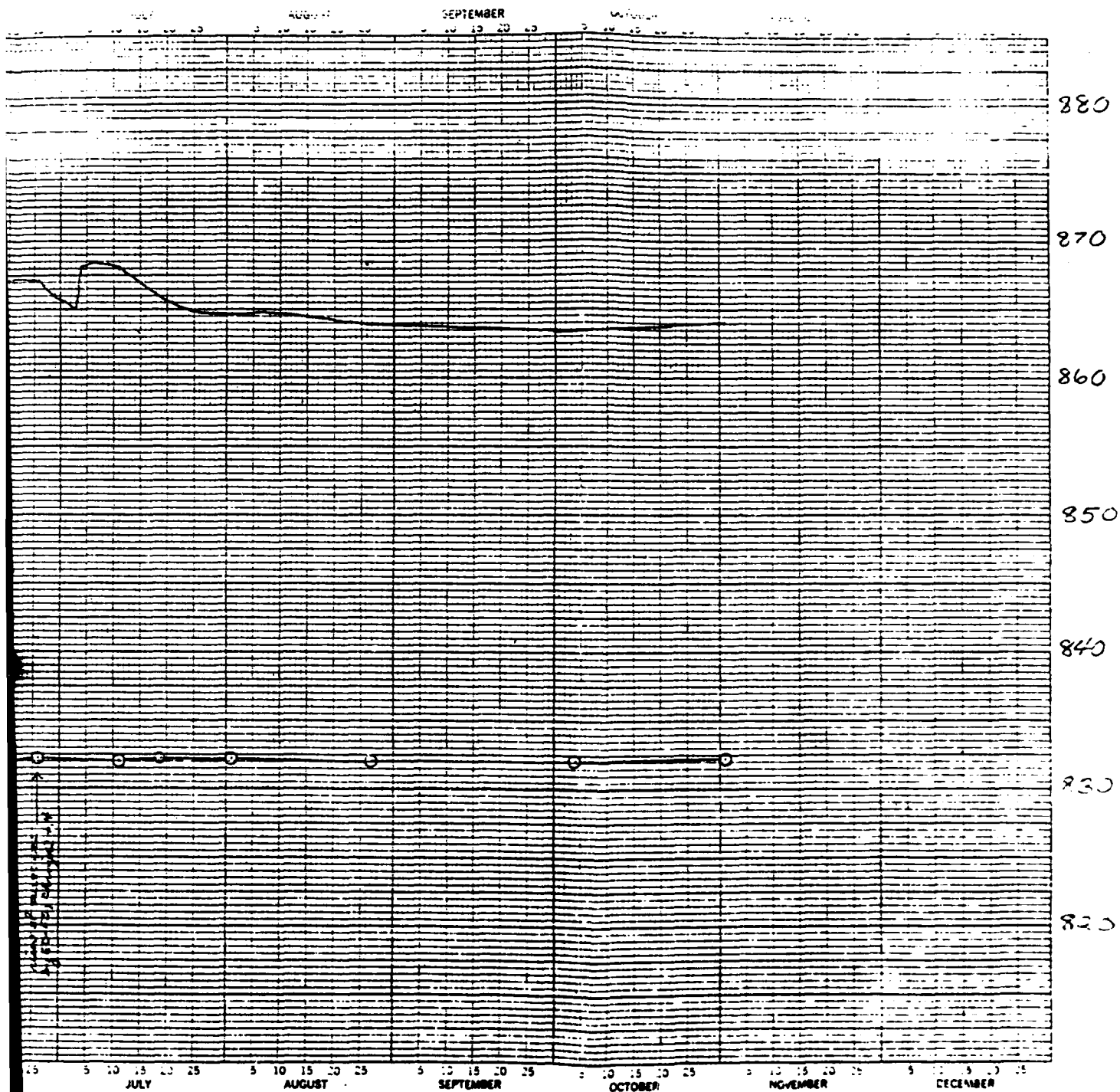
STN SAINT



1984

15-1-7

1



984

STA
RNG
MAT'L

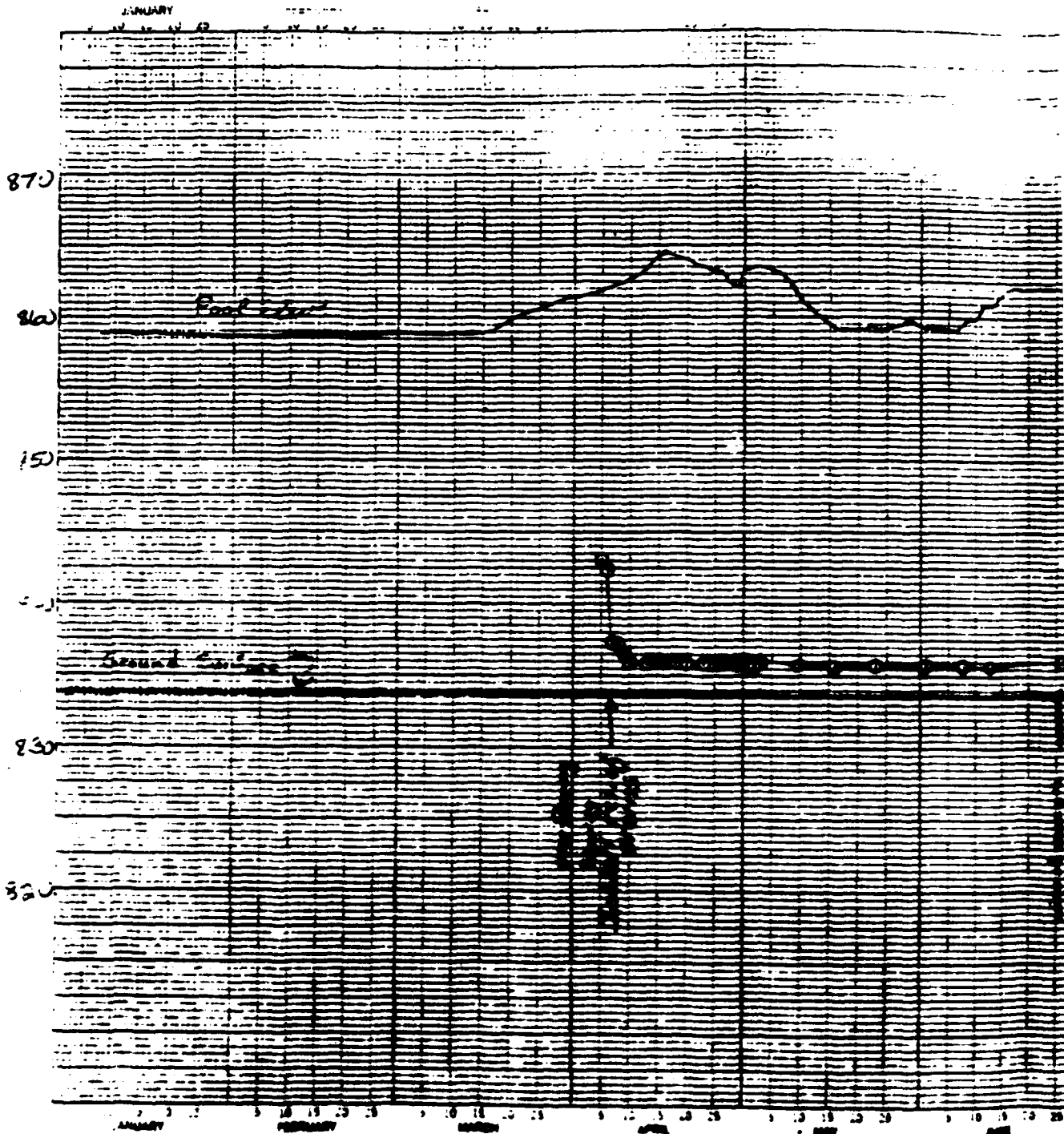
TIP EL 809.6
GRND EL 830.0
RP-3-1738

APPENDIX B Plate No. 18

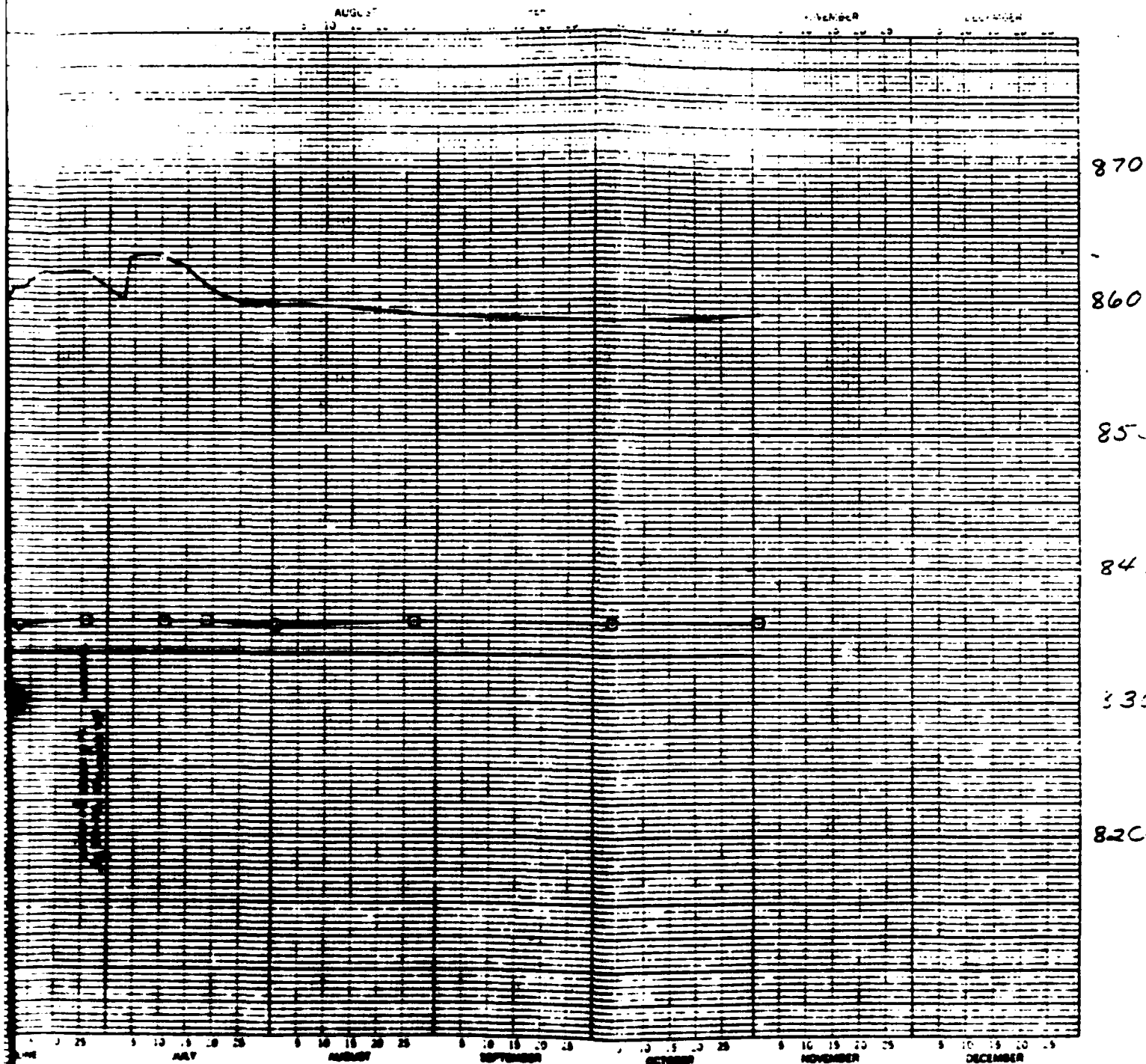
2

47 2813

10-E 1 TEAM MEASURES 4 150 DIVISIONS
REMOVED & LOWER CO. 1000 M. 1000



15-1-7

