# INTERIM FINAL DRAFT REPORT
# ON
# EMBANKMENT FAILURE
# FLORIDA POWER & LIGHT COMPANY
# MARTIN PLANT COOLING RESERVOIR

April 1980

South Florida Water Management District
West Palm Beach, Florida

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# PREFACE

On October 30, 1979, a part of the embankment at the Florida Power & Light Company, Martin Plant failed catastrophically. This event triggered a series of intensive investigations whose primary objective was to determine the cause of the failure, if possible, and to consider those elements of work required to reconstruct the breached area and further examine and then effect repairs to the remainder of the reservoir embankment in order to assure, to the greatest extent possible, that a failure could not reoccur.

The report that follows describes these activities and chronicles in summary fashion, the original exploratory program and the hasis of the original design. This report then describes the failure and the events and activities immediately subsequent to that time and proceeds into the details of the failure investigation and remedial repairs. Finally, conclusions and recommendations are made.

This report is being submitted on an interim basis. The responsibility for the Water Management District is to provide broad public distribution of the results of its investigation and to be responsive to Florida Power & Light Company's public responsibilities makes this approach desirable. The final report designed for broad distribution will be published in the very near future.

# CHAPTER 1

## INTRODUCTION

Without warning about midnight on October 30-31, 1979, the embankment containing Florida Power & Light Company's (FP&L) Cooling Water Reservoir failed catastrophically. This report described the project, its design, the failure event, the various investigations into the causes of the failure, conclusions, developed as a result of these investigations, and recommendations of remedial actions.

### Project Description

The Martin Plant is a major power generation facility of the FP&L system. The cooling water reservoir is an intergral part of that facility, which was scheduled to begin power generation in the summer of 1980. The cooling water reservoir is an offstream facility created by pumping water from Lake Okeechobee via the St. Lucie Canal into a 6600 acre reservoir, which is entirely above ground. Features described in this section are shown in Fig. 1-1.

The reservoir is entirely encircled by an embankment 100,000 feet long. The top of the embankment is at a uniform elevation of 50.0 feet*, and the base of the embankment is at natural ground elevation, which averages about 20 feet over most of its length, except in the extreme northeast corner, where natural ground rises to an elevation of just over 30 feet.

* All elevations hereafter are in feet above sea level (National Geodetic Vertical Datum, NGVD)

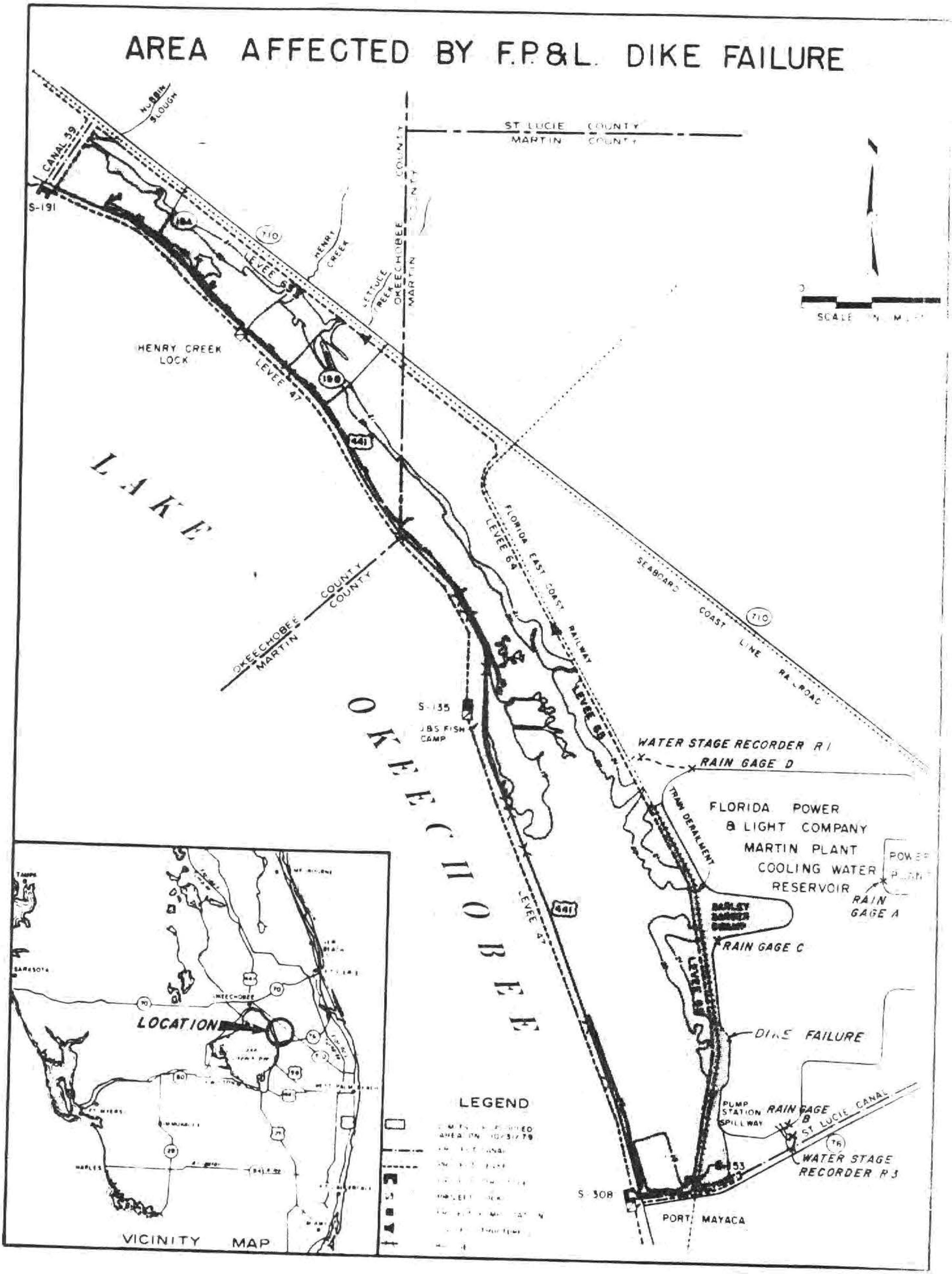

AREA AFFECTED BY F.P.&L. DIKE FAILURE
CANAL 59
S-191
HENRY CREEK LOCK
LEVEE 47
HENRY CREEK
LETTUCE CREEK
OKEECHOBEE COUNTY
MARTIN COUNTY
ST LUCIE COUNTY
MARTIN COUNTY
LAKE
OKEECHOBEE
LEVEE 64
FLORIDA EAST COAST RAILWAY
SEABOARD COAST LINE RAILROAD
S-135
J.B.S FISH CAMP
LEVEE 68
WATER STAGE RECORDER R1
RAIN GAGE D
TRAIN DERAILMENT
FLORIDA POWER & LIGHT COMPANY MARTIN PLANT COOLING WATER RESERVOIR
RAIN GAGE A
RAIN GAGE C
DIKE FAILURE
PUMP STATION SPILLWAY
RAIN GAGE B
ST LUCIE CANAL
WATER STAGE RECORDER R3
S-308
PORT MAYACA
LEGEND
LOCATION
VICINITY MAP

Figure 1.1

Embankment construction was begun in April, 1974 and completed in July, 1977. The reservoir filling was begun on February 6, 1978 by pumping at full capacity 24 hours a day. The minimum operating elevation of 31 was reached on March 7, 1978; the maximum operating elevation of 37 was reached on April 3, 1978, and this level was maintained between 36 and 37.24 until the day of the failure, when pool stage was at 36.74. Since the embankment was founded on pervious material, it was anticipated that considerable seepage would occur, and consequently considerable make-up pumping would be required, except during periods of heavy rainfall. Such indeed was the case and pumping into the reservoir averaged 48 cubic feet per second (cfs).

The embankment was constructed of naturally occurring soil from within the reservoir. It consists of an essentially homogeneous embankment with a 16 foot crest width, an upstream slop of 2 horizontal on 1 vertical (2:1) and a downstream slop of 3 on 1 (3:1). The height varies from 20 to 30 feet and the base width from 110-170 feet. The upstream slope is protected from wave action by a blanket of soil cement, with a horizontal thickness of about 6 feet.

Since the embankment failure occurred on the west side of the reservoir, it is appropriate to discuss other facilities adjacent to and on that side of the FP&L Reservoir.

The Florida East Coast Railroad was constructed in about 1920 and on an alignment parallel to and roughly 500 feet west of what became the west side of the reservoir. The railroad embankment was constructed to an elevation of about 24 feet from material excavated from borrow pits which were located between the railroad and the reservoir.

The District's L-65 Canal was constructed between May 1968 and July 1970. The centerline of this canal lies about 200 feet west of and parallel to the railroad. It has an invert elevation which increases in steps from 3 feet opposite the south end of the FP&L embankment to 7.6 feet at the north end. The canal is flanked by levees with crest elevations of 26.5 feet and 22-25 feet on the west and east side respectively. Seventeen (17) culverts through the east levee act as inlet controls into the canal and control the tailwater stage of the FP&L embankment between elevations 18.5 and 19.0 feet.

The stage in the L-65 Canal is controlled by the automatic gates in structure S-153, which discharges into the St. Lucie Canal. The automatic operation is such that the canal stage at the structure normally varies from 18.4 to 19.2 feet. On occasion, however, as a result of vandalism, prolonged droughts, or exceptionally large storms, the stage can depart considerably from this level.

The St. Lucie Canal roughly parallels the south boundary of the reservoir. This canal is connected to the reservoir by the intake-spillway channel. Consequently, spillway tailwater stages are the same as those in the St. Lucie Canal. The stages in this canal are controlled by the U.S. Corps of Engineers at the S-80 structure, which is located about 20 miles east of the reservoir. This structure is currently operated to control the elevation in the St. Lucie Canal between 14.0 and 14.5. During droughts, however, the canal elevation can, and has dropped to 10, and during a major flood can rise as high as 24.5.

## The Failure Event

The dike failure occurred very suddenly and without warning at 11:30 P.M. on October 30, 1979 at a point about two miles north of the St. Lucie Canal on the west side of the reservoir, as shown on Fig. 1-1 .

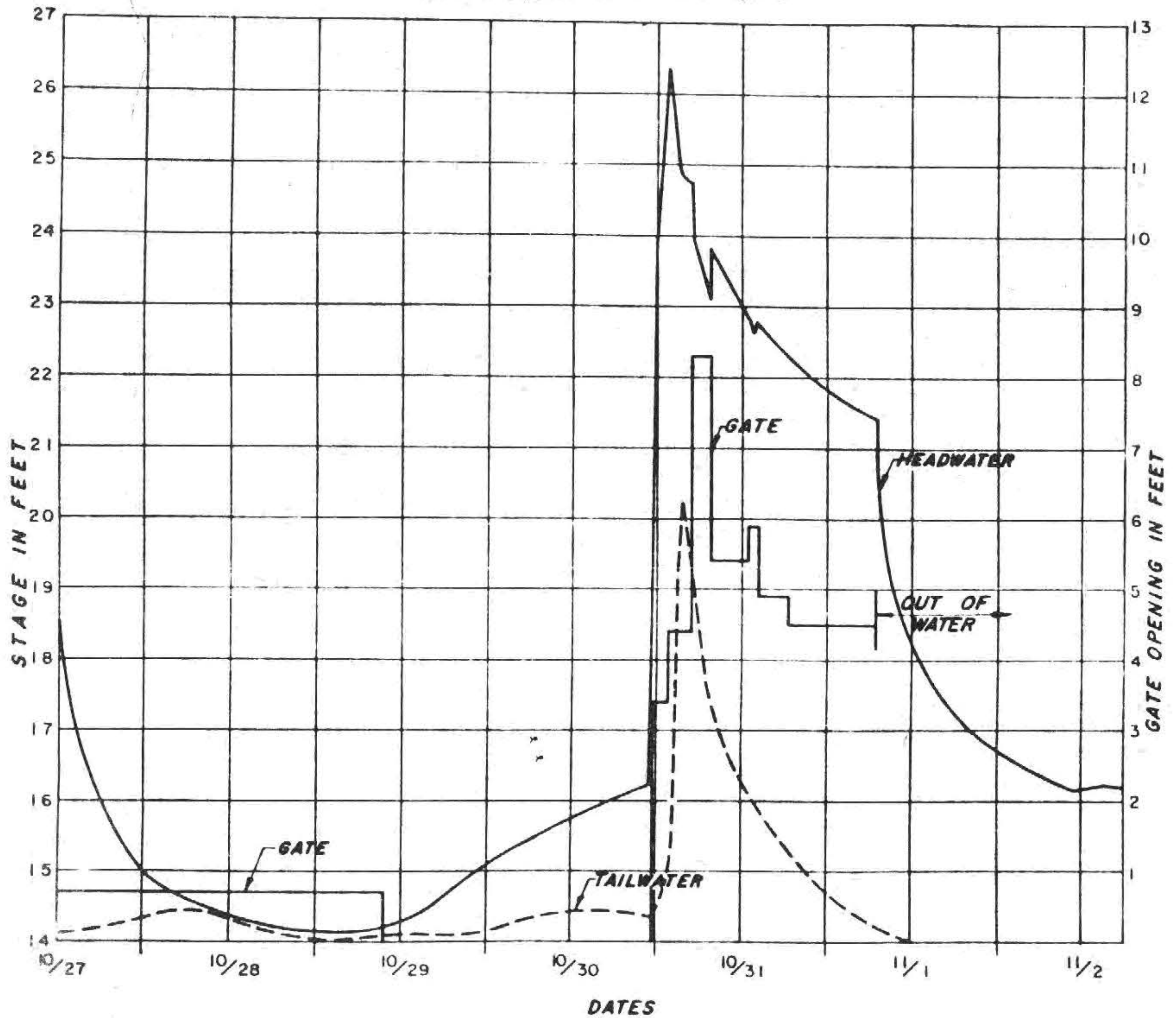
An act of vandalism, though apparently unrelated to the failure, was committed to the District's control structure (S-153) at the south end of the L-65 Canal. Because of the proximity to the failure, both in point of time and geography, this action is described here in some detail.