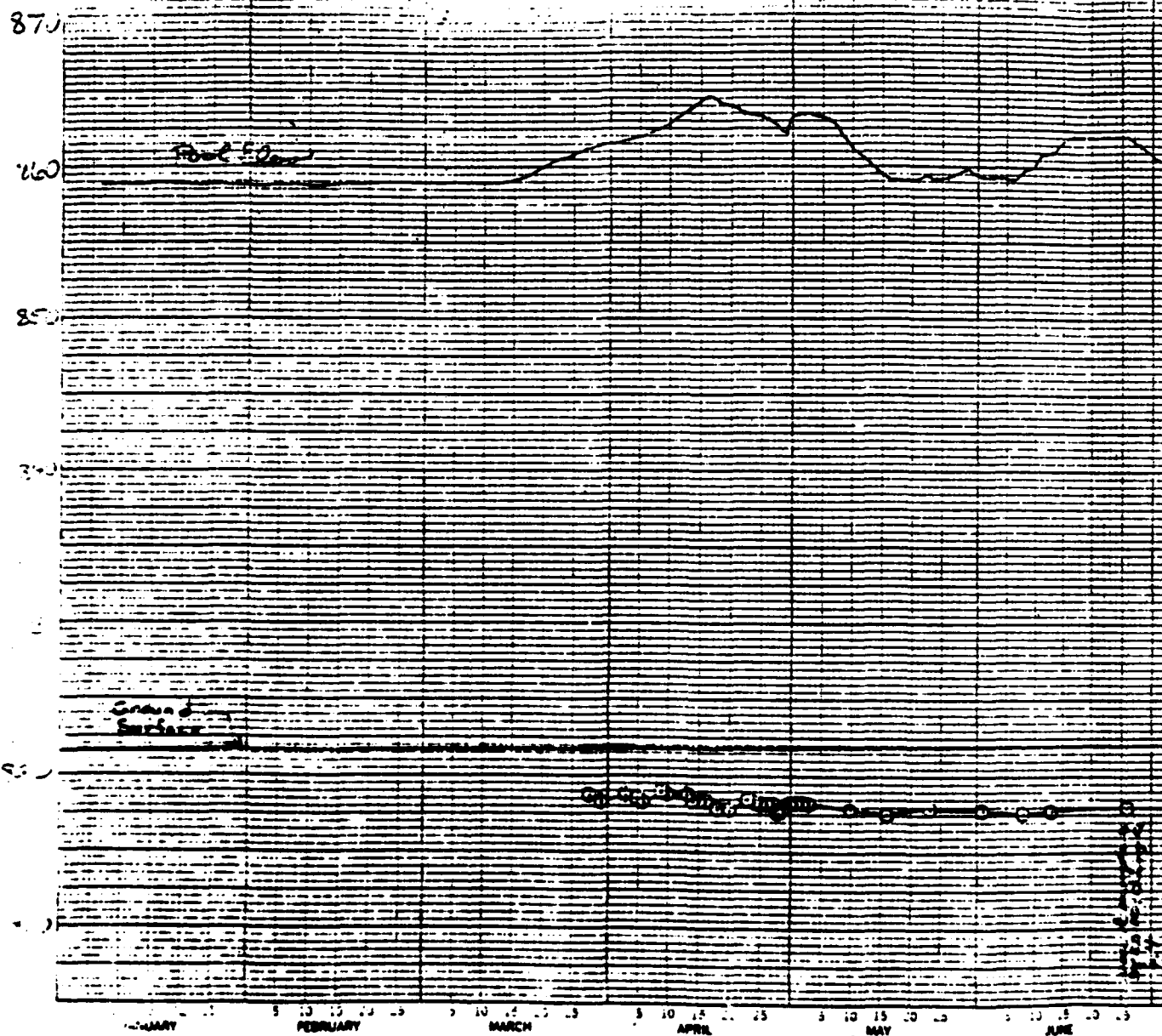


1984

2-552
 STA TIE EL 812.2
 RNC GRN EL 824.1
 MAT'L CLY SAND RP-3-1739
 APPENDIX B Plate No. 19

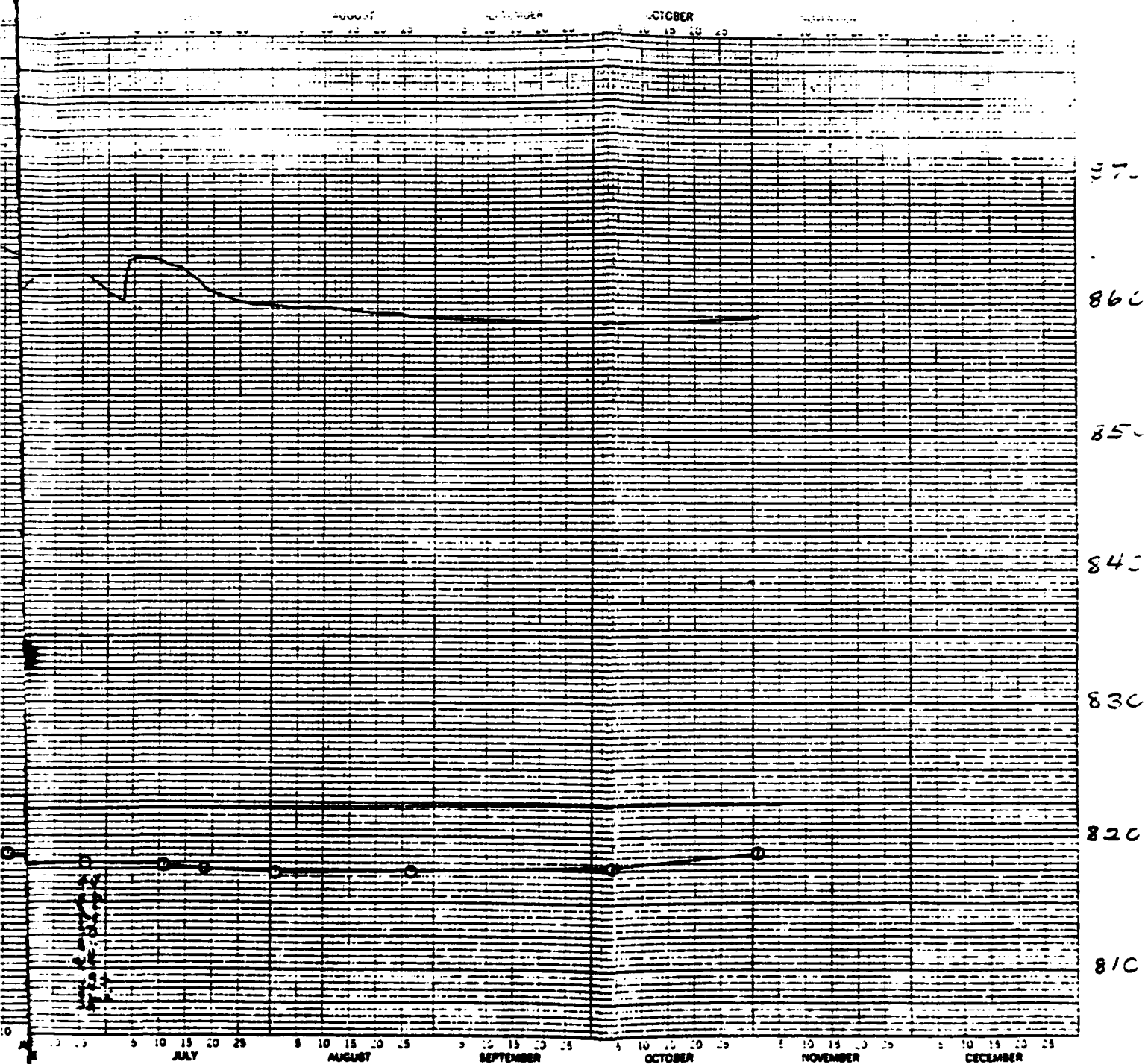
47 2813

ICE PRODUCTION DIVISION



1984

15-1-7



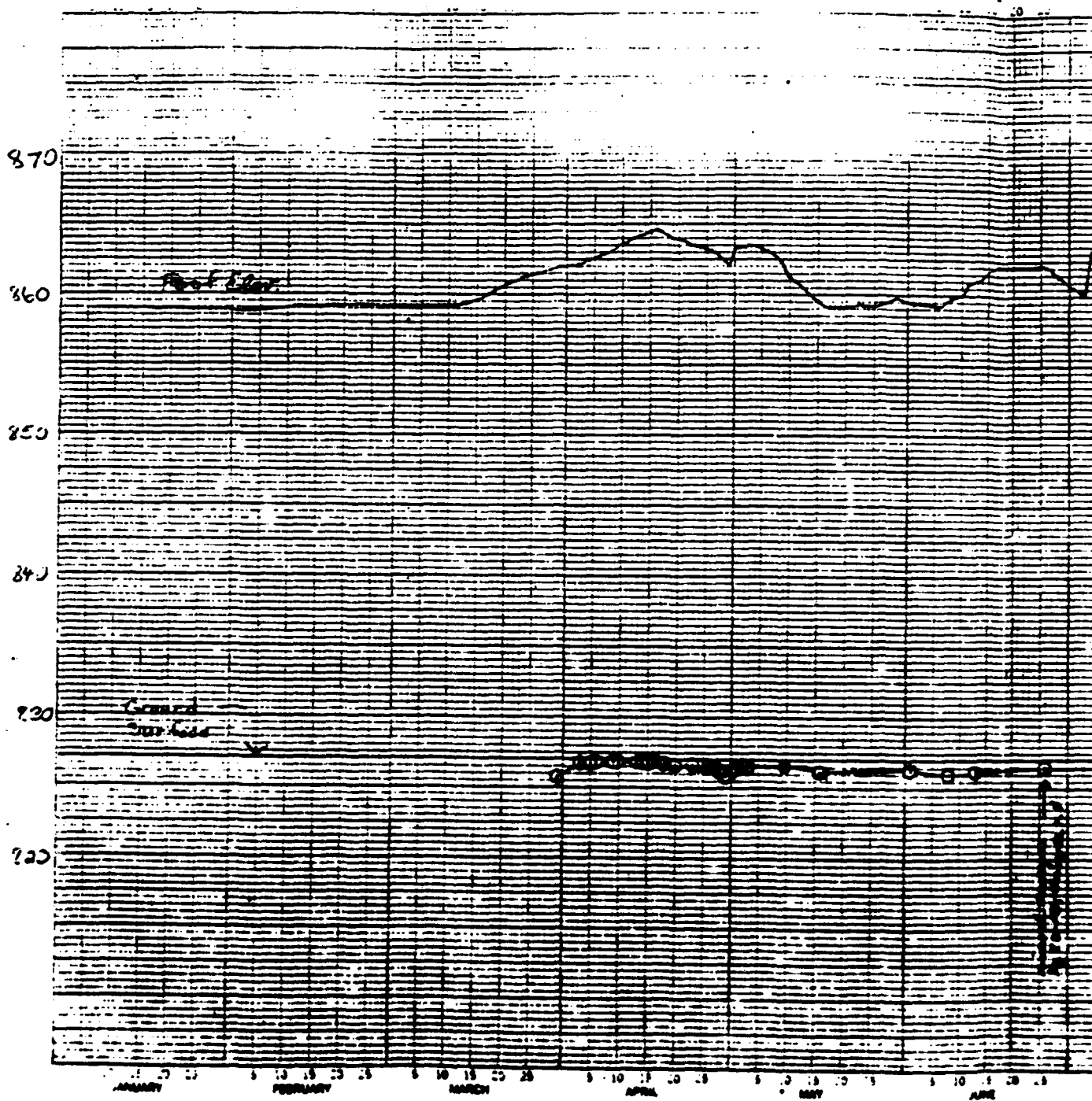
1984

STA TPEL 723.4
 RING GRNG EL 861.6
 MAT'L SDY CLAY RP-3-1740
 APPENDIX B Plate No. 20

2

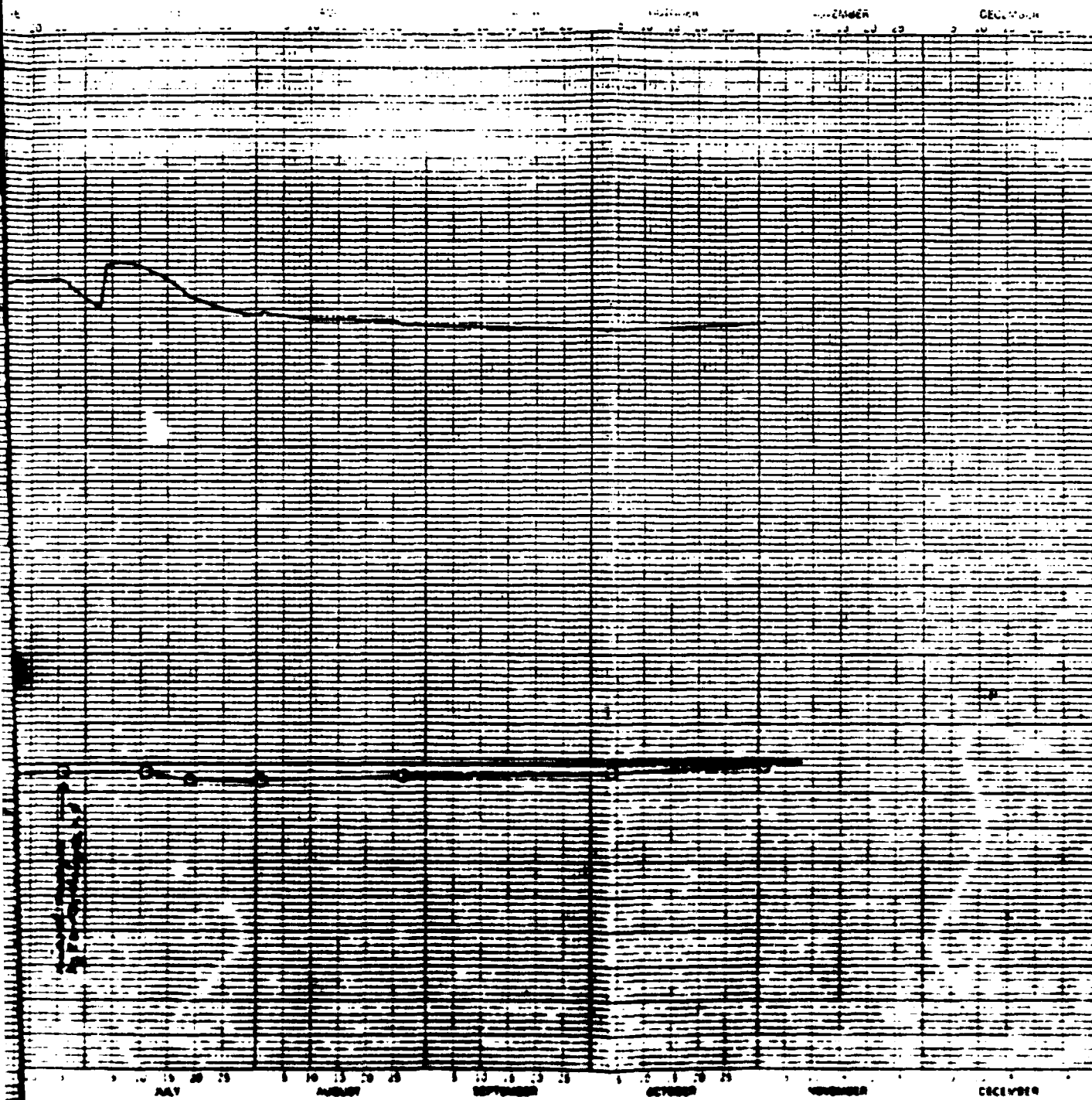
47 2813

ICE 1 YEAR BY DAYS X 10 DIVISIONS
 MEAN & 10% TO 10% MEAN



1954

15-1-7



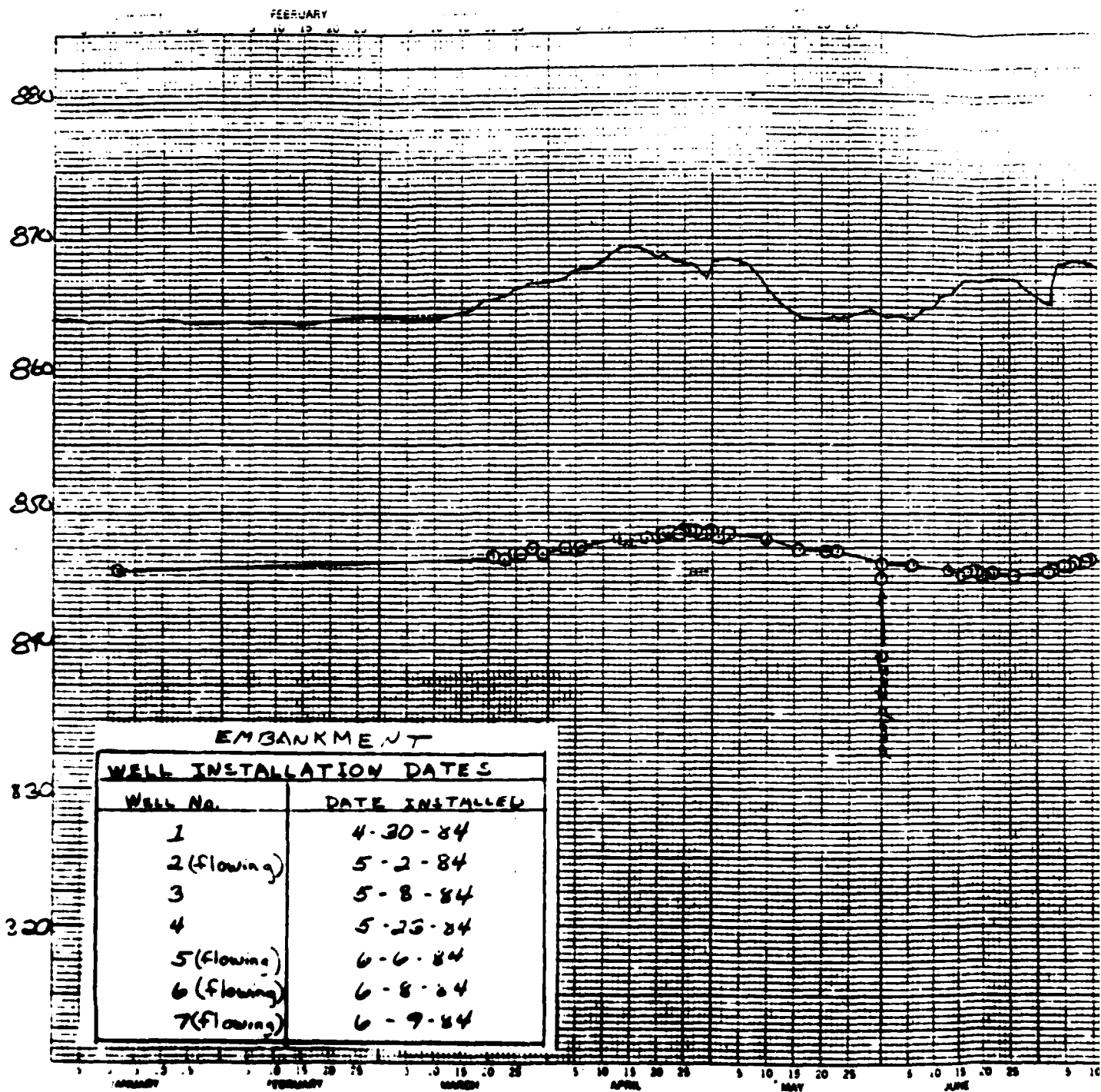
1954

STA TIP EL 2120
 RIG GAGE EL 8670
 MAT'L 13A11 1224 RP-3-1741
 APPENDIX B Plate No 21

1

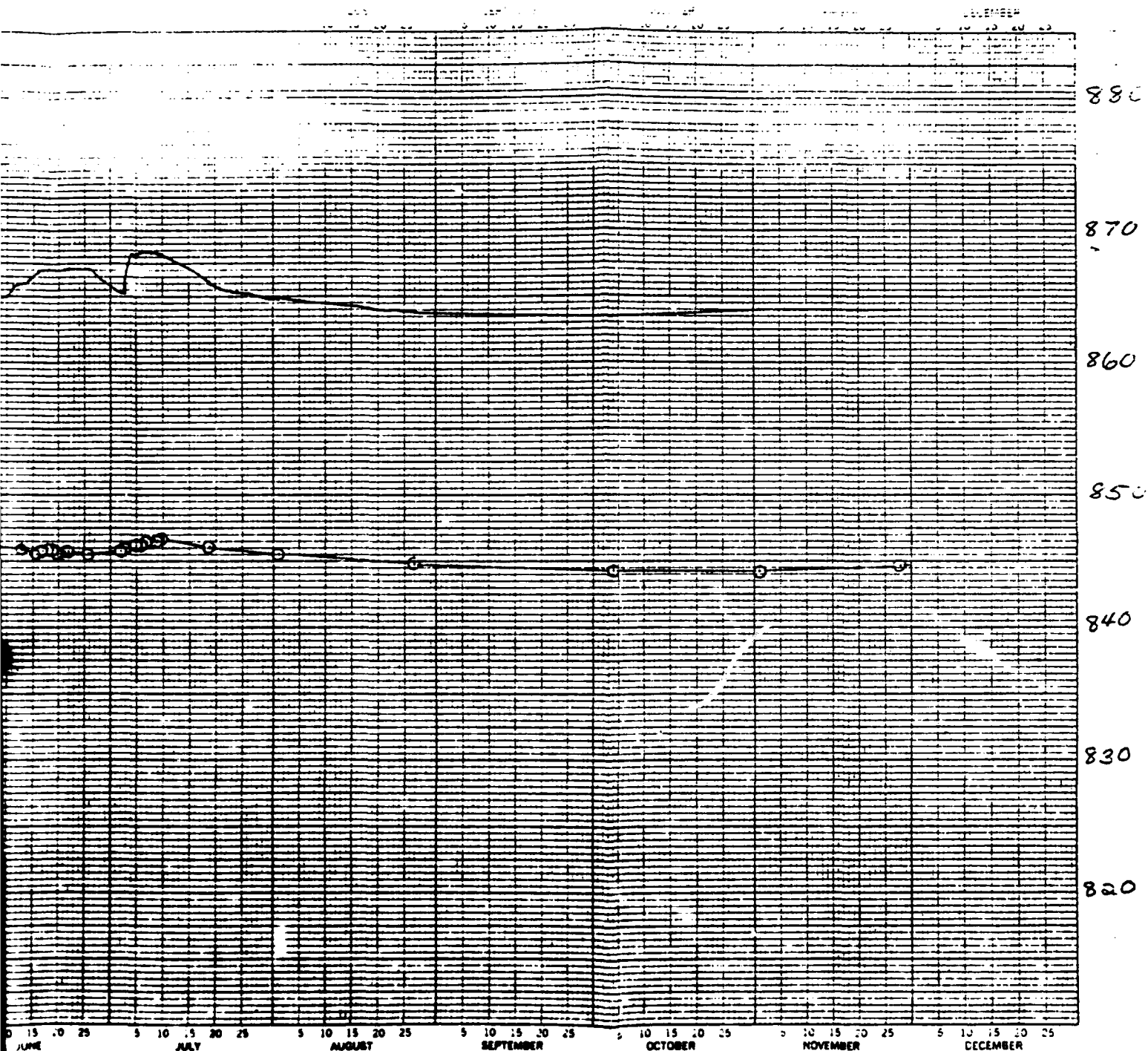
2

47 2813