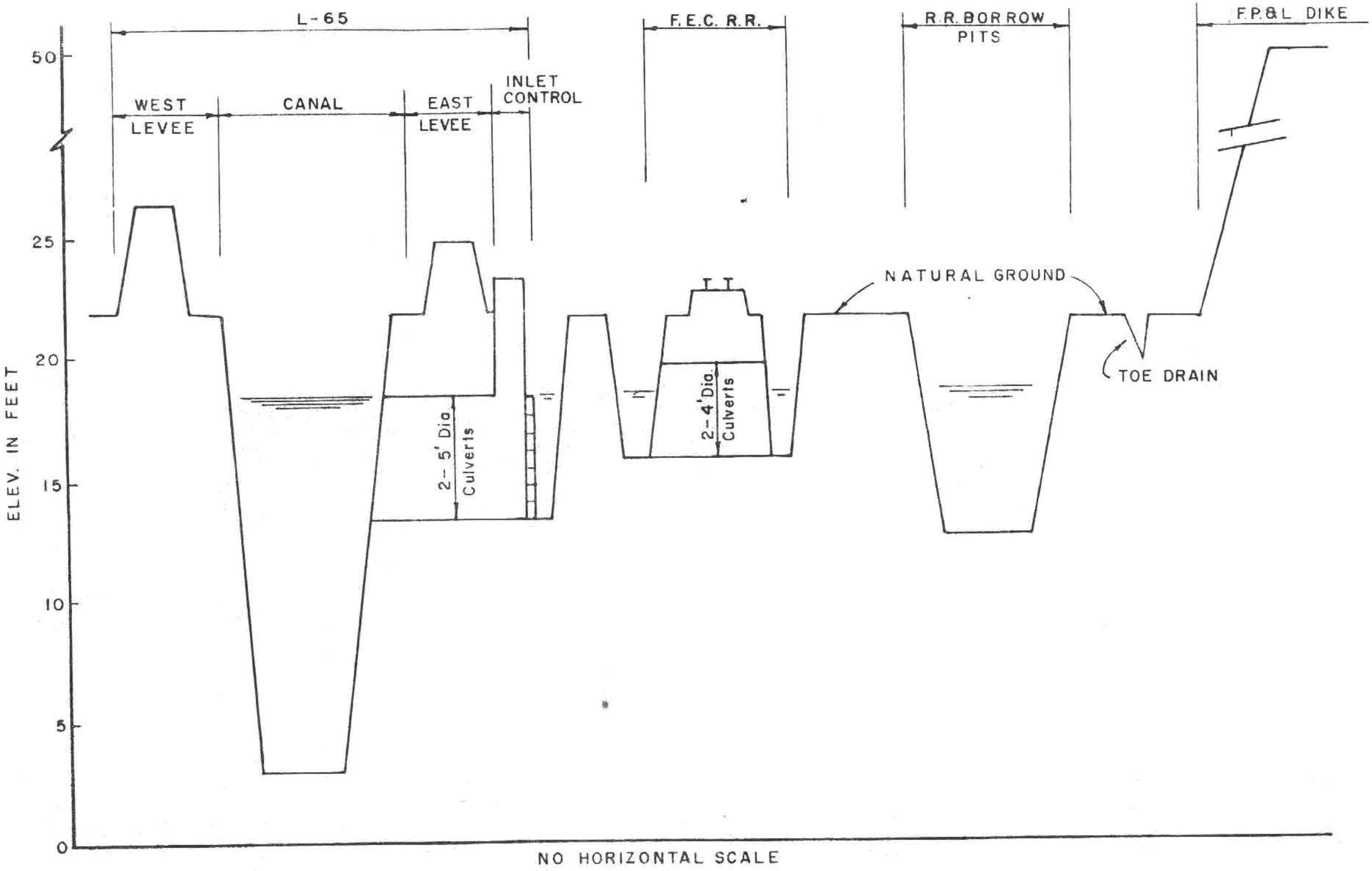
Structure S-153 normally operates in the automatic mode such that when the canal headwater stage rises to elevation 19.2 feet, the two gates begin to open. When the stage falls or rises to elevation 18.85 the gates stop moving, and when the stage falls to 18.37 the gates begin to close. At about noon on Saturday, October 27, the stage reached elevation 19.2 and the gates began to open, the stage then dropped to 18.85, when the gates had opened 0.7 feet. The stage continued to fall, but before it had fallen to 18.37, at about 2:15 P.M., an unknown person turned off the power at the electrical control panel on the outside of the structure. The #1 and #2 gates were found open 0.75 feet and 1.0 feet respectively, by the District water reader who routinely visited the structure about 9:30 A.M., Monday, October 29. At that time the water reader closed the gates, and left the structure on the automatic mode. While the gates were open, however, the L-65 canal stage dropped to the elevation of the St. Lucie Canal, about 14.1. After the gate closure, the L-65 stage recovered to the elevation of 16.1. See Fig. # 1-2 for a graphic representation of headwater and tailwater elevations along with gate openings at S-153 during the period October 27 to November 2.

Although the canal stage was low during this 3 day period, the water stage east of the canal was controlled at or above elevation 18.5 by means of the east levee and inlet controls previously described. Fig. 1-3 shows this relationship. While these facilities were established for

# S-153 OPERATION

OCTOBER 27, 1979 TO NOVEMBER 2, 1979






DIAGRAMATIC PROFILE IN BREACH AREA

Figure 1-3

erosion protection and to guard against scour problems through openings in the railroad grade, when a low water condition exists in the canal, they also serve the dual purpose of preventing unusual drawdowns in the reach contiguous to the reservoir embankment.

As stated previously the initial breach of the reservoir embankment occurred at 11:30 P.M. on October 30. The first indication of the problem was noted by a south bound train on the FEC Railroad adjacent to the embankment. A north-bound freight train had passed the breach area with no problem at 10:00 P.M. on October 30 before the failure occurred. Upon returning southbound, the train in the vicinity of the Barley Barber Swamp became caught in what the crew reported as a "flash flood" and derailed at about 1:00 A.M. on October 31. Unknown to anyone at the time, the "flash flood" reported by the train crew was part of the massive flood wave beginning to leave the FP&L reservoir 1 3/4 miles to the south .

An outflow hydrograph (shown as Fig. 1-4 ) was constructed from the three water stage recorders in the reservoir. This hydrograph shows that the large discharge through the breach occurred almost instantaneously, but continued to increase, probably as the sides of the breach eroded and enlarged, for about four hours. The peak rate of flow of about 100,000 cfs occurred at 3:30 A.M. By this time, over 30,000 acre-feet had been discharged from the reservoir. The recession part of the hydrograph was also fairly steep. By 3:00 P.M. on October 31, the discharge had fallen to less than 15,000 cfs, and about 66,000 acre-feet had been discharged.

As the flood wave escaped through the breach, it removed about 100,000 c.y. of the naturally occurring fine grained sand from beneath and west of the breach. The resulting scour hole was roughly rectangular in plan, about

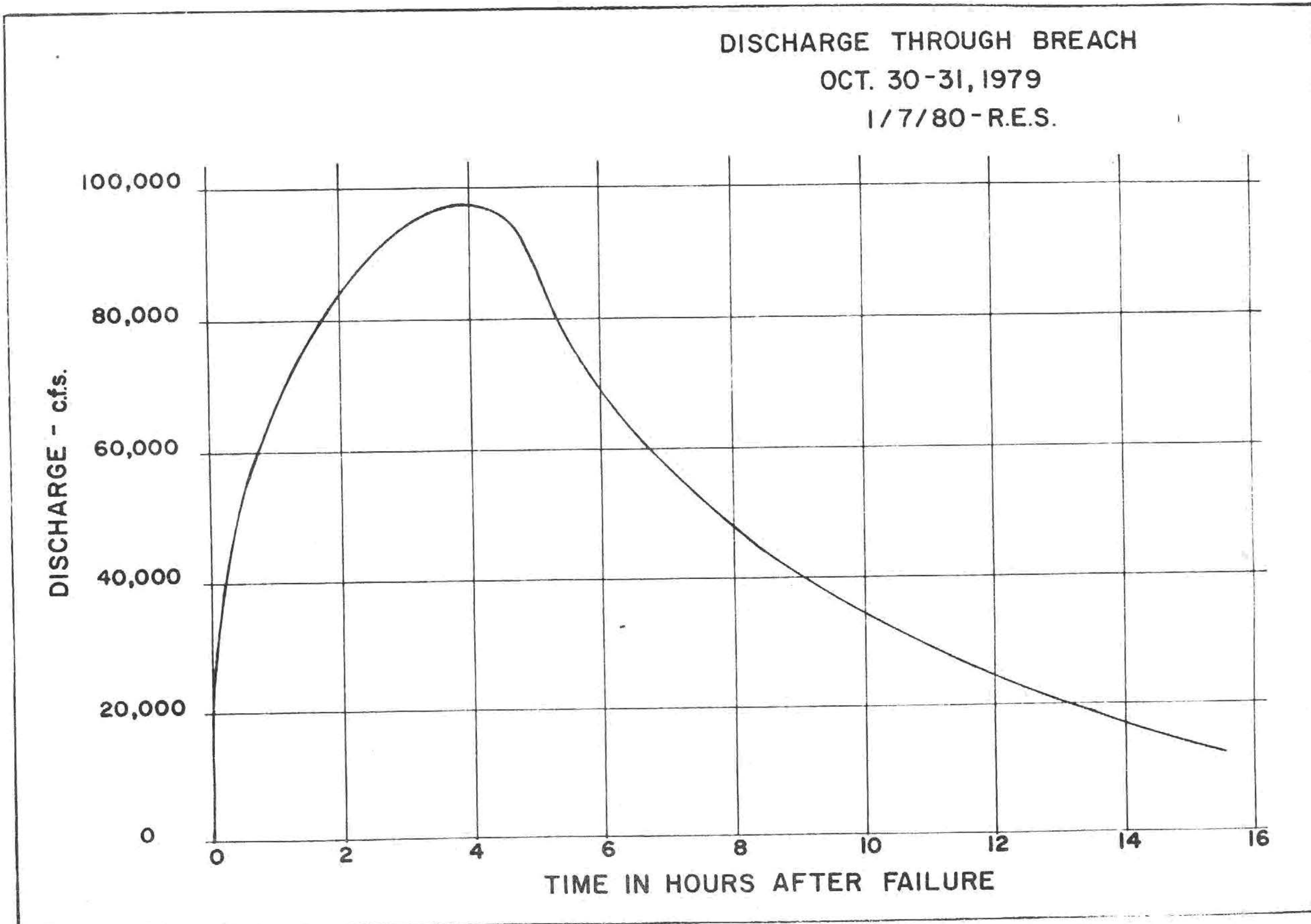
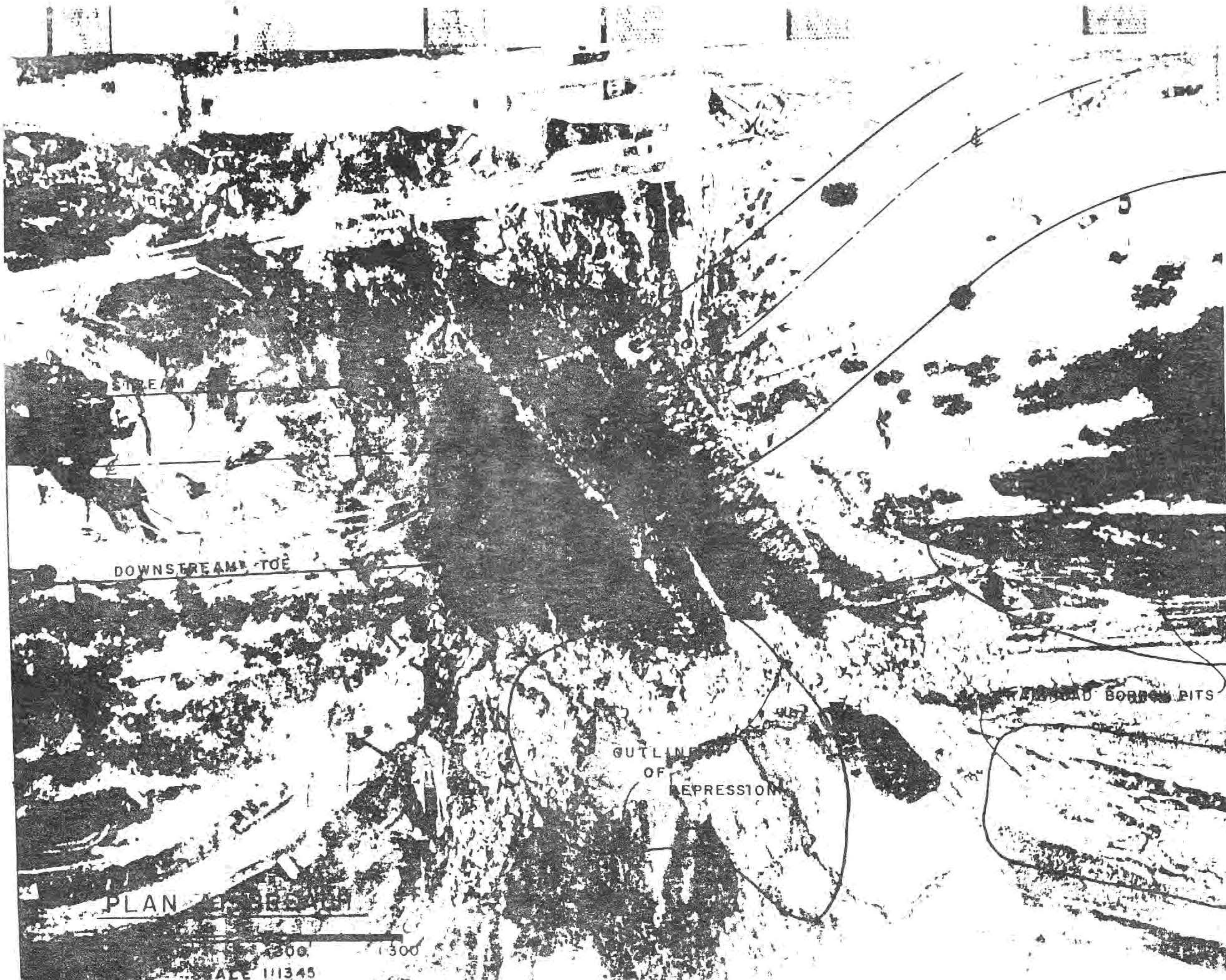

DISCHARGE THROUGH BREACH
OCT. 30-31, 1979
1/7/80-R.E.S.
DISCHARGE - c.f.s.
100,000
80,000
60,000
40,000
20,000
0
0
2
4
6
8
10
12
14
16
TIME IN HOURS AFTER FAILURE

Figure 1.4

450 feet wide (parallel to the dike) by about 700 feet long (perpendicular to the dike). See Fig. 1-5 . The deepest part of the scour hole was located in the center and west of the breach. This deepest hole was also rectangular in plan, 190 feet wide by 400 feet long. The invert of this hole was at elevation -7 feet at the east end; it sloped upward to an elevation of about +4 feet at the west end. Surrounding this deep scour hole a dark brown silty sand stratum was uncovered at about elevation 0 to +4. All material above this stratum was eroded away in the scour area. A vertical section along the embankment center line through the breach was U-Shaped, about 600 feet across the top, with vertical faces from natural ground to the embankment crest.

The soil cement slope protection collapsed in place, ending in a heap along the embankment alignment except in the very center of the breach. See Fig. 1-5 . In this location a gap was left in the remnents of soil cement, about 10-15 feet wide. The soil cement as it fell into its final resting place broke into large random sized slabs on an average of 6 feet x 6 feet. Each slab was about 6 inches thick, the original thickness. On each side of the center gap in the soil cement, the remaining pieces were moved about 40 feet to the west. A few large slabs about 5 feet square were deposited roughly 300 feet below the embankment and numerous smaller pieces (1± feet square) were carried as far downstream as the railroad.

The flood wave over-topped the railroad, washed it out and spilled over and through the L-65 east levee, into the canal sending a flood wave to the north and south along the canal and westward over and through L-65 west levee toward Lake Okeechobee. The southbound wave caused the S-153 structure



STREAM
DOWNSTREAM TOE
OUTLINE
OF
DEPRESSION
BORROW PITS
PLAN
300
300


Figure I-5

to open automatically and begin discharging into the St. Lucie Canal, however, the flood wave surge far exceeded the structure's capacity with the gates only opening at a rate of 6 inches per minute. Consequently the headwater continued to rise, reaching a peak elevation of 26.13 at 2:00 A.M. on October 31. (Peak discharge at S-153 was approximately 3700 cfs).

The high stages produced by the flood wave caused the flood waters to by-pass S-153 to the east and over-top the St. Lucie Canal North Tieback Levee. It is estimated that about 15,000 acre-feet flowed into the St. Lucie Canal through and around S-153 on October 31.

The stage in the St. Lucie Canal rose rapidly once S-153 opened, and this Canal peaked at elevation 20.27 at about 4:00 A.M. at Port Mayaca. The Corps of Engineers began opening S-80 at about 2:30 A.M. and this structure peaked at an elevation of 15.8 at about 4:30 A.M. when the discharge was estimated to be in excess of 15,000 cfs. The Corps also opened both the spillway and the lock at S-308 allowing about 4,000 cfs to enter the Lake between 4:00 A.M. and 9:30 A.M. when the St. Lucie stage again dropped below the Lake stage which was at elevation 17.4.

The flood wave spread over the sugar cane fields to the west before over-topping U.S. Highway 441 and entering the Rim Canal east of the Herbert Hoover Dike. The surge then traveled northward in the Rim Canal reaching the District's Pump Station S-135 (7 miles north of the St. Lucie Canal) between 4:00 A.M. and 5:00 A.M. The pump station crew had already been alerted and were pumping S-135 at full capacity into Lake Okeechobee by 3:30 A.M. The stage at S-135 continued to rise until a peak elevation of 21.15 was reach by noon on October 31. The flood wave continued northward through the more populated areas which had been evacuated earlier by the

Martin County Sheriff's Department. The Rim Canal reached a peak the next day (November1) at the north end of the basin, 17 miles from the St. Lucie Canal. The flood was contained at this northerly point by the Nubbin Slough Tieback Levee along Canal 59. The maximum area flooded, was about 14,100 acres.

Authority

The Water Resources Act of 1972 (Chapter 373 FS) as amended gives to the Water Management District broad powers to manage and regulate the water resources within the District. Essential parts of the District's responsibilities are contained in Part 4 of the act which addresses management and Storage Of Surface Waters. Further, on June 8, 1973, an agreement was entered into by the District and FP&L which relates to the taking of water from District regulated facilities for purposes of filling the reservoir, special conditions pertaining thereto and other related matters. Acting under the provisions contained in these documents, in addition to water use permits issued in consequence of the above, the District, in response to the reservoir failure convened a special investigative committee.

Scope

The essential charge to the committee given by the Water Management District was to conduct an impartial, independent investigation to determine if possible, the cause of the failure, how the breached area will be reconstructed, determine what safeguards are necessary to prevent failure to any part of the remainder of the reservoir and finally what safeguards will be implemented to monitor adequately future operations.

Investigation Committee

The Committee was organized on November 13, 1979 and consisted of:

Richard Slyfield

Ronald F. York

Abe Kreitman, Chairman

To provide additional expertise, the Committee retained the services of Mssrs. William Clevenger of Woodward-Clyde Consultants and Harry Cedergren, Consulting Engineer.

The report that follow details the results of this investigation.

# CHAPTER 2

# BASIS OF ORIGINAL DESIGN

## Foundation and Materials Investigation

Pre-design investigations of foundation and materials were conducted by National Soil Services, Inc. The details incorporating a series of recommendations concerned with basic design of the embankment and appurtenant structures were submitted in report from to Mid-Valley, Inc. the engineers for the project. The details of these data can be reviewed in the following reports:

Volume 1 - Soil Investigation, Project Seminole. April 1972

Volume 2 - Field & Laboratory Data, November 1973

Volume 2A- Field & Laboratory Data, November 1973

Volume 2B - Field & Laboratory Data, December 1973

Volume 2C - Field & Laboratory Data, February 1974

Volume 3 - Design Analyses and Recommendations
Embankment & Appurtenant Structure, April 1974

The project design was reviewed by a Board of Consultants consisting of Messrs. V. Conn, R. Linsley and J. Sherand.

## Results of Exploratory Program

The basic data and associated analysis and interpretation of results presented in these reports are comprehensive and detailed. In summary form these volumes contain:

A series of seventeen borings were made that described the general lithology and established the strategraphic sequence of the rock formations that could be anticipated throughout the site. The specific objectives in developing this program were as follows:

a) Obtain standard penetration measurements and undisturbed cores in order to develop an initial assessment of bearing and an in-situ, - relatively undisturbed sample of the various soil layers.

b) Provide selective samples for the determination of relative density and correlation of penetration resistance with the density

c) Investigate with a series of borings the lithology and other typical characteristics of the numerous closed, shallow sink hole type depressions that existed throughout the site. Obviously it was of fundamental importance to recognize at the earliest possible time any significant network of solution riddled rock or other subsurface condition associated with the depressions that could endanger the proposed reservoir to perform as intended.

d) Initially it was contemplated to run geophysical logs in each of the exploratory holes. The purpose of these surveys was to assist in defining the various lithologic units and to provide a means more accurately to correlate the areal and vertical extent of these layers throughout the reservoir site. These surveys were discontinued when it was demonstrated that no such benefits could be derived.

e) Perform an initial pump test to develop a preliminary understanding of the permeability of two basic units - the shallow sediments to a depth of about 25 feet below grade and a permeable shell rock that was found to exist below a somewhat impermeable limestone. From an analysis of a series of pump tests a set of relationships was developed that included both the horizontal and vertical components of permeability, and the associated water levels and potentiometric gradients across these two units. These data also allowed for an initial assessment of seepage and flow through the system.

Many of the results of the field classifications as described above were verified by means of laboratory testing. Thus, classification and uniformity of soils and their characteristics (specifically permeability and relative density) were determined. In addition, moisture content and liquid and plastic limits were determined. These data served as input to basic reservoir design considerations.

On the basis of these determinations a series of recommendations concerning reservoir design were made relative to:

a) Embankment soils

b) Estimates of anicipated seepage losses

c) Foundation conditions

Volume II and associated supplementary volumes (a,b,c) are essentially basic data reports. These volumes contain the bulk of the field and laboratory data that formed the final basis of design for the reservoir and associated structures and appurtenances.

Subsurface lithologic features are defined primarily by:

a) A series of approximately 125 standard penetration borings.

b) A series of shallow test pits designed to locate, define and ultimately provide recommendations for the development of proposed borrow pits and to investigate the occurrence and effect of strata of cemented shell and of ground water conditions in the vicinity of the borrow areas.

c) An extensive series of pump tests designed to determine the vertical and horizontal components of permeability, and the degree of hydraulic communication across the various beds and between the formations lying above and below a hard, relatively impermeable limestone layer.

d) A special program of exploration designed to investigate the large number of shallow, closed depressions that are scattered throughout the reservoir site. This program consisted of an extensive series of probes and borings across numerous such depressions in order to determine the nature and extent of the depressions and their relationship to underlying formations. The essential concern in this investigation was the degree of potential hydraulic communication that these physical features had with the hydrogeologic regimen under full reservoir conditions and if these features could compromise the structural integrity of the reservoir.

Laboratory testing and analysis of samples and materials derived from this field work was performed and are presented in these volumes. These data consist of:

a) Classification tests on foundations soils.

b) A selected number of compaction and cement stabilization tests on borrow soils.

c) Chemical and mineralogical analysis of selected samples to evaluate the closed, shallow depressions.

Volume III assimilates all of the data as described above and presents the conclusions derived from these analyses and concludes in a series of recommendations concerned with the design and subsequent construction of the reservoir embankment and appurtenant structures. The major elements of the study and analysis include:

a) A definition of soil parameters and the treatment to be followed for the design and construction of the embankment and appurtenant structures.

b) Final development of lithologic and stratigraphic relationships throughout the site.

c) An assessment and quantification of surface seepage and underseepage conditions that may be anticipated under reservoir operating conditions.

Figures 2-1, 2-2, 2-3 taken from FP & L reports show the location of various elements of the subsurface exploratory program; a location plan of the shallow depressions that were examined and the detail of a typical depression. investigation.

## Embankment Design

### Design Criteria

Because of the materials available for construction and the foundation exploration described above, it was apparent that the embankment to be constructed would be a homogeneous sand dam built on a sand foundation. For this situation, the embankment and the foundation design could not be separated. Consequently, this section will describe the soil characteristics and basis of design of both.

It was assumed that the foundation consisted of three strata, each of which was composed of homogeneous material. No tests of the strength of the foundation materials were presented. The strength parameters used in the design appear to be based on an aribitrary reduction from the tested values obtained for the embankment material which had very similar grain size distribution curves. The statement is made in the design report that, "critical foundation soil strengths or strata of minimum shear resistance were not revealed by the borings".

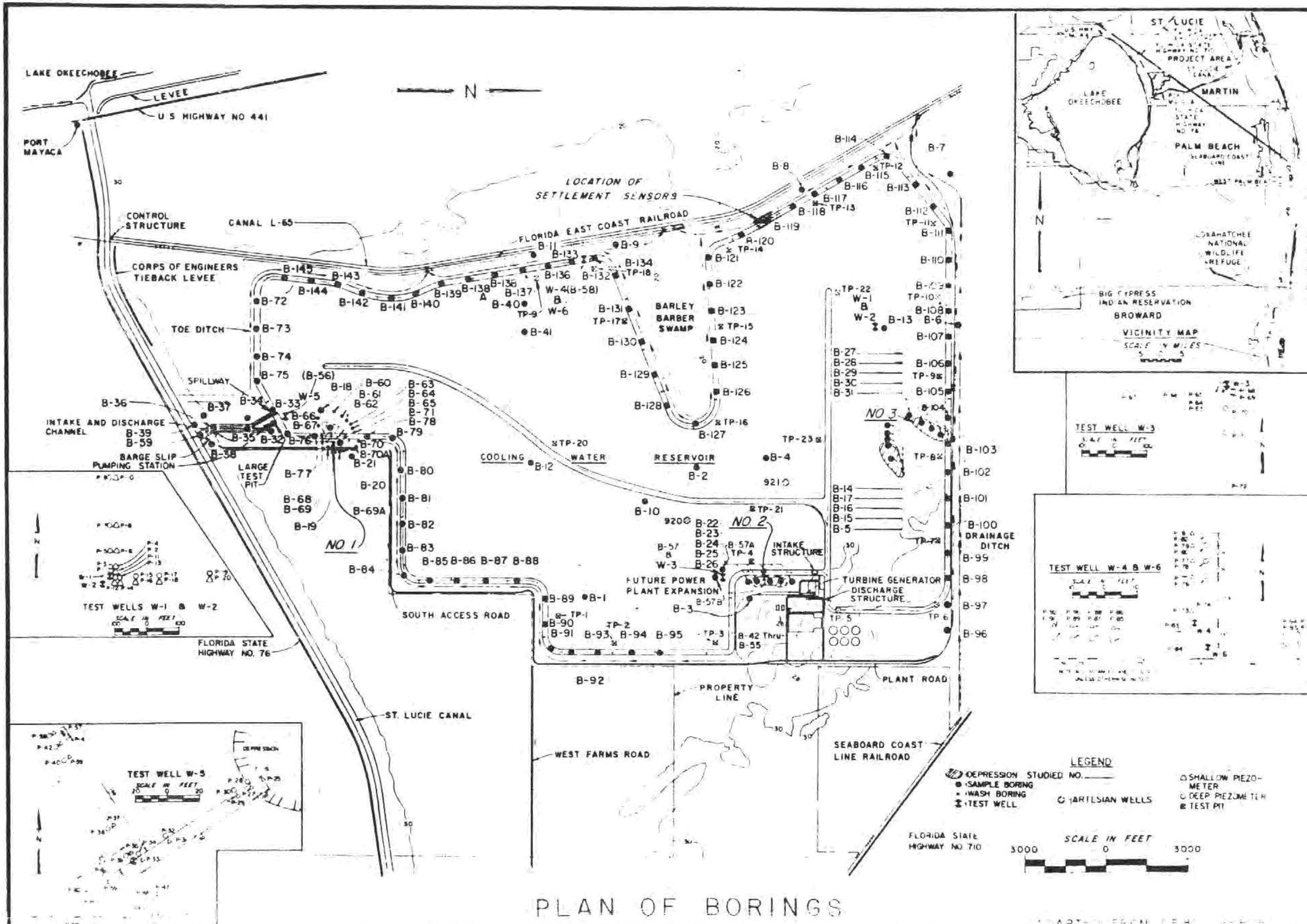
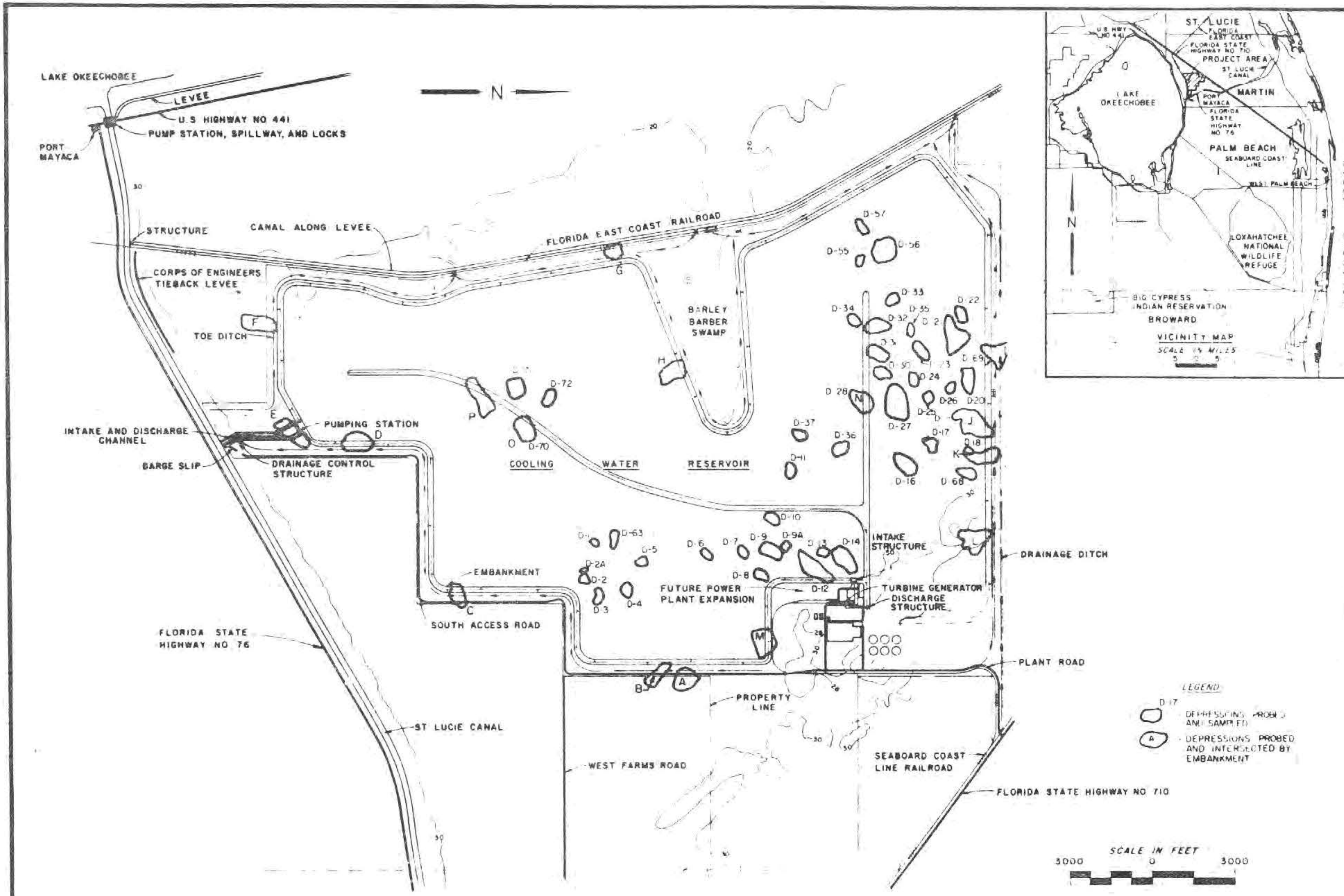
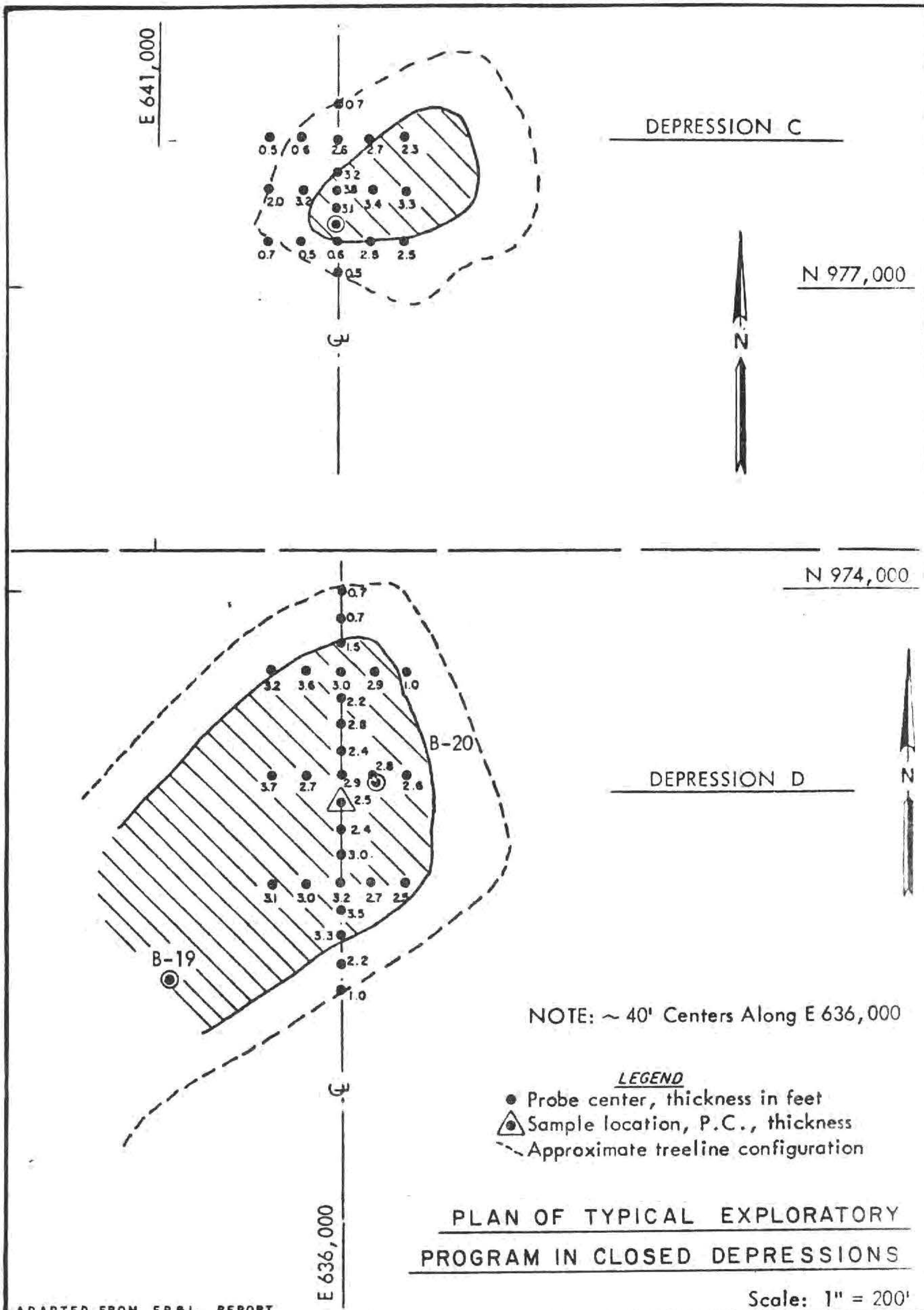