K-E 1 YEAR BY DAYS A 1/2 DIVISIONS
REUSEL 6 INCH TO 1/2 INCH

1984

15-1-7

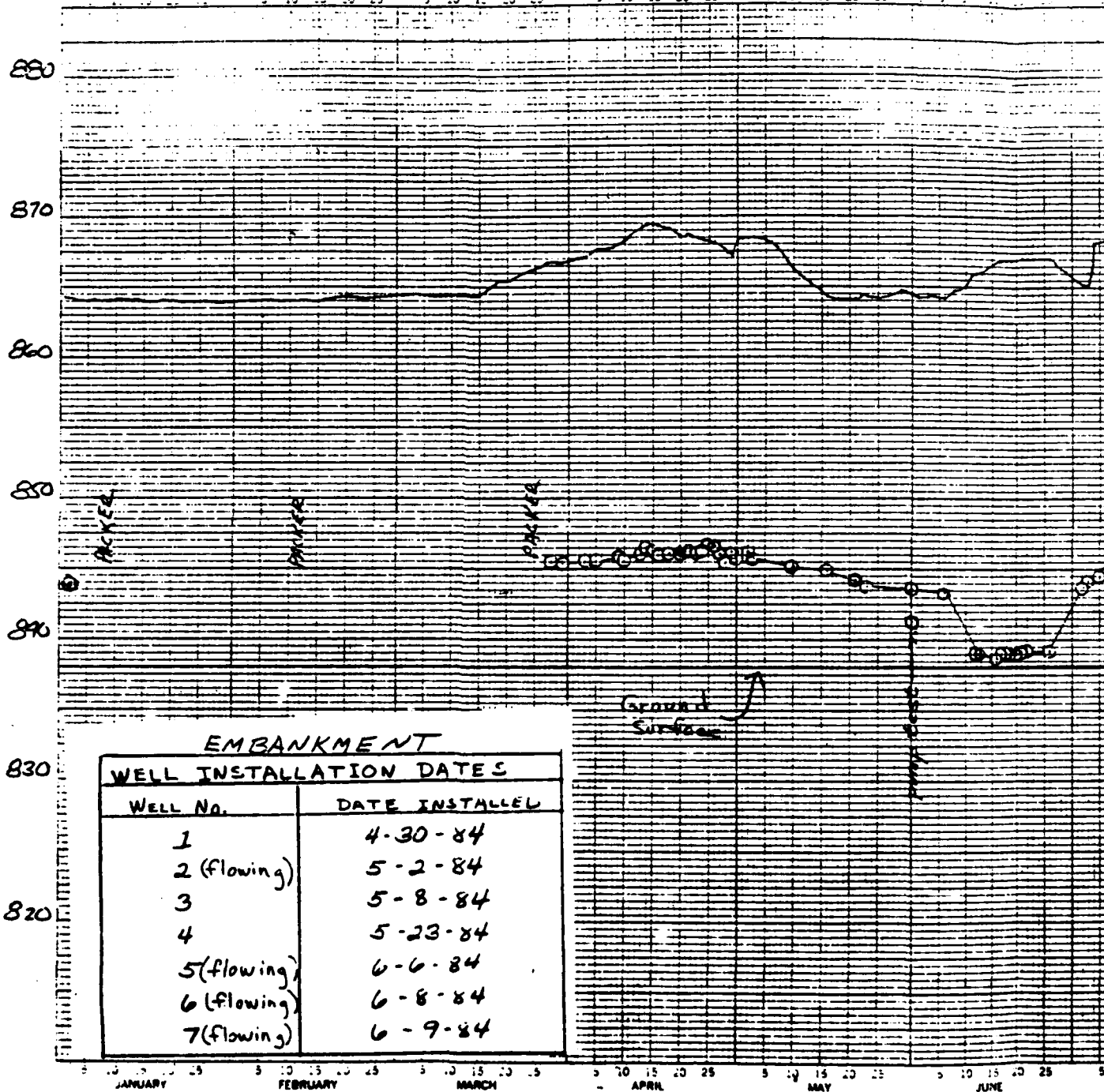


1984

P-106-4
 STA 106+02 TPI EL 807.9
 RNG 15D GRND EL 894.9
 MAT'L SIY GRV SD RP-3-1742
 APPENDIX B Plate No. 22

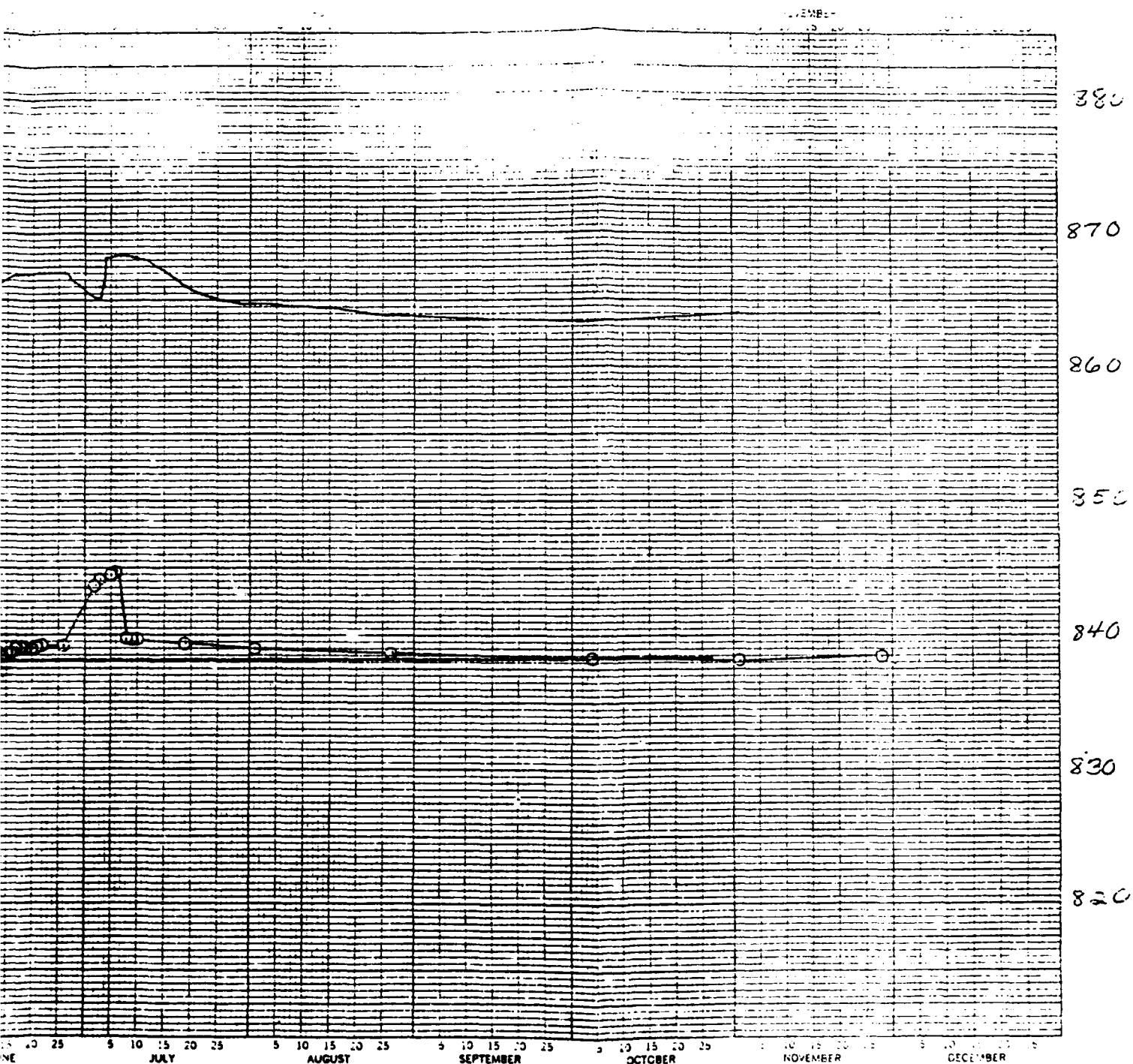
47 2813

16-E 1 YEAR BY DAYS X 151 DIVISIONS
NEUFEL & ASSOC. CO. MADE IN U.S.A.