PLAN OF BORINGS
LAKE OKEECHOBEE
LEVEE
U S HIGHWAY NO 441
PORT MAYACA
CONTROL STRUCTURE
CANAL L-65
CORPS OF ENGINEERS TIEBACK LEVEE
TOE DITCH
SPILLWAY
INTAKE AND DISCHARGE CHANNEL
BARGE SLIP
PUMPING STATION
LARGE TEST PIT
LOCATION OF SETTLEMENT SENSORS
FLORIDA EAST COAST RAILROAD
BARLEY BARBER SWAMP
COOLING WATER RESERVOIR
FUTURE POWER PLANT EXPANSION
INTAKE STRUCTURE
TURBINE GENERATOR
DISCHARGE STRUCTURE
DRAINAGE DITCH
SOUTH ACCESS ROAD
PROPERTY LINE
PLANT ROAD
WEST FARMS ROAD
ST. LUCIE CANAL
FLORIDA STATE HIGHWAY NO. 76
SEABOARD COAST LINE RAILROAD
FLORIDA STATE HIGHWAY NO 710
TEST WELLS W-1 & W-2
TEST WELL W-5
TEST WELL W-3
TEST WELL W-4 & W-6
VICINITY MAP
LAKE OKEECHOBEE
ST LUCIE
MARTIN
PALM BEACH
BROWARD
PROJECT AREA
BIG CYPRESS INDIAN RESERVATION
LEGEND
DEPRESSION STUDIED NO.
SAMPLE BORING
WASH BORING
TEST WELL
ARTESIAN WELLS
SHALLOW PIEZOMETER
DEEP PIEZOMETER
TEST PIT
SCALE IN FEET
3000 0 3000

Figure 2-1




DEPRESSIONS PROBED AND SAMPLED

Figure 2-2



E 641,000
DEPRESSION C
0.7
0.5
0.6
2.6
2.7
2.3
3.2
2.0
3.2
3.8
3.4
3.3
3.1
0.7
0.5
0.6
2.8
2.5
0.5
N 977,000
N
N 974,000
0.7
0.7
1.5
3.2
3.6
3.0
2.9
1.0
2.2
2.8
B-20
2.4
2.8
DEPRESSION D
3.7
2.7
2.9
2.6
2.5
2.4
3.0
3.1
3.0
3.2
2.7
2.5
3.5
3.3
B-19
2.2
1.0
NOTE: ~ 40' Centers Along E 636,000
LEGEND
Probe center, thickness in feet
Sample location, P.C., thickness
Approximate treeline configuration
E 636,000
PLAN OF TYPICAL EXPLORATORY PROGRAM IN CLOSED DEPRESSIONS
Scale: 1" = 200'
ADAPTED FROM F.P.&L. REPORT

Table 2-1 summarizes the soil strength characteristics used in the design of the embankment and the foundation as well as the average test values of those materials previously described.

TABLE 2-1

STRENGTH OF MATERIALS

| Material | Angle of internal friction in Degrees | | Cohesion in lbs/sq.ft. | |
|---|---|---|---|---|
| | Design | Test | Design | Test |
| Embankment | 35 | 38 | 200 | 0 |
| Foundation | | | | |
| Elev. +20 to -20 | 33 | None | 0 | None |
| Elev. -20 to -80 | 35 | None | 0 | None |
| Elev. -80 to -100 | 35 | None | 0 | None |

of potential embankment borrow materials was 104 pounds per cubic foot. This value represents the density of the soil with a moisture content within 2% of optimum. The average optimum moisture content of the six samples was 14.5%. The only in place density tests were performed in the power plant area. These tests also gave unit weights in the same order of magnitude. They ranged from 103 to 115 pounds per cubic foot for samples from depths between 7 and 20 feet.

It was assumed that settlement of the dike would be insignificant because the foundation was assumed to be "dense to very dense" material.

The amount of seepage was based on a flow net analysis of three typical sections. It was assumed, on the basis of the exploratory borings that an impervious layer was present some distance below natural ground under the entire embankment. The top of this layer was located at three typical elevations. The length along the embankment of these three typical sections is as follows:

| Top Elevation of impervious Stratum | Length in feet |
|---|---|
| -60 | 18,500 |
| -70 | 55,500 |
| -80 | 24,000 |
| Total | 98,000 |

It was further assumed that the permeability of all foundation material above this impervious stratum was homogeneous, but non-isotropic. The horizontal and vertical permeabilities were as follows:

$$K_h = 240 \times 10^{-4} \text{ feet per minute}$$
$$K_v = 15 \times 10^{-4} \text{ feet per minute}$$

The embankment was assumed homogeneous and isotropic with a permeability of $5 \times 10^{-5}$ feet per minute. The upstream soil cement blanket was assumed to be transparent to flow through the embankment. Assumptions concerning the embankment, however, were not important since the flow nets indicated that almost all of the flow was through the foundation. A flow net was constructed with a transformed section, based on the above 16 to 1 relationship between vertical and horizontal permeability.

The embankment stability was based on the Bishop method of slices, and was "programmed for computer solution". Presumably this means that for the assumed embankment and foundation geometry and materials characteristics, the safety factor against sliding was calculated for a series of trials of potential failure circles with various centers and radii. The "true" factor of safety then was that obtained from the trial with the least value so determined. Factors of safety for six cases were thus determined, three upstream and three downstream. The cases considered were construction (no water in the reservoir), steady seepage, and steady seepage with an earthquake (with a horizontal acceleration of 0.1g). No case for rapid drawdown was analysed.

The soil cement was designed for standard brush-weight loss test as specified by the Portland Cement Association on specimens which had been compacted to 98% of the maximum standard density at optimum moisture content after being subjected to 12 cycles of wetting and drying. Only soils with less than 1% organic content were employed. After the required cement content was determined by the above test, the cement content was arbitrarily increased by 2%. It was hoped to achieve a seven day compressive strength of 600 psi by this procedure, but most samples tested did not achieve this objective.

The numerous shallow, sink hole type depressions both in the reservoir and beneath the dike alignment at least potentially represented a special design problem. Based on the theory of the formation of the depressions as discussed above, however, the designers concluded that virtually all the soluble calcite and aragonite was removed during the Pleistocene epoc from the foundation of soils in the depressions. Hence these strata were assumed to be stable and would represent no design problem.

Another potential design problem was represented by the old railroad borrow pits on the west side of the proposed reservoir. No analyses of the effects of the presence of these old borrow pits was presented in the design report, either to show their effect on embankment stability or on seepage. A special section was designed where the embankment traversed a borrow pit. This section called for cofferdams 200 feet on either side of the embankment centerline. The area between coffer dams was then dewatered and fill material placed to natural grade before the above grade embankment was placed. Such a technique did not make any provision for locations where the proposed embankment passed near but not through an existing borrow pit. The plans did ~~not~~ call for "spoil disposal" in the old pits, but presumably placement of material in the pits was at the contractors option, as the specifications called for disposal of all unsuitable material "in depleted borrow areas or depressions located with in the reservoir".

Results of Design

Application of the assumed nature of the embankment and foundation materials and the methods of analysis previously discussed resulted in an embankment which was indicated to be very conservative and safe. The embankment stability analyses yielded very conservative factors of safety as shown in the following table.

| Condition | Factor of Safety | |
|---|---|---|
| | Upstream | Downstream |
| Construction | 1.99 | 2.92 |
| Steady Seepage | 1.92 | 1.92 |
| Steady Seepage with earthquake | 1.27 | 1.27 |

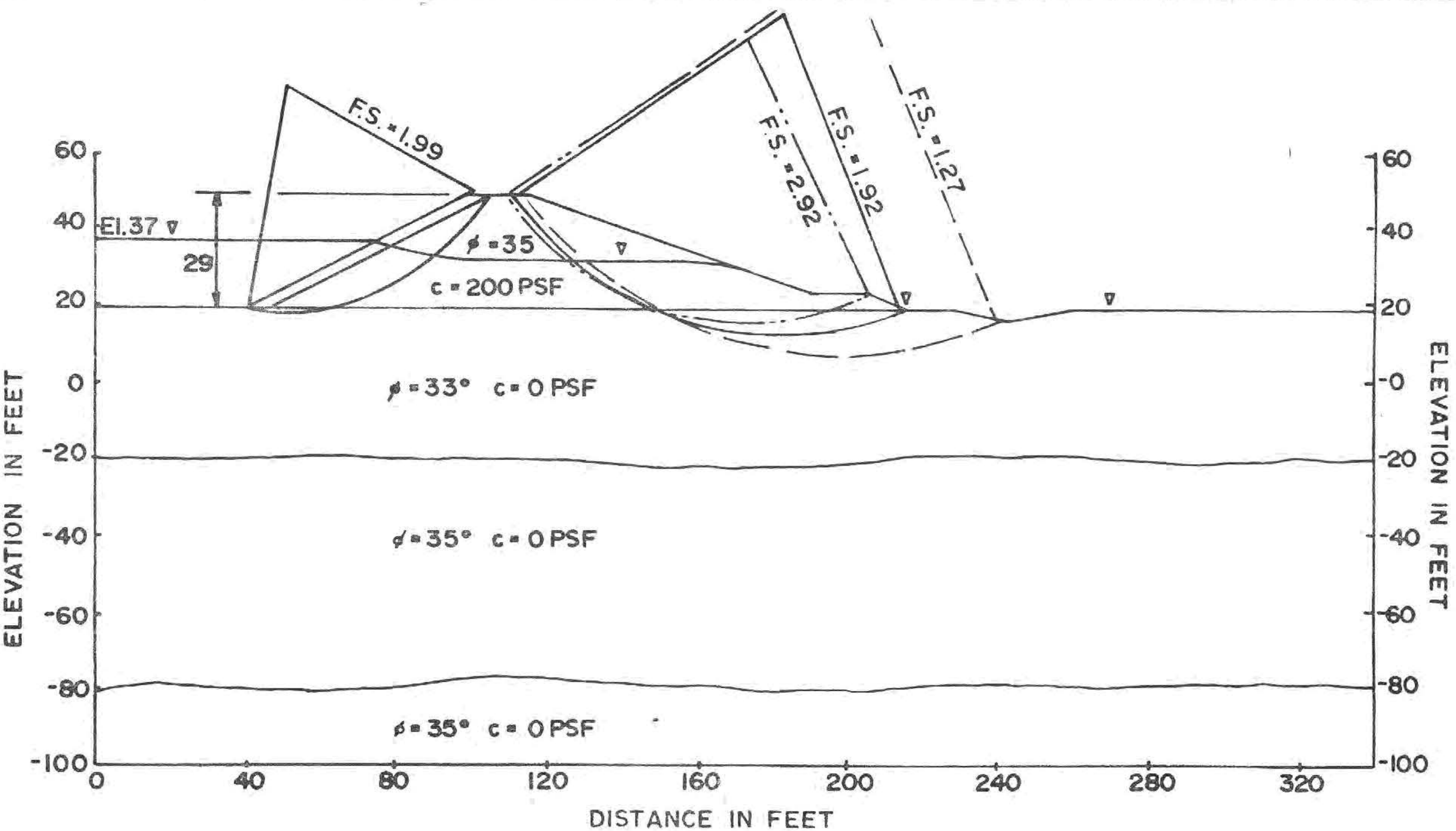
The critical failure circles are shown in Figure 2-4.

The flow net calculations for the assumed permeability and depths of pervious foundation materials yielded a total seepage from the reservoir which was a function of equation one as follows:

$$Q = 7.1 \quad (W.S. - 20) \qquad (1)$$

where Q is total reservoir seepage in cfs
W.S. is reservoir water surface elevation

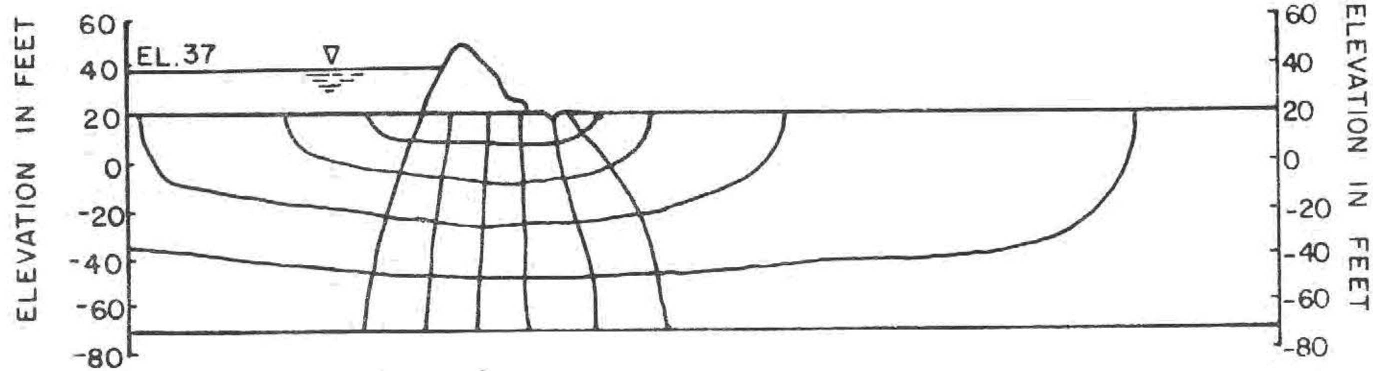
Thus, for a water surface elevation of 37, the total seepage would be 121 cfs. A typical flow net is shown in Figure 2-5.

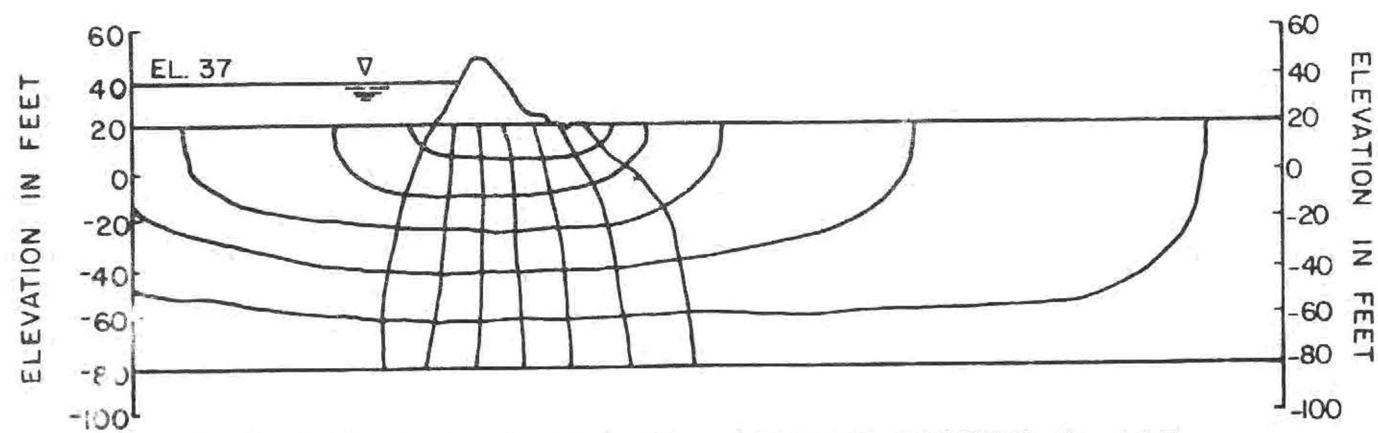



DESIGN STABILITY ANALYSIS

FROM NATIONAL SOIL SERVICES INC.
DESIGN ANALYSES & RECOMMENDATIONS
EMBANKMENT & APPURTENANT
STRUCTURES – VOL. III

Figure 2.4




**TYPICAL DESIGN FLOW NETS TRANSFORMED SECTION $K_H = 16K_V$**

FROM NATIONAL SOIL SERVICES INC.
DESIGN ANALYSES AND RECOMMENDATIONS
EMBANKMENT & APPURTENANT STRUCTURES - VOL. III

Figure 2-5

# CHAPTER 3

## CONSTRUCTION

In order to fully investigate the embankment failure, it was considered necessary to review not only the basic design and the exploratory program upon which it was predicated, but also to review the construction of the reservoir. The following therefore, is a summary of construction operations.

### Work Schedule

Based on field logs and scheduling diagrams of the design engineer's field inspectors, major work was started about the first of April, 1974 and extended into the first part of 1977, thereby covering approximately three years. The filling of the reservoir commenced in February, 1978 and was completed on April 4, 1978. During this period, various delays were experienced apparently due mostly to labor disputes and equipment modifications. Although the field logs were furnished selectively by FP&L to the investigating committee a general impression of procedures and problems was gained.

The scheduling of work varied with the character of operation. There were times when three shifts were working six days per week and other periods when one shift was adequate. Labor problems made three shifts less productive and therefore less desirable.

On of the earliest of construction operations was the plugging of all existing wells in the reservoir area. Much attention was given to this item and a well-drilling firm of some prominence was awarded this contract which took place while all such wells were still undamaged by equipment or otherwise undisturbed.

Equipment

Equipment used in earthwork was of the usual dozer, motorized van, and dragline types,but the soil cement applicator was very sophisticated and included special modifications to dozer, roller and conveyor to effect a train arrangement. This last item was developed to suit this project and was the cause of significant delays to construction of the soil cement upstream embankment facing. Until it was suitably perfected, significant amounts of soil cement were rejected and replaced. Crumbling of the leading edges of the soil cement risers of each lift was initially experienced along with inadequate compaction. This was corrected by the addition of a curved flange on the outer edge of a larger compacting roller which retained and shaped the leading edge of the freshly placed mix. This was one of the numerous unique adaptations to conventional construction equipment which was found necessary.

Embankment Construction

Embankment construction generally followed the design specifications. Logs occasionally referenced routine problems. Not all remedial work was likewise reported.

The borrow pits were all located within the reservoir confines but did not always honor the required 250 feet set-back from the centerline of the embankment as defined on the plans. This is apparent from construction photos and post-failure observations. Any possible variance can not be determined in the area of failure however due to scour and because as-built survey data on borrow pits were not available. Drainage ditches were used throughout the interior and were of considerable depth. One ditch east of the breach area (well beyond the 250 ft. set-back requirement) was re-excavated during post failure investigations and was found to have been originally dug to about elevation 14.0 that is, approximately 6-8 feet deep. The appearance of this feature does not appear to be significant.

Dewatering of borrow pits through the above ditches appears on photos to have been carried through embankment areas at several places, probably by pipes. Methods of refilling or abandonment condition of this system does not appear in logs. Post failure inspection of at least one location (station 353+00±) within the reservoir shows obvious traces of settlement and exposed debris.

Treatment of railroad borrow pits was recorded in some detail concerning stripping deleterious material and adequately compacting the backfill. Only those portions under the embankment were so treated. As reported elsewhere in this report, exterior borrow pits were not completely filled. Such pits apparently existed only along the western boundary of the reservoir. Records indicate that the embankment area was given much attention by inspectors with clearing, grubbing, construction inspection trench, filling, compacting, and shaping all prominently recorded in field logs. Problems appearing consisted mostly of poor compaction due in part to inadequate pre-draining and organic contamination of borrow stockpiling. The follow-up remedial work was not always recorded and those cases requiring rechecking can only be taken for granted. One location prominently cited for such a problem was in the Barley Barber Swamp area. It may be coincidental that this area was the place mentioned in an anonymous post-failure report to investigating agencies. See section Embankment Performance for further discussion.

Compaction in 9 inch layers to 95% of the maximum density by ASTM D 698-70-C and D1556-6Y methods was required on embankment construction. Although there is no way to verify the actual layer thickness, it is clear from the records that the density of compacted fill was significantly monitored. Much effort was spent in verifying the effectiveness of compacting equipment and logs indicate at least one type of vibrating roller was replaced due to low efficiency as indicated by test results.

Soil cement protection work was to follow a very comprehensive specification including an in-place density of fresh mix to 98% of maximum density by the same testing methods as above. This phase of embankment construction required by far the most effort as described previously. Large areas were rejected due to various deficiencies including improper thickness, low density, insufficient width, and damage by equipment. These areas were removed for lengths of several hundred feet and redone.

Post failure examination showed no evidence of poor or mislocated foundation layers although wide horizontal cracks subsequently indicated some shifting of soil cement. It has been concluded that this phenomena occurred due to the rapid draw-down at failure and that during the short time in service the facing was generally intact. Cracking and displacement is obviously more widespread in the interior embankment soil cement.

Disposal of spoil was specified to be placed within the confines of the reservoir and be buried with at least 3 feet of cover but with no specified compaction. This spoil would have consisted of the grubbed material and construction debris but only after burning combustibles.

Logs indicated some problem in getting this disposal done properly. This committee called attention to one area (mentioned previously) in which discarded construction waste (metal and rubber hose) material is exposed. It is presently expected that this will be an action item in the FP&L remedial program.

Special Problems

Special problems during construction which impacted upon progress were in the areas of labor, equipment, and materials. From the reports, weather did not appear to be a significant problem.

There were numerous labor disputes during constructon, details of which are not clear, but which shut down the job on at least several occasions. Shorter periods of work stoppage routinely occurred. For example, during one particular night shift the entire equipment operator crew, as a group, left "sick" and the job was shut down until the following day.

Equipment downtime of the earth movers did not appear to impede significantly the project but the soil cement equipment certainly did. As mentioned earlier in this report, the cement application train was modified or replaced on at least two occasions with delays extending into weeks.

On several occasions, cement delivered to the pug mill batching plant was rejected due to leakage of moisture into the railroad cars.

There is no accurate way to measure any harmful effects to the construction caused by the above problems. In view of the vast area and the long construction period, the problems cited do not appear unique.

Inspection

Inspection of construction consisted of two separate operations. One was by on-site resident inspectors on a day to day basis and the other was by the team of design consultants scheduled periodically, with special interim inspections as required. The resident inspectors were assigned to cover each shift although there is the impression that work considered "non-construction" was not attended by the resident inspectors.

Reporting, based on what has been displayed to this committee, consisted of daily field logs by resident inspectors and weekly summaries. The contents of these reports varied with the author but ranged from vague progress reports of quantities of work items to others which actually discussed significant problems and their solutions. The solutions to problems mentioned in some report generally did not appear in subsequent logs, leaving the impression that even though discrepancies were noted they may or may not have been corrected. One specific example of this problem was the identification of wet uncompacted fill in the Barley Barber area which was obviously a serious concern to the inspectors.

Monthly inspection reports by the design engineers reflected more design related topics and there is evidence that these inspections had a strong influence on the conduct of the construction work. These reports likewise provided much assistance in the way of background to this committee.

CHAPTER 4

# MONITORING

## Introduction

Subsequent to the construction of any impoundment, hydrologic characteristics become a vital and fundamental part of its operation. It therefore follows that major pre-design considerations be directed to the questions associated with the operation of the reservoir and the effect and impact on the surrounding hydrologic regimen (both surface and subsurface) that occur as a consequence of the imposition of this reservoir system. The imposed stress takes on significance when viewed in terms of site conditions. The embankment that contained the reservoir is constructed of fine sand and is founded on a sand base. As a consequence, special concern and design criteria had to be directed to a careful analysis of anticipated seepage conditons and water levels. As a part of the routine associated with the operation of the reservoir, careful monitoring and reporting of seepage conditions and of water levels in piezometers installed around the reservoir would be required.

In addition to monitoring for seepage which was of primary concern during these investigations, a broad range of other observations were initially planned that was designed to monitor comprehensively the operational characteristics and stability of the reservoir. Thus, provision was made to maintain downstream slope vegetation, to clear the toe ditch vegetation, to inspect the crest road and toe road, analyze strip chart water level recorders in order to detect any abrupt change in reservoir level, to check flows in the toe ditch, & surface drainage facilities to determine whether there were any unaccountable changes in flow, to collate various derived hydrometeorologic data into a comprehensive water accounting system.

Design

Prior to construction of the reservoir, the design consultants determined that a comprehensive monitoring network was essential. In order to operate the system in a consistent and a technically accurate fashion a reservoir surveillance, operation and maintenance manual was developed. This monitoring network was of two general types -- instrumented and non-instrumented. The discussion which follows describes this system in some detail.

Non-Instrumented Monitoring

The non-instrumented portion of the monitoring program consisted of several discrete activies. The monitoring was performed by FP&L reservoir operation personnel. These may be summarized as follows:

As originally conceived and recommended, daily observations were conducted by personnel traveling both the service road on top of the embankment and the road running along the downstream toe. In general, the observations included such items as embankment stability (sloughs, slides, gullying, washouts, seeps), and at the downstream toe area that included toe ditch boils, seeps and boggy areas or other unusual or abnormal conditions.

These inspections were to be conducted "until the reservoir and its embankment are judged to have reached a stable condition."

Inspections were conducted of the reservoir and surrounding area on a weekly time frame. These inspections included observations of those same parameters noted for the daily inspections outlined above, and additionally included specific observation of the embankment slopes and ditches for erosion and the inspection of culverts and drain pipes for plugging. Additionally these inspections were made immediately following each "heavy" rainstorm event. On the upstream side of the embankment, the weekly inspection included a check of the soil cement for signs of serious erosion or damage particularly after periods of sustained high winds.

Quarterly inspections were conducted during February, May, August, and November. The November and May inspections coincided with semi-annual and annual inspections described below. During these inspections specific attention was directed toward development of unfavorable conditions at the downstream slope and embankment toe ditches such as boils, seeps, boggy areas, sloughs, etc. Appurtenant facilities were similarly inspected. These include the spillway area, plant circulating water intake and discharge areas and the reservoir water supply pipeline area.

Inspections incorporating the quarterly routine were supplemented by examination by boat of the upstream soil cement slopes for evidence of cracks, subsidence, serious erosion or deterioration; culverts and headwalls were inspected for evidence of blockage by debris and for effectiveness. In addition, appurtenant structures were examined, including spillway, plant intake and discharge structures and reservoir water supply pipelines and discharge structures. Of specific concern during these inspections were such items as abnormal settlement or lateral movement, cracking of concrete or differential displacement of horizonal and vertical joints, deterioration of concrete and abnormal leakage around abutments and construction joints or contraction joints.

An annual inspection was made of the entire facility by the Engineer familar with the embankment design and the expected performance of the facility. One such inspection was made during May, 1979. This inspection incorporated all of the items covered during earlier inspections.

In addition to monitoring described above, nine comprehensive "special" inspections were made by the Board of Consultants. The results of their inspection were detailed in report form which included recommendations for repair or modification of specific items.

Instrumented Monitoring

In addition to the non-instrumented monitoring of reservoir performance as described above, significant instrumented monitoring was also conducted. An analysis of the results obtained from this system is contained elsewhere in this report.

Groups of piezometers were placed at seven locations around the periphery of the embankment. At each location from 7 to 9 piezometers were arranged in a line outward from the embankment centerline with the tips of each sealed at different levels in order to measure the piezometric pressures in the various strata. The piezometers were read twice monthly prior to reservoir filling, weekly during reservoir filling, and later monthly until the failure occurred on October 30, 1979. Thereafter, they were read daily until the piezometric levels receded to pre-filling levels.

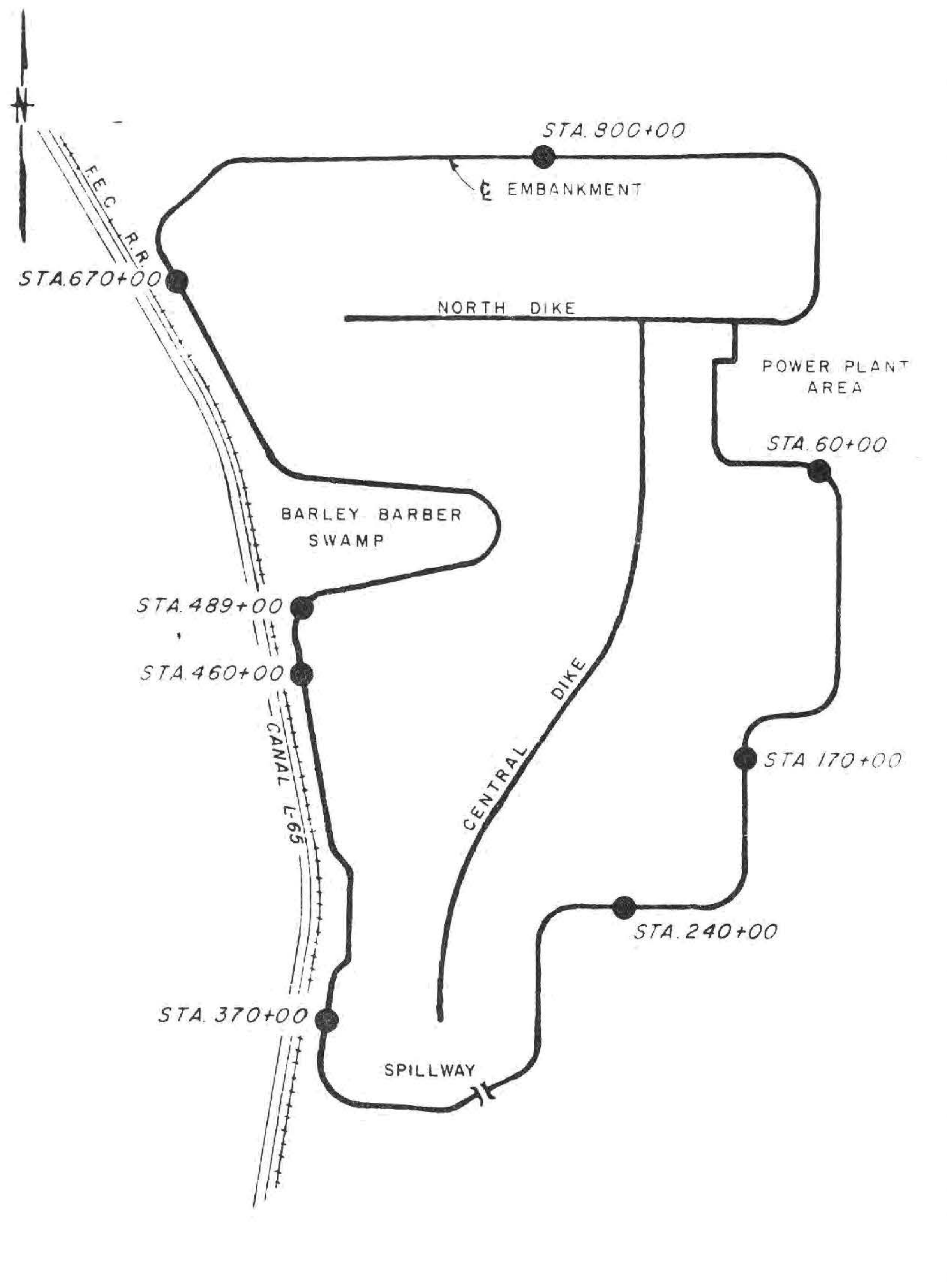
The location of the piezometer stations are shown in Figure # 4-1 and a typical profile along one of the networks is shown as Figure # 4-2 . The essential purpose of the piezometers was to monitor water levels before reservoir filling and subsequent to that time. From an analysis of these data, a well designed and properly installed network could provide important advance warning concerning the stability of the reservoir embankment and its impact on the hydrologic regimen of the affected peripheral areas.