1984

15-1-7



1984

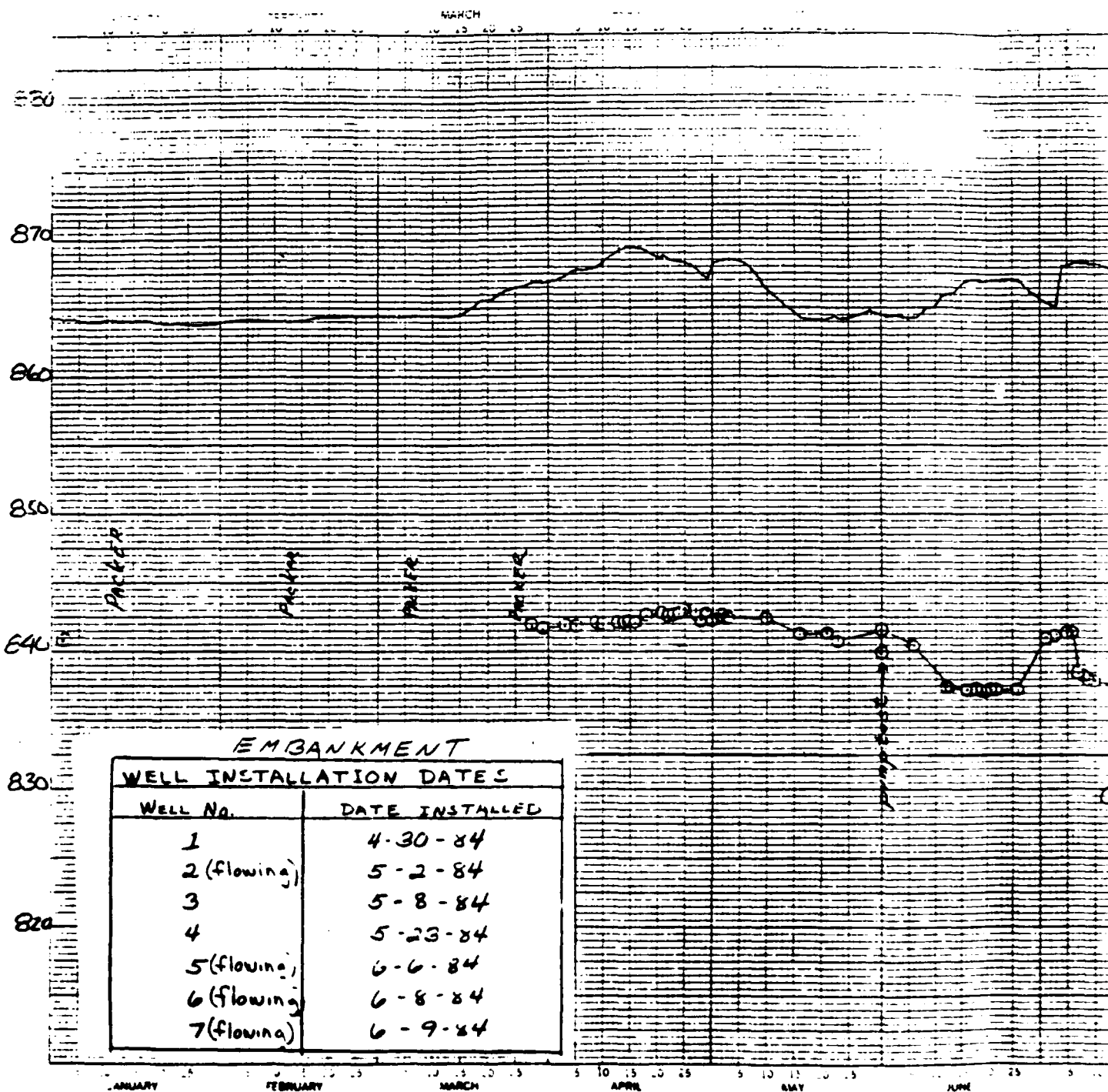
STA 110+00 TIP EL 808.1
 RNC 348 D GRND EL 868.1
 MAT'L SDY CL + GRV RP-3-1743

APPENDIX B PLATE NO. 23

2

47 2813

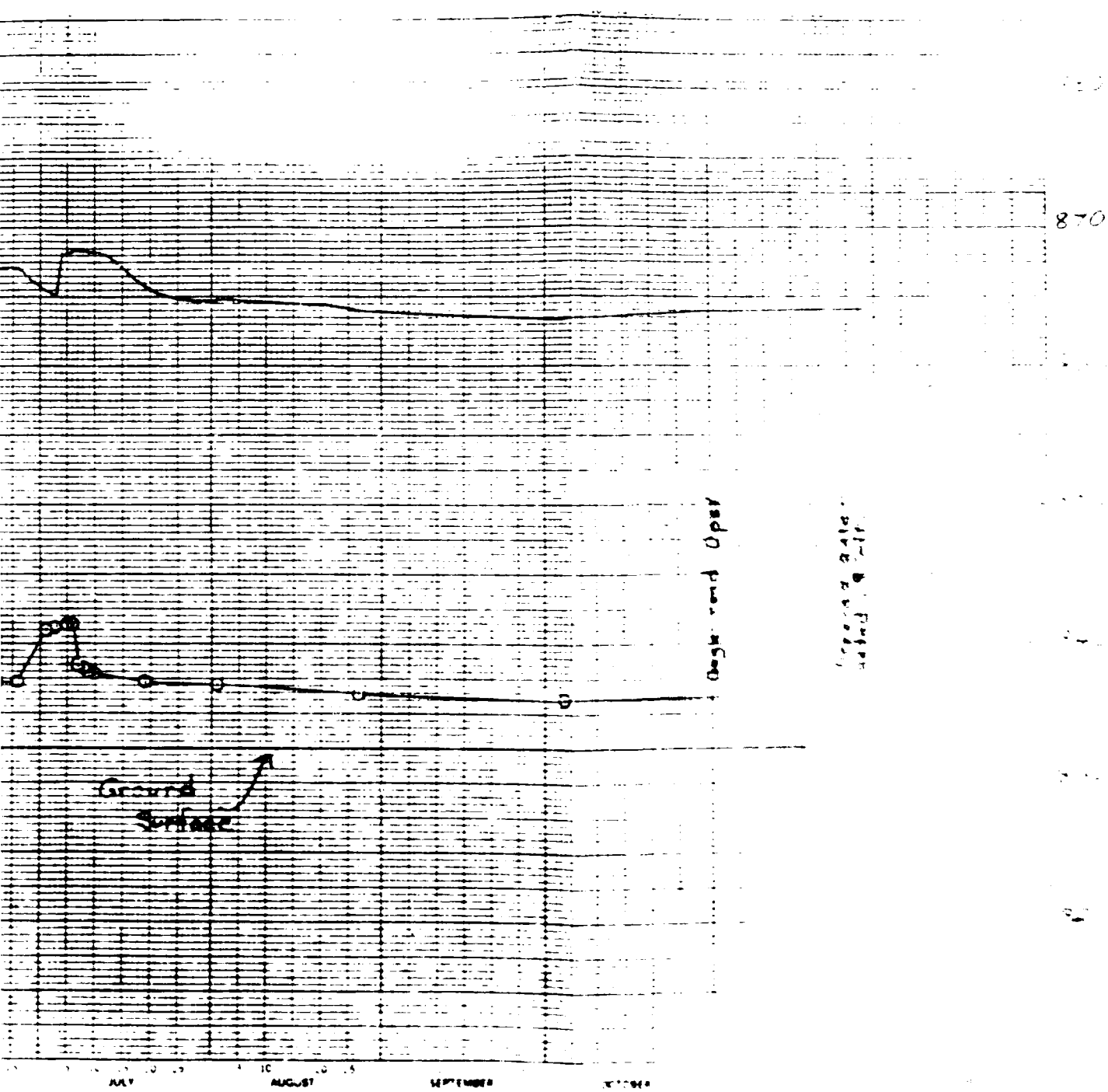
16-2 1 YEAR BY LOTS & 100 DIVISIONS
KUSSEL & SONS CO. NEW YORK



1954

15-1-7

1



1944

7-2
8-1
11-1

47 2813

880

870

860

850

840

EMISANKMENT

WELL INSTALLATION DATES

WELL No.	DATE INSTALLED
1	4-30-84
2 (flowing)	5-2-84
3	5-8-84
4	5-23-84
5 (flowing)	6-6-84
6 (flowing)	6-8-84
7 (flowing)	6-9-84

SUGAR WELLS

PUMP LOSS

WELL PLUGGED

WELL PLUGGED

FEBRUARY

MARCH

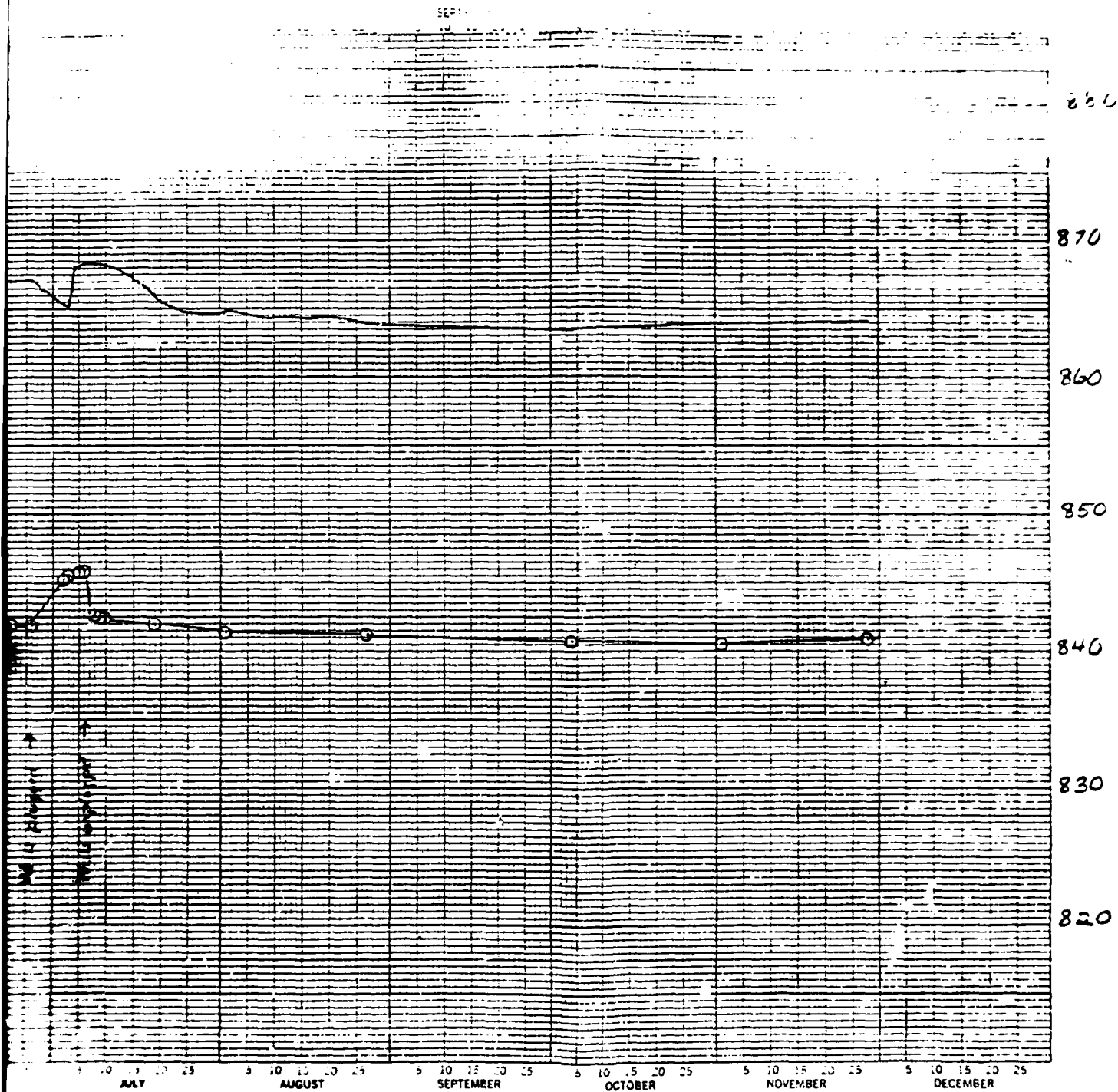
APRIL

MAY

JUNE

1984

15-1-9



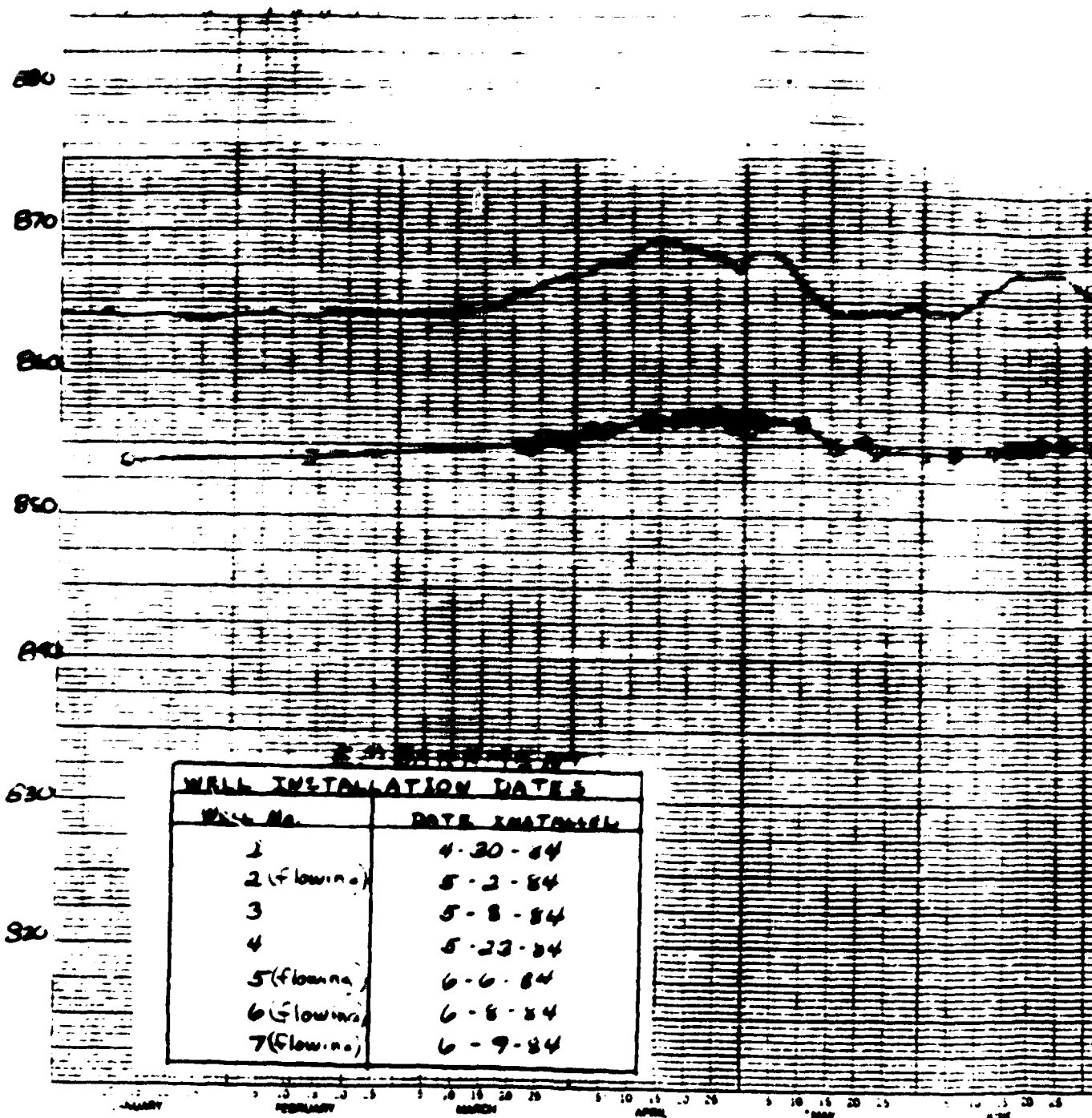
STA 110+00 TIE EL 811.0
 RP-3-1745 RING 109 D GRND EL 863.0
 APPENDIX B PLATE NO. 25 MAT'L CLY SDY GRV

P-110-8

2

47 2813

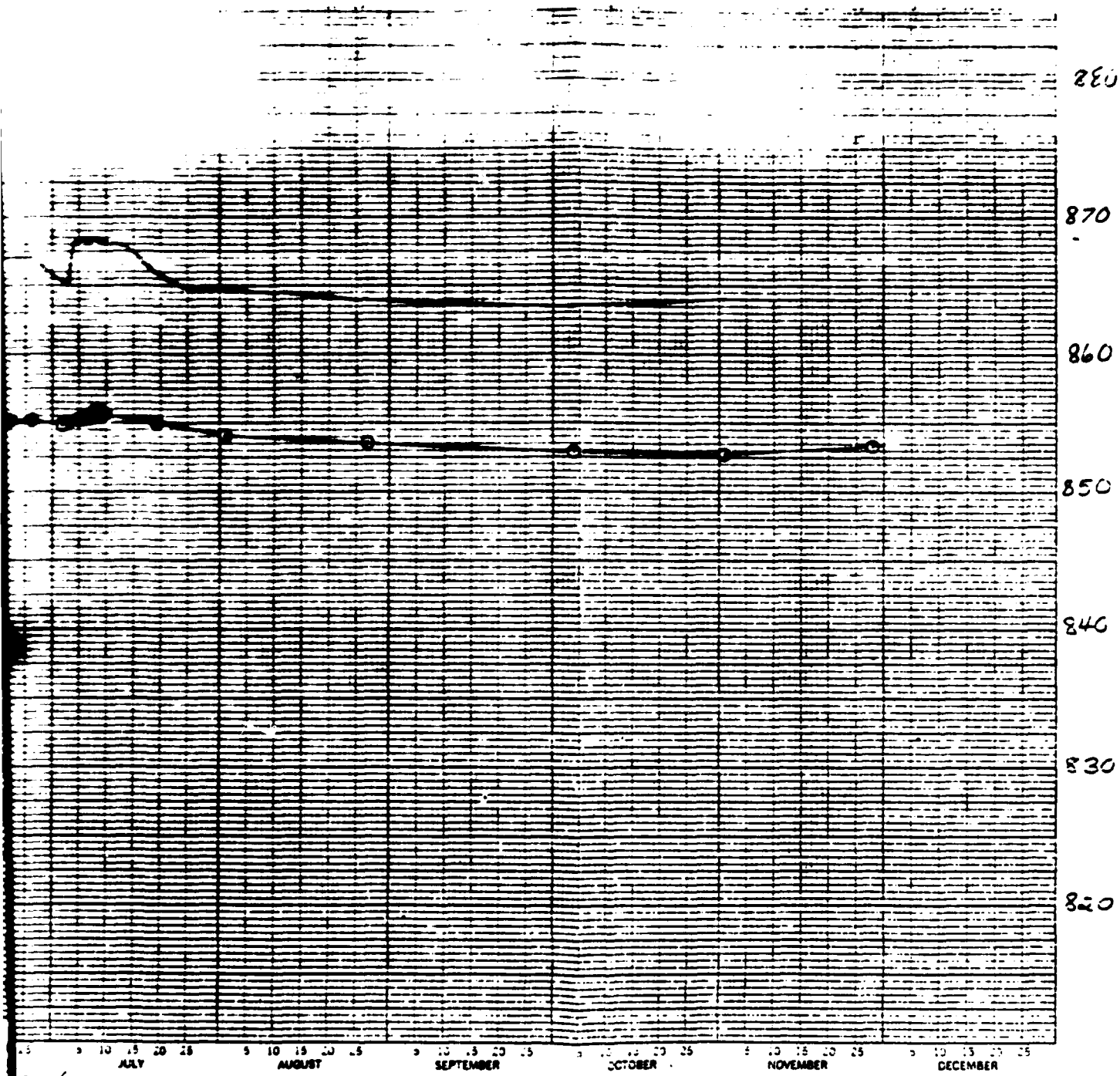
ICE CUBES IN CUP & IN DRINKS
ICE CUBES IN CUP & IN DRINKS



WELL INSTALLATION DATES	
WELL No.	DATE INSTALLED
1	4-30-84
2 (Flowing)	5-2-84
3	5-8-84
4	5-23-84
5 (Flowing)	6-6-84
6 (Flowing)	6-8-84
7 (Flowing)	6-9-84