The reservoir stage was recorded continuously at three stations located at the pump station on the south side of the reservoir, and the intake and outlet to the power plant near the northeast corner of the reservoir.

Rainfall and wind were measured at four stations on the embankment. Thiessen polygons were used to estimate the direct rainfall contribution of each of these stations to the reservoir. Their location (shown on Fig 1-1 ) and percent contributions are as follows:

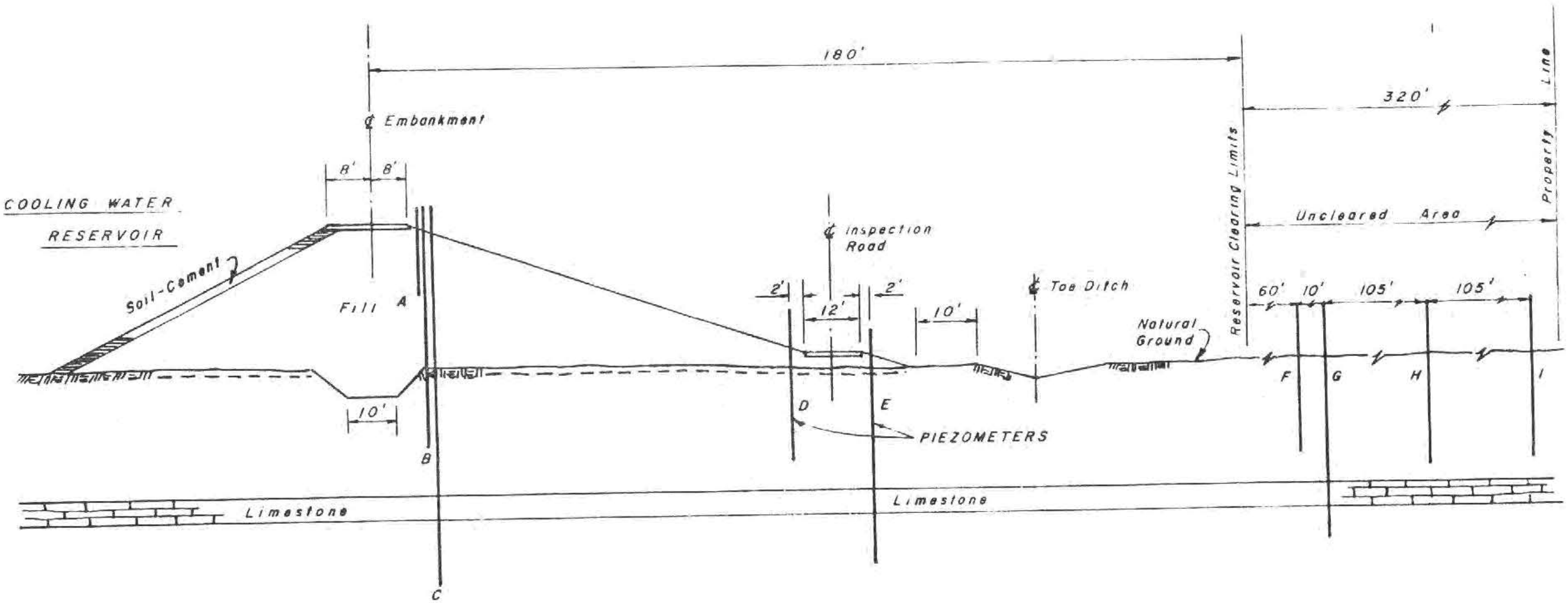
| Station | Location | Percent of Direct Rainfall |
|---|---|---|
| A | Power Plant | 40 |
| B | Pump Station | 20 |
| C | Barley Barber Swamp | 25 |
| D | North West Corner | 15 |

In addition to the visual seepage observations inherent in the periodic reservoir monitoring, some special studies were conducted for the purpose of obtaining an environmental background base line. The impetus for these studies was the fact that FP&L plans for future power units to be fueled by coal, in contrast to the present petroleum-fueled units. These




PIEZOMETER PLAN

ADAPTED FROM F.P. & L. REPORT




TYPICAL PIEZOMETERS TRANSECT

ADAPTED FROM F.P.&L. REPORT

Figure 4-2

studies, though not part of the original monitoring system, shed some interesting light on the reservoir seepage.

In that study a series of water control structures and water stage recorders were installed at various points north and east of the reservoir. The records of these gages extend from April 13, 1979 to present. The locations of the two gages of interest to this investigation are shown on Figure 1-1 as R1 and R3.

The rate of pumped inflow was continuously recorded for each of the four pump units. The instrumentation circuitry, by integrating the pumping rate with respect to time, calculated the total daily discharge of each unit.

## Monitored Embankment Performance

In the previous section, the monitoring network, its scope and purpose was described. In the section that follows, this network is evaluated and analyzed in terms of its adequacy relative to reservoir performance.

### Inspection Reports

A most thorough and practical list of items was included in the inspection reports. It is understood from questioning by this committee that they were indeed carried out initially for some time. Due to the repeated "no change" reports, the actual notation was eventually reduced to comments made on the inspector's time sheet. An examination of time sheets revealed that there was usually nothing noted at all leaving the obvious question, "Was the reservoir inspected?" Although it is understandably practical from a paperwork standpoint, the human element tends to diminish the true value and significance of the inspection.

Inspection reports and other communications between designers and FP&L revealed the development of significant seepage boils at various locations around the reservoir. These boils appeared at or near the downstream toe of slope. Reports had them located on the easterly side of the reservoir, however, pre-failure plot by FP&L also located them on the other three sides, none in the area of failure. In the latter area there was not even dampness on the inspection road as was the case in over half of the embankment length. Reports also indicated the existance of numerous boils, several as much as "twelve inches in diameter". All communications noted indicated little concern with boils of this size, and directions were given by the designers for repair by digging to the bottom of the piped hole and filling with crushed stone filter media. These repairs were confirmed in writing by FP&L. The absence of these boils in the failure area later provided a significant element in analyzing the mechanics of the event.

During the period from completion of downstream embankment face until filling, very significant erosion occurred as evidenced by photos and field log notation. This erosion appeared to be quite deep and cover a wide area of the total embankment area. Although the failure is not attributed to this condition, it will be recommended that during the initial period of ground cover growth that a more timely repair be made as rains erode the freshly built areas.

A strict setback requirement for excavation of any kind should be observed. Just prior to failure several small borrow pits were dug adjacent to the Barley Barber Swamp area to provide fill to nature paths. This condition would possibly contribute to an adverse situation similar to that caused by the railroad borrow pit configuration.

The condition of the soil-cement embankment, although reasonably sound prior to failure, has experienced considerable subsidence and deterioration due to the rapid drawdown at the time of failure. With detailed repair it can be returned to its originally sound condition.

As previously described herein, observations of seepage was inconclusive, but beyond that no changes in the magnitude of seepage were noted by any-one prior to failure. Therefore, determination of total seepage was marked by many factors including overall reservoir size, gaps in monitoring, surface boils and dampness at the downstream toe.

A special part of the overall monitoring program concerns a number of compreshensive inspections made by the Board of Consultants at various times during the period of construction and operation of the reservoir. These inspections encompassed the entire reservoir area and appurtenant structures and facilities. Three of these were singled out for specific review and comment.

a. April 17,18, 1978 - This inspection was conducted at that critical point in time when the reservoir first reached normal stage and the surrounding hydrologic regimen was initially responding to the imposed stress. In addition, all associated facilities received their first test associated with pool stage operations.
b. December 7, 1978 - A complete review of reservoir and appurtenant areas was made after the reservoir had been full for a number of months and the entire facility had reached what may be described as steady state conditions.
c. May 24,25, 1979 - This inspection was made as a part of the annual routine associated with the operation and performance of the reservoir.

In reviewing these reports, is appeared pertinent to examine the findings of these experts, not only to examine and review a historical record of events in which the District did not participate, but possibly more importantly to examine those elements of their recommendations, specifically to focus on problem areas they had uncovered. The objective here is to evaluate the management function as it relates to the reservoir in order to determine the degree of responsiveness of management to those recommendations.

The following, therefore, summarizes the consultants reports defined above. District staff commentary as appropriate is also included.

## Inspection - April 17 & 18, 1978

Immediately after filling the reservoir, sand boils had become evident at several locations around the reservoir at the downstream toe of slope. These phenomena are not uncommon in dam construction and their appearance is not viewed with alarm as long as they are kept under constant surveillance, are of small diameter and as observed, do not carry volumes of sand. The development of sand boils thus observed was kept under a daily inspection routine. A program was immediately initiated to cover larger boils with filter sand in order to stabilize this conditon. Most of the boils were less than six inches in diameter although one, at about station 740, was 12 inches in diameter. Additionally most of the boils became "inactive" with time in that their ability to move sand ceased. A program to effect remedial repair was begun almost at once and has been continued as required since that time.

Somewhat associated with boils, seepage through the embankment was evident along major portions of the downstream toe road at a number of locations.

Although not considered hazardous, it was recommended that immediate corrective action be taken. The remedial work recommended and accepted involved the placement of a filter sand and perforated pipe drain along the toe road area. Although recommended in April 1978, no action was taken as of October 30, 1979 - the day of the failure.

The general subject area of seepage was addressed in two of its aspects; (a) seepage looses from the reservoir, and (b) flow measurements of seepage from discrete reaches of the toe ditch. As originally designed, reservoir seepage losses were grossly calculated as a part of the reservoir accounting program. This accounting program is discussed in detail elsewhere in this report.

The overall conclusion reached by the consultants as a consequence of the inspection was that the embankment was performing satisfactorily, recognizing the repairs and maintenance items specified above.

<u>Consultants Inspection - December 7, 1978</u>

A second inspection relevant to the reservoir embankment failure investigation was conducted for the purpose of reviewing "the present condition and recent performance of the embankment after the first rainy season of its life." In summary form, the conclusions reached by the consultants as a consequence of their inspection are as follows:

1. "The dam is performing well and there are no serious problems."

2. "The present efforts to obtain a good protective grass cover on the downstream slope should continue, as well as the current surveillance program."

3. "It is not essential to rehabilitate the wet and soft toe road immediately, provided the periodic inspections of the dam toe are made with vehicles capable of travelling along the wet toe area."

4. "We agree with Mid Valley proposal for simple staff gages at about 14 points along the toe ditches, as a compromise solution to monitoring flow in the ditches."

<u>Consultants Inspection - May 24, 25, 1979</u>

In summary form, the following represents key observations made during this inspection:

1. Establishment of a good grass cover on the downstream slope was again emphasized as desirable, along with filling of gullies caused by erosion.

2. Seepage from the embankment was noted to occur over perhaps 50% of the area. It was recommended that remedial repairs suggested in an earlier report be effected.

3. Clearing of tall vegetation that obscured inspection of the toe ditches was recommended. This activity was already underway and was to continue.

4. Sand boils were examined and described. Recommendations were made to continue their observation and to repair those boils that were considered to be significant, consistant with procedures previously established.

5. The soil cement on the upstream slope was inspected by boat.

Although certain limited areas do show some signs of wear, they were minimal and not considered to be of a serious nature.

6. All other elements of the inspection appeared to be satisfactory.

District investigating team comments to the summary conclusions and to certain aspects contained in the body of the consultants reports are as follows:

1. "Piezometers... show remarkably constant conditions with no appreciable influence of the rainy weather."
2. The sand boils noted in the previous consultants reports appeared to have diminished in number and activity and thus were not a serious problem especially in that there was no apparent proliferation of this phenomenon.
3. It was the general impression that the total quantity of seepage water in the toe ditches appeared to be about the same as observed during the reservoir inspection and thus the conclusion was reached that the sand foundation was generally behaving as intended in the design and is under control.

4. The wet areas in the downstream toe road enlarged and at the time of inspection increased in linear length to cover 58% of the circumferential area of the reservoir and that specific reaches of toe road that were wet ranged from 50 feet to as long as 11,000 feet; and that vehicular access via the toe road, while not absolutely essential, the consultants hesitated to recommend abandonment of this inspection route. The consultants further agreed that the placement of staff gages as a resonable compromise to the actual measurement of seepage ditch flows was acceptable since this aspect of monitoring was perceived only as a system to detect changes in the rate of seepage, and not as a method of measuring total seepage.
5. As a final comment, the Consultants indicated that they were impressed at the then current level of surveillance and the maintenance program being carried out.

Instrumented Monitoring

In addition to the non-instrumented or visual monitoring system, the instrumented monitoring sytem was designed to produce results which could contribute essential data for determining how the reservoir was performing.

In analyzing the piezometer record, the Committee recognized some highly anomalous readings. Accurate surveys were run to determine the elevation of the measuring point, since all results are reported in terms of mean sea le vel and an error in this datum could result in meaningless data and/or possibly dangerous conclusions. The results of this survey are presented as Table 4-1. From a review of these data, it is readily apparent that numerous major errors exist.

TABLE 4-1

COMPARISON OF ELEVATION OF MEASUREMENT POINT-FPL vs WMD SURVEY 2/7/80

| | ELEV/FPL | ELEV/WMD | DIFFERENCE (FT.) |
|---|---|---|---|
| STA 60 | | | |
| A | 52.5 | 52.4 | 0.1 ↓ |
| B | 52.8 | 52.6 | 0.2 ↓ |
| C | 52.6 | 52.4 | 0.2 ↓ |
| D | 34.6 | | |
| E | 33.3 | 33.0 | 0.3 ↓ |
| F | 30.6 | 30.4 | 0.2 ↓ |
| G | 30.9 | 30.3 | 0.6 ↓ |
| STA 170 | | | |
| A | 53.2 | 53.6 | 0.4 ↑ |
| B | 53.1 | 52.4 | 0.3 ↑ |
| C | 53.0 | 52.3 | 0.7 ↓ |
| D | 30.4 | 30.3 | 0.1 ↓ |
| E | 26.4 | 28.3 | 1.9 ↑ |
| F | 27.9 | 27.9 | 0 |
| G | 28.6 | | |
| STA 240 | | | |
| A | 52.9 | 52.7 | 0.2 ↓ |
| B | 52.8 | 52.7 | 0.1 ↓ |
| C | 52.9 | 52.7 | 0.2 ↓ |
| D | 30.1 | 28.2 | 1.9 ↓ |
| E | 29.2 | 29.0 | 0.2 ↓ |
| F | 26.1 | 25.9 | 0.2 ↓ |
| G | 26.0 | 25.8 | 0.2 ↓ |
| STA 370 | | | |
| A | 52.1 | 52.0 | 0.1 ↓ |
| B | 52.1 | 52.1 | 0 |
| C | 52.2 | 52.1 | 0.1 ↓ |
| D | 25.6 | 27.5 | 0.9 ↑ |
| E | 27.4 | 26.8 | 0.6 ↓ |
| F | 22.4 | 22.1 | 0.3 ↓ |
| G | 22.2 | 22.3 | 0.1 ↑ |
| H | 22.2 | 22.2 | 0 |
| I | 22.3 | 22.4 | 0.1 ↑ |
| STA 460 | | | |
| A | 51.9 | 51.7 | 0.2 ↓ |
| B | 52.2 | 52.2 | 0 |
| C | 52.4 | 52.4 | 0 |
| D | 29.1 | 29.1 | 0 |
| E | 26.6 | 26.6 | 0 |
| F | 25.4 | 25.4 | 0 |
| G | 25.3 | 25.4 | 0.1 ↑ |
| H | 25.9 | 25.8 | 0.1 ↓ |
| I | 27.0 | 27.0 | 0 |
| STA 670 | | | |
| A | 52.0 | 51.9 | 0.1 ↑ |
| B | 52.2 | 52.1 | 0.1 ↓ |
| C | 52.0 | 51.9 | 0.1 ↓ |
| D | 30.2 | | - |
| E | 27.5 | 27.4 | 0.1 ↓ |
| F | 27.0 | 27.0 | 0 |
| G | 27.2 | 27.1 | 0.1 ↓ |
| H | 27.3 | 27.2 | 0.1 ↓ |
| I | 26.8 | 26.7 | 0.1 ↓ |
| STA 800 | | | |
| A | 52.4 | 52.4 | 0 |
| B | 52.6 | 52.6 | 0 |
| C | 52.5 | 52.5 | 0 |
| D | 35.7 | 35.6 | 0.1 ↓ |
| E | 34.2 | 34.1 | 0.1 ↓ |
| F | 30.7 | 30.5 | 0.2 ↓ |
| G | 30.5 | 30.4 | 0.1 ↓ |

Six elevations are incorrect by more than 0.5 feet, and in two instances, the error is as much as 1.9 ft.