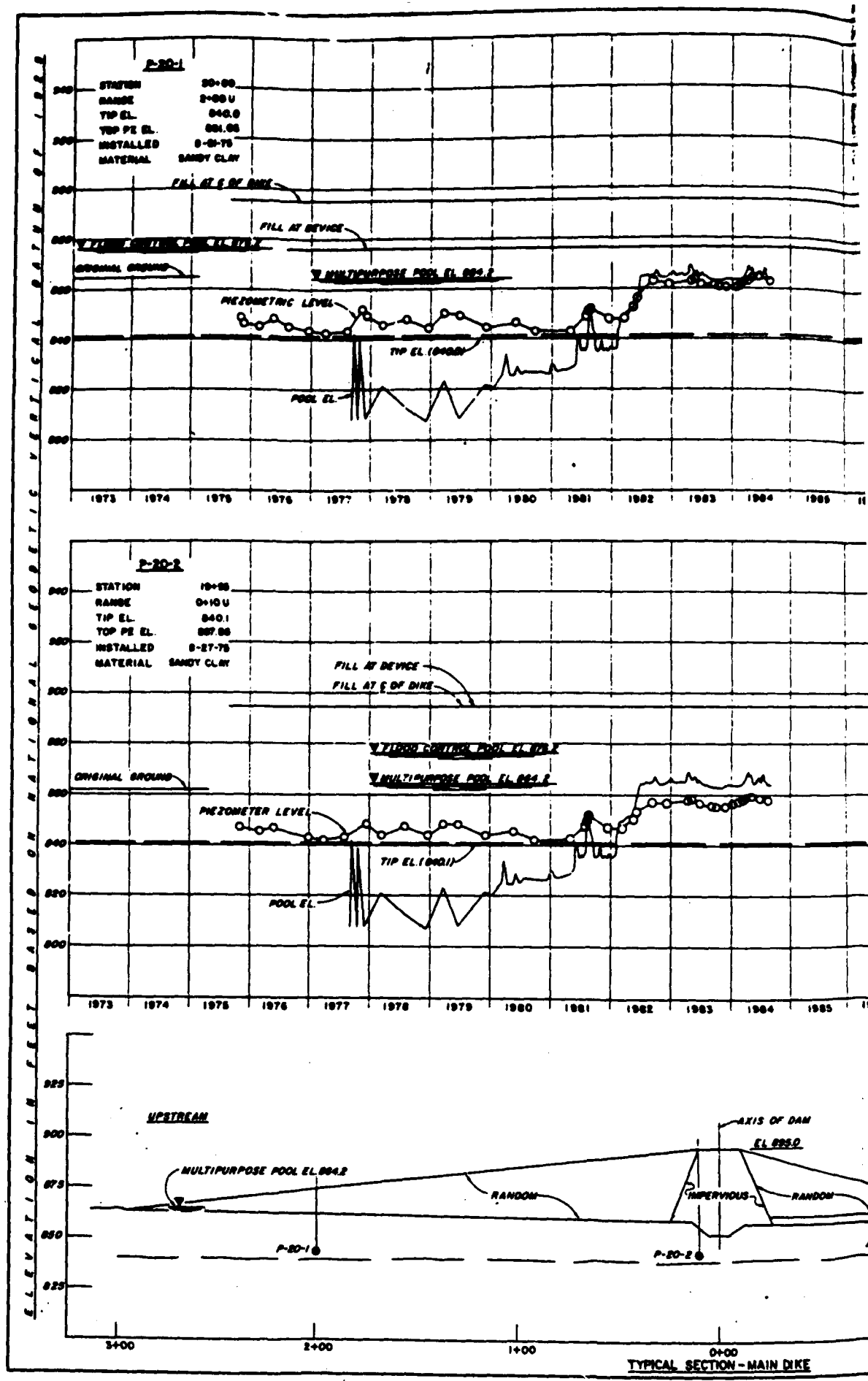
15-1-7

1984



984

P-118-1
 STA 118+00 TPEL 307.0
 RNG 100'D GRND EL 868.5
 MAT'L GRVY CLY SD
 RP-3-1746 2 Plate No. 26



15-1-7

YEARS OF RECORD

YEARS OF RECORD

DOWNSTREAM

LINE OF ROAD

RANDOM

APPROX. TOP OF BEDROCK

REVISED SEPTEMBER 1964
LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE

PIEZOMETRIC LEVELS
P-20-1 (OPEN TUBE)
P-20-2 (OPEN TUBE)

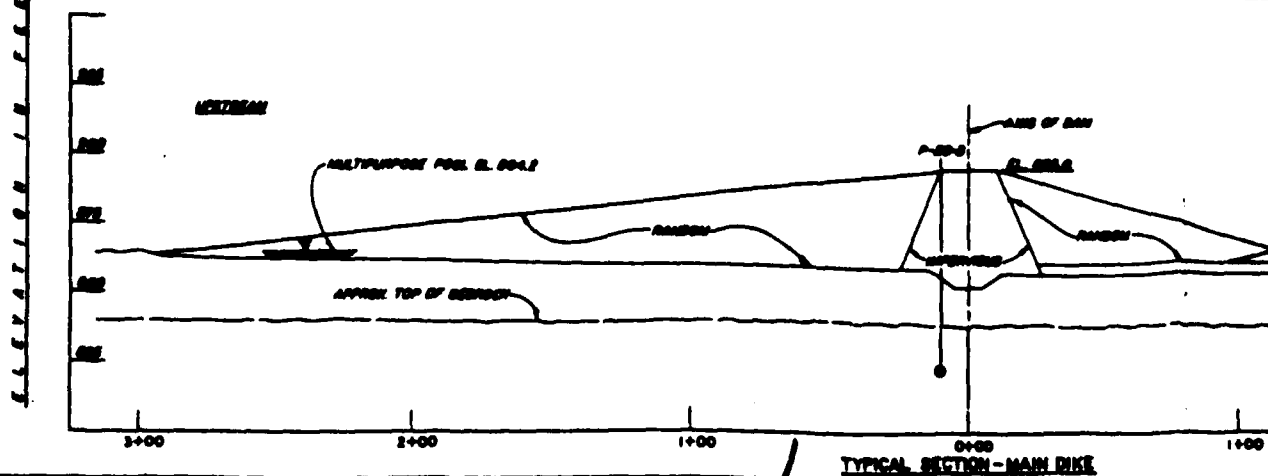
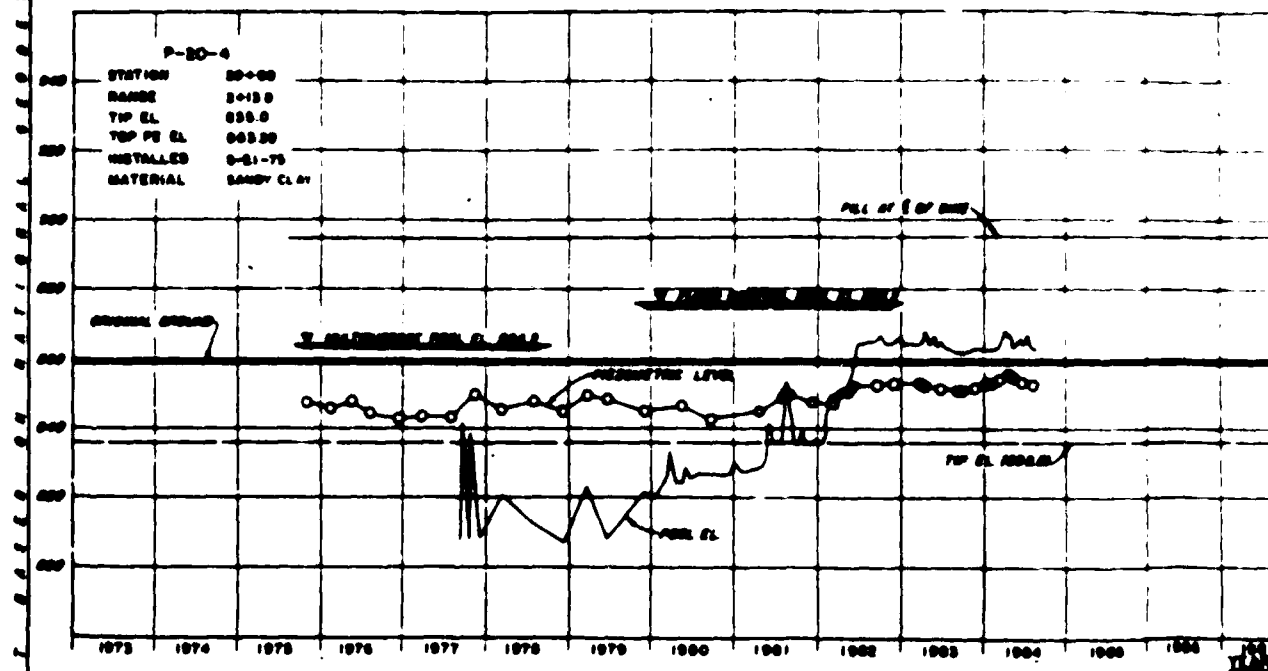
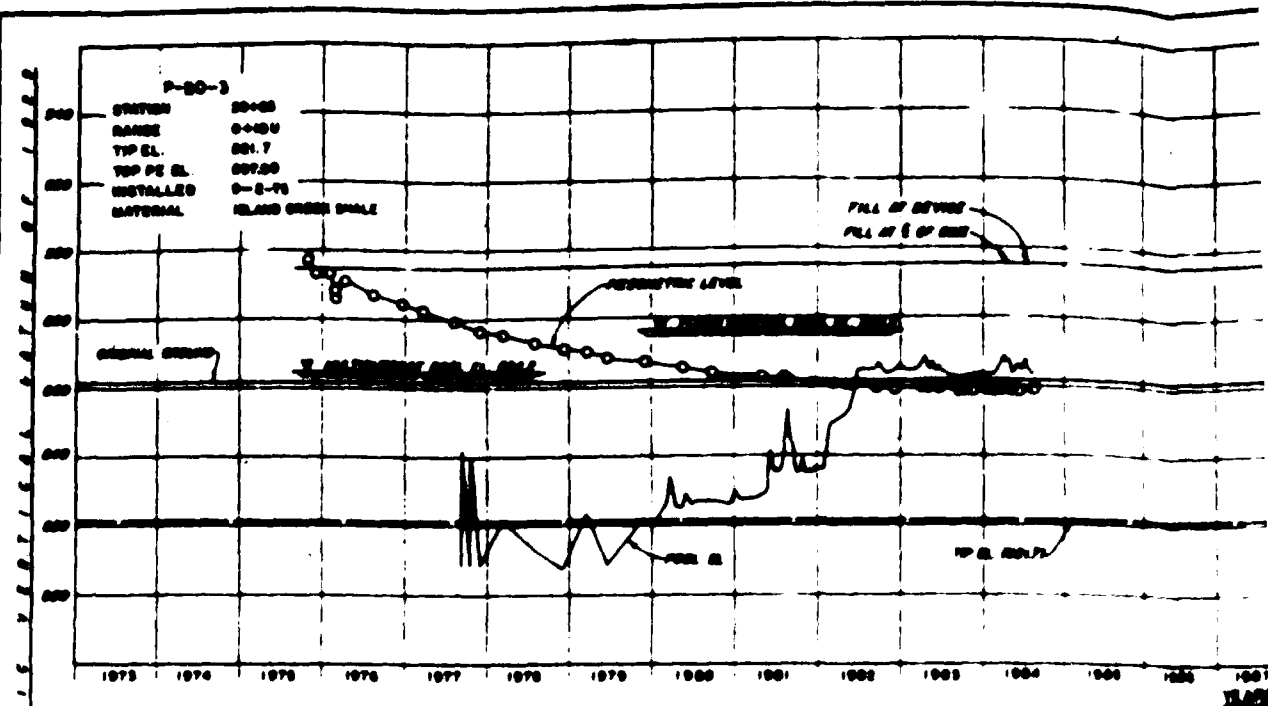
In 1 sheet

Sheet No. 1

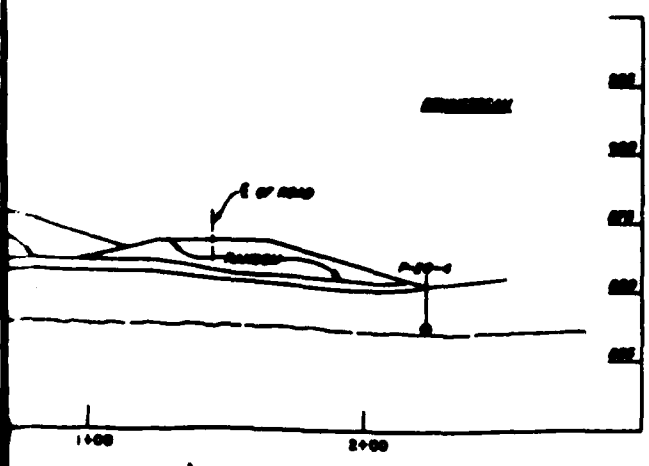
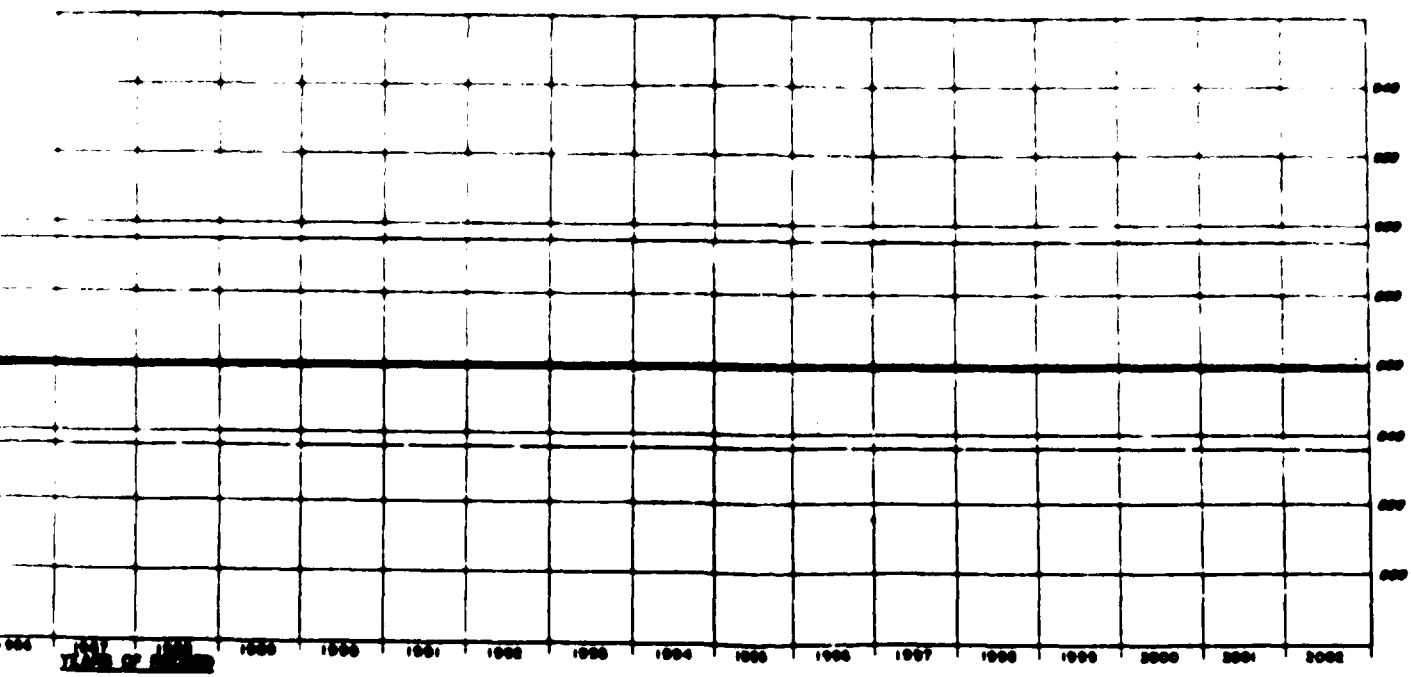
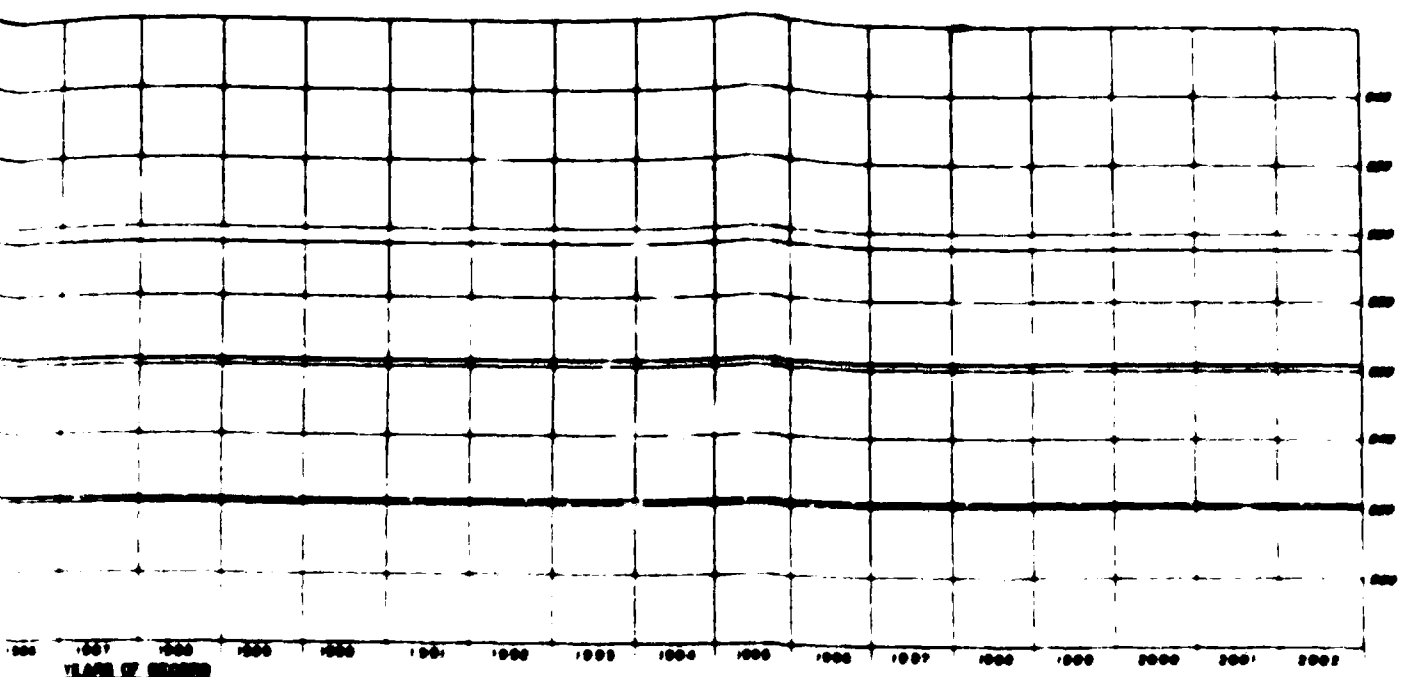
Scale: as shown

CORPS OF ENGINEERS U. S. ARMY
KANSAS CITY DISTRICT
FILE NO. ~~RP-3-1747~~ RP-3-1747
MARCH 1961

APPENDIX B PLATE NO. 27



15-1-7



REVISED SEPTEMBER 1961
 LITTLE PLATTE RIVER, MISSOURI
SMITHVILLE LAKE

PIEZOMETRIC LEVELS
 P-20-3 (OPEN TUBE)
 P-20-4 (OPEN TUBE)

In 1 sheet

Sheet No. 1

Scale as shown

CORPS OF ENGINEERS U. S. ARMY
 KANSAS CITY DISTRICT

FILE NO. RP-3-1748
 MARCH 1961

APPENDIX B PLATE NO. 28