In consequence of the events described above, District staff remeasured the total depth of the wells. In this survey, several of the outlying piezometers were omitted. The results of this survey indicate that all the wells contain some sediment in the bottom. A number of them have several feet of mud and debris. Our concern here is that of plugging and the possibility that water level readings taken did not reflect true levels due to this clogging.

The "A" piezometers in each of the seven transects were installed at the side of the service road on top of the embankment. The vital function of these piezometers ~~were~~ was to measure the phreatic surface in the interior of the embankment, ~~and~~ which is the "connecting link" in establishing the potentiometric gradient between water levels inside the reservoir and that which exists at the downstream toe of the embankment. When the elevation of water levels in these wells is compared to the installed total depth of the wells, it is seen that the depths to water measured are deeper than the wells themselves. This of course, suggests that on face value the data reported ~~is~~ are meaningless. On investigation, it was determined that the wells were actually installed up to several feet deeper than reported, which would account for this dilemma. Nevertheless, considering the "true" depth of the wells as subsequently measured compared to water levels reported and considering the fact that soft mud and other sediment exists at the bottom of these wells, there is a significant degree of uncertainty as to whether the water levels measured were actually that or rather were they measuring the water entrained in the soft mud existing in these wells? Table 4-2 displays the comparison between installed depths as reported with the results of District's surveys.

Through the approximately 28-month history of measurements and concurrent use of the downstream toe road and service road by various personnel in the pursuit of other tasks, many of the piezometer wells were destroyed or damaged such that their use was compromised. Thus, at the time of the last set of piezometer readings (October 10, 1979) taken prior to the embankment failure, at least five piezeoeters, around the reservoir, in the embankment (the most critical point in the piezometer transect) were inoperative. Some of these piezometers were in this derelict condition for some considerable period of time.

As a final point of concern regarding the piezometer network, District staff questions the its density. Aside from the several "special" piezometers located at the pump station and spillway, a total of seven transects were installed around a reservoir having a circumferential length of some 19 miles. This equates to a data point every 2.7 miles. If one can recognize that the piezometers, particularly those on the center line of the embankment and at the downstream toe, could forwarn of the onset of structural defects or hydrologic problems arising out of the operation of the reservoir, it would appear that the density

TABLE 4-2

COMPARISON OF INSTALLED DEPTH OF PIEZOMETERS

| | FPL ELEV. | FPL TD | WMD ELEV. | WMD TD | DIFFERENCE IN DEPTH | |
|---|---|---|---|---|---|---|
| STA 60 | | | | | | |
| A | 52.5 | 14.5 | 52.4 | 17.90 | 3.4 | Deeper |
| B | 52.8 | 44.8 | 52.6 | 43.81 | 0.99 | |
| C | 52.6 | 75.6 | 52.4 | 73.15 | 2.45 | |
| D | 34.6 | 29.6 | | - | - | |
| E | 33.3 | 56.3 | 33.0 | 55.50 | 0.8 | |
| STA 170 | | | | | | |
| A | 53.2 | 15.2 | 53.6 | - | - | |
| B | 53.1 | 39.1 | 52.4 | - | - | |
| C | 53.0 | 57 | 52.3 | 68.20 | 11.2 | Deeper |
| D | 30.4 | 21.4 | 30.3 | 17.13 | 4.27 | |
| E | 26.4 | 40.4 | 28.3 | 42.26 | 1.76 | Deeper |
| F | 27.9 | 18.9 | 27.9 | 18.28 | .62 | |
| G | 28.6 | 42.6 | | 39.96 | 2.64 | |
| STA 240 | | | | | | |
| A | 52.9 | 15.9 | 52.7 | 18.32 | 2.42 | Deeper |
| B | 52.8 | 45.8 | 52.7 | 44.32 | 1.48 | |
| C | 52.9 | 77.9 | 52.7 | 73.51 | 4.39 | |
| D | 30.1 | 38.1 | 28.2 | 35.45 | 2.65 | |
| E | 29.2 | 54.2 | 29.0 | 49.88 | 4.32 | |
| F | 26.1 | 34.1 | 25.9 | 31.89 | 2.21 | |
| G | 26.0 | 51.0 | 25.8 | 51.85 | 0.85 | Deeper |
| STA 370 | | | | | | |
| A | 52.1 | 16.1 | 52.0 | 18.21 | 2.11 | Deeper |
| B | 52.1 | 44.1 | 52.1 | 44.08 | 0.02 | |
| C | 52.2 | 87.2 | 52.1 | 82.67 | 4.53 | |
| D | 25.6 | 37.6 | 27.5 | 27.50 | 10.1 | |
| E | 27.4 | 62.4 | 26.8 | 60.53 | 1.87 | |
| STA 460 | | | | | | |
| A | 51.9 | 15.9 | 51.7 | 17.38 | 1.48 | Deeper |
| B | 52.2 | 46.2 | 52.2 | 48.78 | 2.50 | Deeper |
| C | 52.4 | 77.4 | 52.4 | 75.69 | 1.71 | |
| D | 29.1 | 35.1 | 29.1 | 33.33 | 1.77 | |
| E | 26.6 | 51.6 | 26.6 | 49.27 | 2.33 | |
| F | 25.4 | 31.4 | 25.4 | 26.75 | 4.65 | |
| G | 25.3 | 58.3 | 25.4 | 54.82 | 3.48 | |
| H | 25.9 | 32.4 | 25.8 | | | |
| I | 27.0 | 32.8 | 27.0 | 31.01 | 1.79 | |
| STA 670 | | | | | | |
| A | 52.0 | 16.0 | 51.9 | 17.08 | 1.08 | Deeper |
| B | 52.2 | 41.2 | 52.1 | 40.48 | 0.72 | |
| C | 52.0 | 72 | 51.9 | 38.64 | 33.36 (?) | |
| D | 30.2 | 33.2 | | | | |
| E | 27.5 | 47 | 27.4 | 47.49 | 0.49 | Deeper |
| F | 27.0 | 24 | 27.0 | 23.46 | 0.52 | |
| G | 27.2 | 47.1 | 27.1 | 46.35 | 0.75 | |
| H | 27.3 | 23.8 | 27.2 | 23.13 | 0.67 | |
| I | 26.8 | 24.2 | 26.7 | 23.90 | 0.3 | |
| STA 800 | | | | | | |
| A | 52.4 | 14.4 | 52.4 | 17.47 | 3.07 | Deeper |
| B | 52.6 | 41.6 | 52.6 | 40.58 | 0.6 | |
| C | 52.5 | 71.5 | 52.5 | 70.75 | 0.75 | |
| D | 35.7 | 36.7 | 35.6 | 37.66 | 0.96 | Deeper |
| E | 34.2 | 53.2 | 34.1 | 53.16 | 0.04 | |
| F | 30.7 | 31.7 | 30.5 | 32.16 | 0.46 | Deeper |
| G | 30.5 | 49.5 | 30.4 | 51.96 | 2.46 | Deeper |

of the monitoring network is inadequate particularly when viewed in conjunction with several other problem areas which have been described above. It is, therefore, obvious that a significant, and fundamental improvement to the entire question of monitoring is necessary.

An analysis of the corrected piezometer results is contained in the section of this report concerning the Failure Investigation.

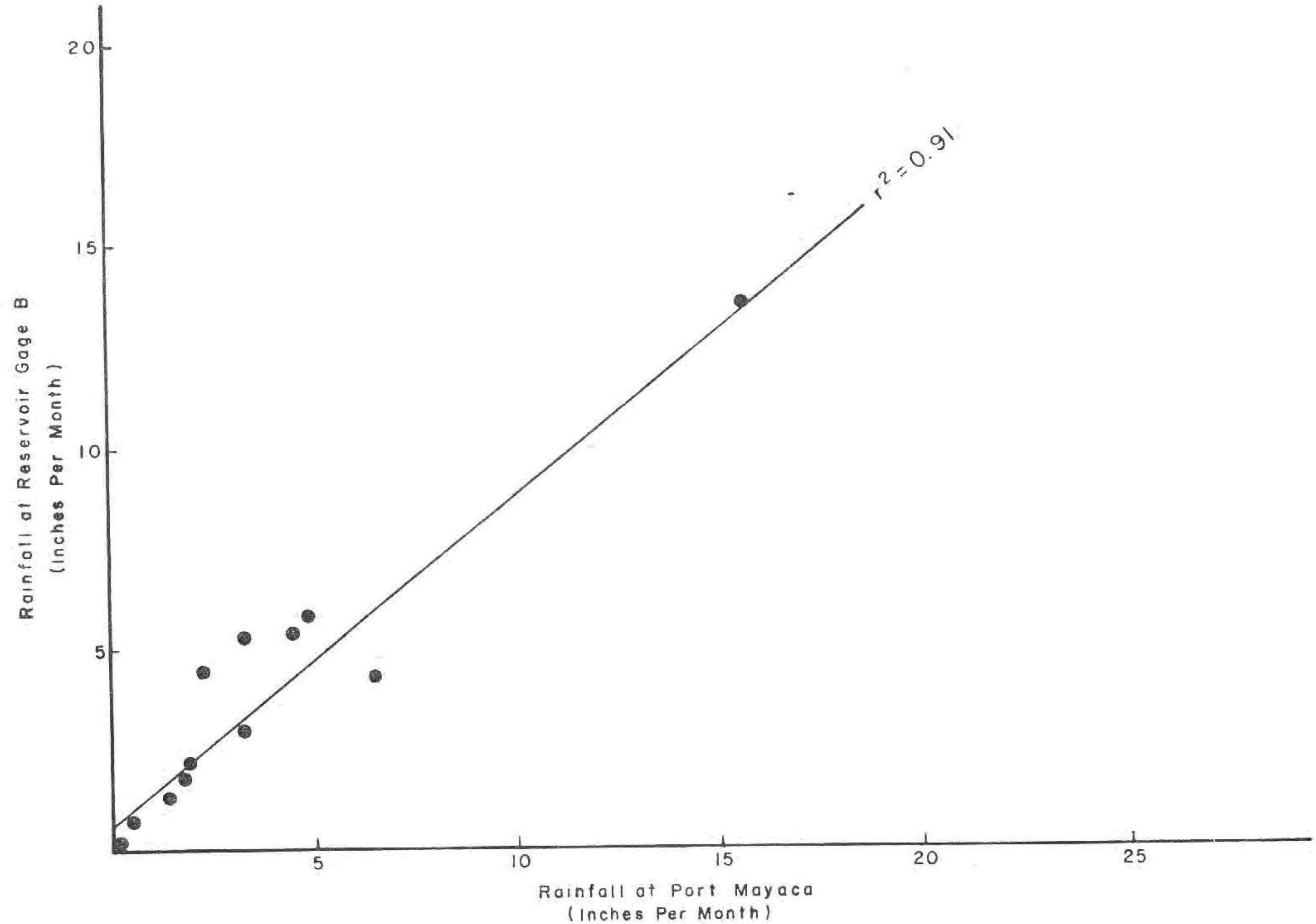
The piezometric pressures were read periodically, as described previously. Apparantly the results of these measurements were not deemed to indicate any unusual conditions or events, because they did not give rise to any significant comment in any of the inspection logs or consultants reports.

The design of the rain gage system is deficient in two respects, according to the standards of the World Meteorological Organization. First, the four-inch diameter rain gages are too small. The WMO recommends a 200 to 500 $cm^2$ catchment opening, or a diameter of six to 10 inches. Second, the gages are improperly located atop the 30-foot embankmant.

The mouth of the gage should be located as close as possible to an optimum 30 cm. above surrounding level ground.

The effect of the embankment is to cause strong air currents which in turn cause anomalous rainfall, as measured by the gage.

Although the above criticisms of the rain gages are theoretically valid, a comparison of these records was made with the record of the U.S. Corps of Engineers standard gage at Port Mayaca, see Fig. 4-3, several miles southwest. This comparison consisted of a regression analysis of the monthly rainfall in 1979. The correlation coefficient ($r^2$) for these




RAINFALL CORRELATION BETWEEN PORT MAYACA & RESERVOIR GAGE B 1970 four records was from 0.89 to 0.94, indicating an excellent correlation, considering the fact that a perfect corellation has a coefficient of 1.00.

Part of the District's permit to FP&L authorized replacement of seepage and evaporation losses from the reservoir to maxima of 40,000 and 50,000 acre-feet per year respectively. Consequently, it was important to estimate these losses with reasonable accuracy to determine authorized makeup pumping requirements and to monitor any ususual events from a safety standpoint. Since seepage was not originally measured directly, the amount of seepage was estimated from the balancing of the hydrologic equation of the reservoir as follows:

$$\Delta St = P + R - E - O - S \qquad (2)$$

where the meaning of all terms, expressed in Acre-feet, are as follows:

$\Delta St$ = Change in storage

P = pumped inflow

R = rainfall

E = evaporation

O = reservoir releases

S = seepage

All quantities were measured except the seepage. The problem with such a method, however, is that all the errors in any of the measurements are thrown into the seepage function. The reservoir releases were zero during the entire period from the filling to the failure. Presumably, the change in reservoir storage could be calculated quite accurately. The rainfall and the evaporation figures were imperfectly measured. Rainfall in Florida varies greatly from point to point, especially during periods of heavy summer thunderstorm activity, when most of the rainfall occurs. As was pointed out, the rain gage system as constructed leaves something to be desired. Because of the excellent correlations, it is suggested that further study be devoted to the applicability of the present system before any modifications are considered.

Finally, evaporation is very difficult to estimate. It was estimated on the basis of an empirical relationship with windspeed and water vapor pressure at the reservoir surface, developed by John Hopkins University for the Edison Electric Institute. This relationship gave unreasonable results according to FP&L and was modified by them in order to yield "reasonable" evaporation values. In its modified form, the relationship is expressed by the following formula:

$$E = 1.7405 \times 10^{-5} (e_s - e_d)(70 + 0.7W^2) \qquad (3)$$

Where E = total daily reservoir evaporation in inches of depth

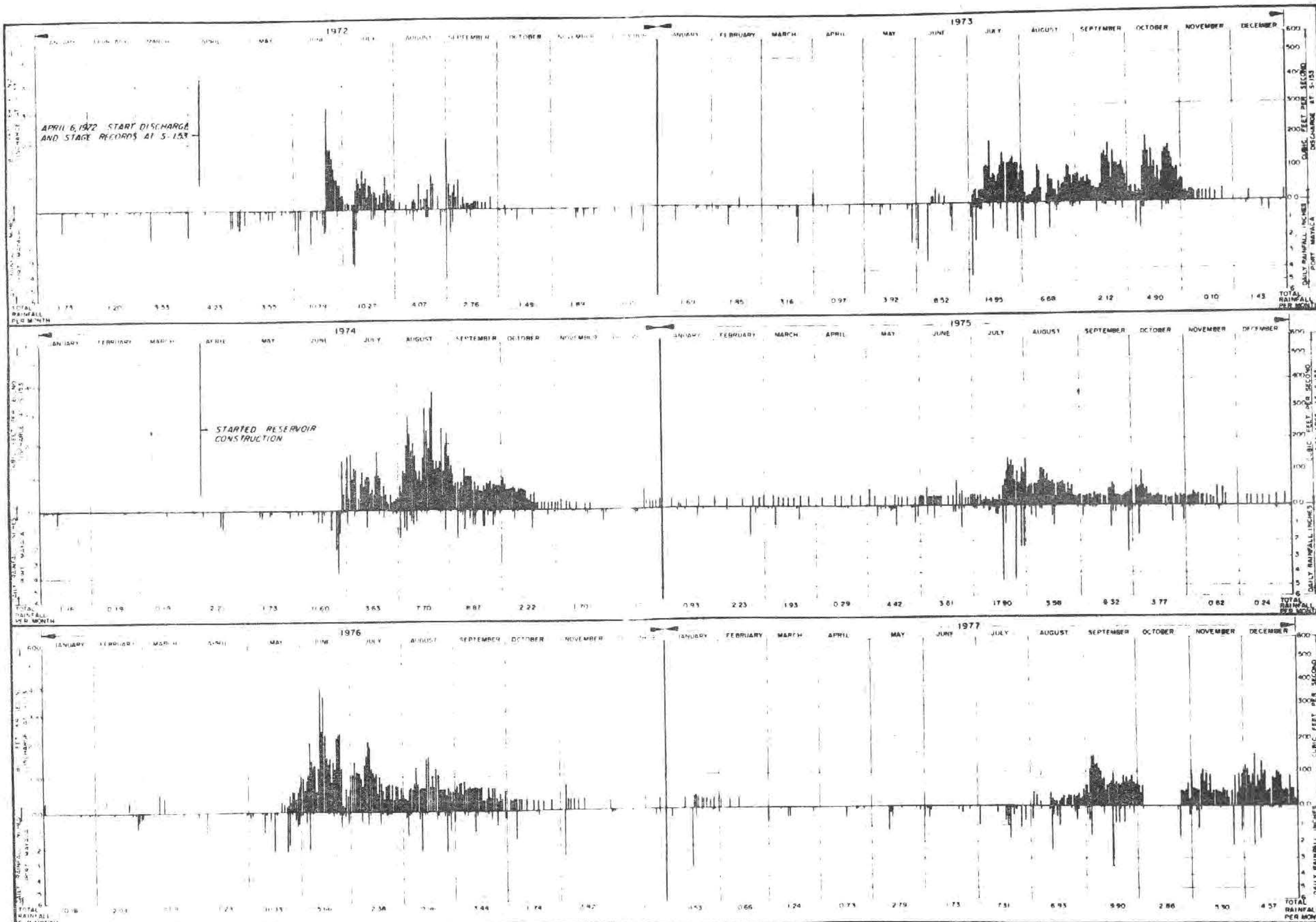
$e_s$ = saturated vapor pressure of the reservoir water surface, expressed in inches of mercury, at the average surface water temperature.

$e_d$ = absolute vapor pressure, expressed in inches of mercury, at the temperature of the average daily dew point.

W = average daily wind speed in miles per hour, measured at the spillway.

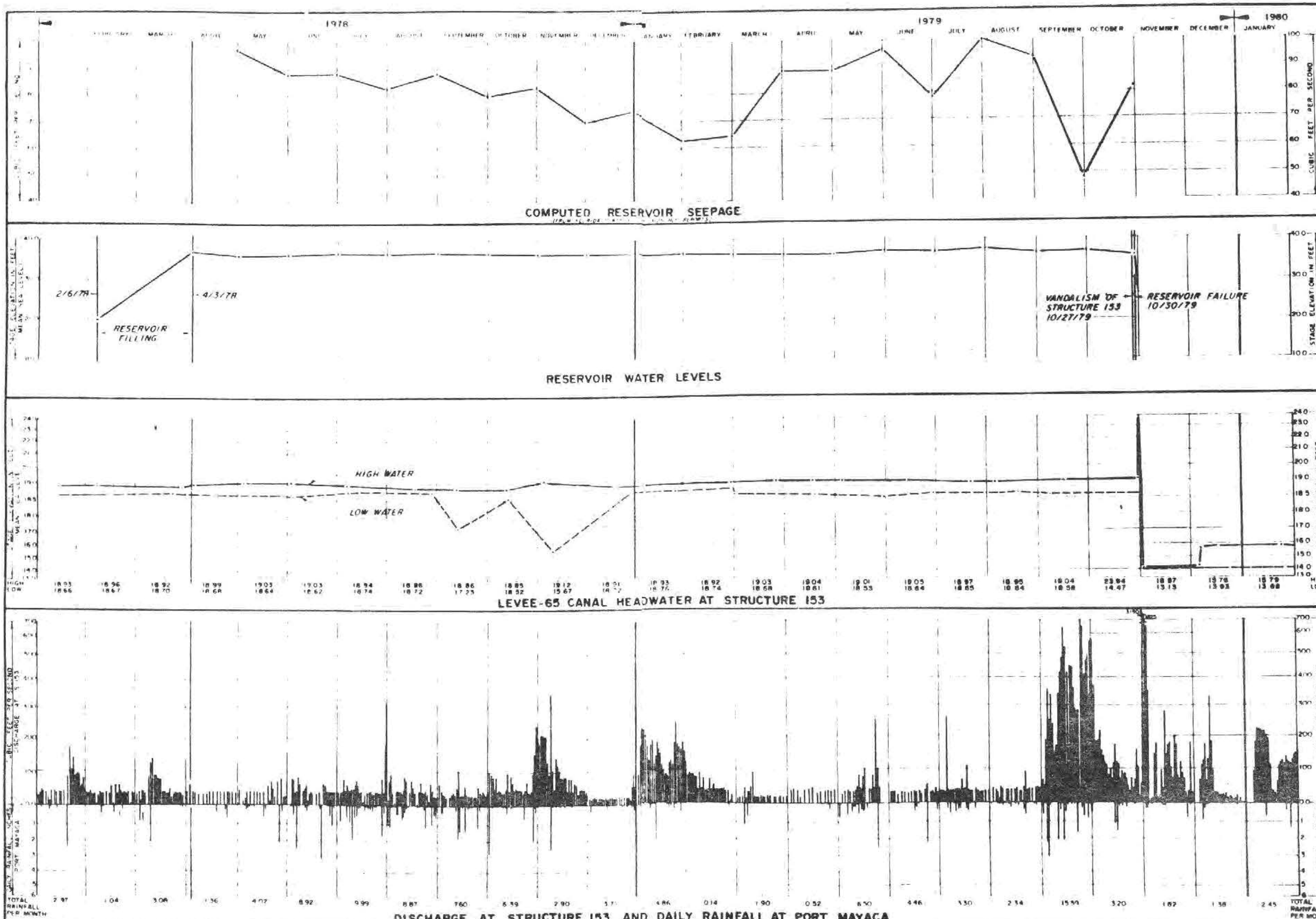
The values of vapor pressure were obtained from a standard tabulation of saturated vapor pressures for the appropriate temperatures.

Seepage losses, calculated from formula (2) are shown in Figures 4-4A+B. Unfortunately, these values are meaningless because the "reasonable" evaporation values obtained from formula (3) are specious. These values range from 0.2 to 0.6 inches per month. Class A pan evaporation, measured by the National Weather Service at Moore Haven Locks, ranges from 4 to over 10 inches per month during the same time period. It must be recognized,




COMPOSITE DATA CHART

Figure 4-4A




COMPOSITE DATA CHART

Figure 4-4B

however, that pan evaporation is only of the same order of magnitude as Lake evaporation.

If the evaporation is calculated as 0.7 times the Moore Haven pan evaporation, the average of the 19 monthly seepage calculations from equation (2) is 37.3 cfs. This value compares quite well with the calculated seepage of 2 cfs per mile of embankment discussed below. Table 4-3 summarized the seepage estimated by this method as follows:

TABLE 4-3
Reservoir Seepage in Cubic Feet Per Second
(estimated from formula (2) and pan evaporation @ Moore Haven)

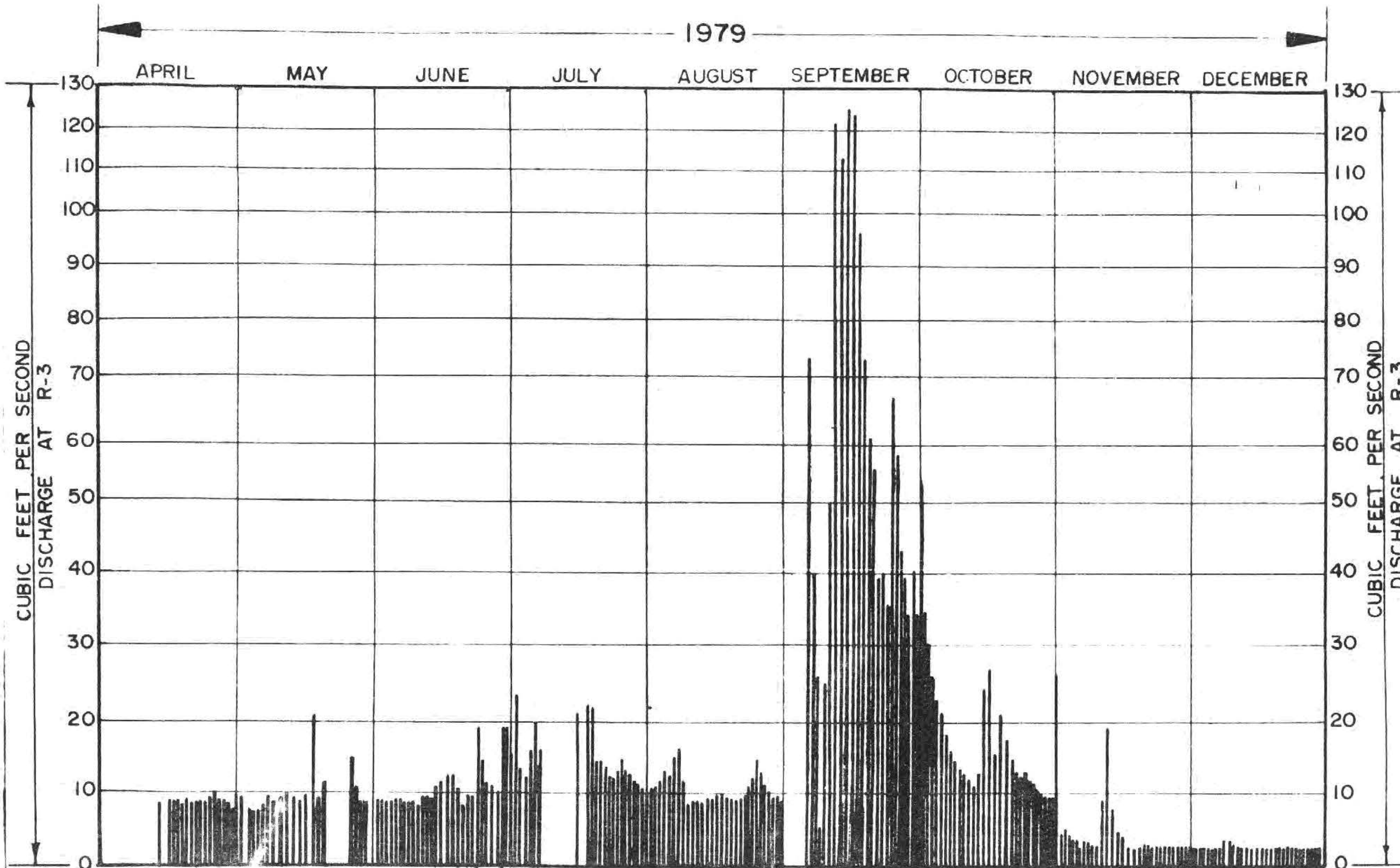
| MONTH | SEEPAGE | MONTH | SEEPAGE |
|---|---|---|---|
| April, 1978 | 46.6 | January 1979 | 36.7 |
| May | 36.6 | February | 37.4 |
| June | 38.6 | March | 45.2 |
| July | 33.9 | April | 35.2 |
| August | 37.2 | May | 32.6 |
| September | 36.9 | June | 14.0** |
| October | 50.7* | July | 43.4 |
| November | 45.4 | August | 38.5 |
| December | 51.4* | September | 6.3+ |
| | | October | 42.2 |

* In these months the calculated evaporation was probably too low.
** In this month the calculated evaporation was probably too high.
\+ In this month the calculated rainfall was probably too low.

The use of pan evaporation at Moore Haven is not being suggested for use in determining evaporation from the reservoir at Indiantown. It is only mentioned as a method of determining the order of magnitude of the seepage. The results of this determination compare favorably with other pieces of evidence, whereas those calculated by FP&L do not.

Direct measurements of seepage described previously were done under contract by Dames & Moore, Consulting Engineers, who calculated discharge from gages R1 and R3. The R1 gage collects runoff from a large area north of the reservoir as well as seepage from the entire north embankment. Because of the diverse source of this runoff, these records are of little value and have not been considered further. The calculated flow at R3 on the other hand is very significant. Unfortunately the drainage area of this gage includes about a two-square mile area between the power plant and SR 710. Consequently, after substantial rainfall, this latter area contributes heavily to the runoff. After sustained drought periods, R3 records the seepage from the reservoir and a small base flow. Figure # 4-5 shows a rather consistent discharge of about 8 cfs between April 13 and May 15, 1979. There was little rainfall recorded between March 8 and May 15. Again, after the embankment failure on October 30, the discharge at R3 rapidly declined to a rather consistent level or a base flow of about 2 cfs from November 15 to December 31, 1979. Rainfall in the antecedent period from October 15 to December 31 was also very light, except for a one day event on November 12. Thus it may be concluded from an analysis of this hydrograph, that the pre-failure flow from the six-mile reach on the east side of the reservoir of 8 cfs was equal to the seepage of 6 cfs plus a base flow of 2 cfs. The runoff from seepage therefore was roughly one cfs per mile. Much of the seepage, however, probably didn't reach the toe ditch, but evaporated along the very wet toe of the embankment. Thus the total seepage might reasonably be 2 cfs per mile.

All of this discussion is somewhat academic because the proposed remedial




DISCHARGE AT R-3

Figure 4-5

measures, discussed below, will provide for a more direct measure of seepage. The only value in reviewing previous seepage estimates is to determine if any abnormal increase in seepage occurred prior to the failure.

Once the reservoir was filled, stages were held to a very constant level. These ranged from elevation 36.41 to 37.24. Make-up pumping was implemented during the time when the reservoir was full. The operation of the pumps averaged about three days per month.

CHAPTER 5

FAILURE INVESTIGATION

## Response to Failure

In response to the embankment failure that occurred in the late evening of Tuesday, October 30, 1979, a number of investigative efforts were set in motion. Aside from the owners and their consultants, designers and contractors, investigative teams from several state and local agencies were formed who have pertinent jurisdiction and/or a compelling interest in the events leading up to and including the impacts of the event and who have appropriate current legal jurisdiction relating to dam safety and protecting the public interest. The central broad objectives common to all parties involved in whole, or in part, were to: (1) determine probable cause(s) of the failure, (2) identify possible triggering mechanisms, (3) examine the entire reservoir relative to assessing its structural integrity to function as planned and designed, (4) determine appropriate repairs to the breached section and associated damaged areas, (5) determine appropriate corrections to all, or such portions of, the reservoir as subsequent investigations show to be necessary or desirable, (6) evaluate and approve a bold, imaginative and technically competent monitoring program designed to accurately apprise the owner, his representatives and state and local interests as to both surface and subsurface hydrologic conditions at any stage of reservoir condition, (7) review this event relative to other reservoirs of all types and uses from the perspective of safety, (8) provide insight into additional legislation or review and possible revision of existing regulations concerning impoundments of all kinds, and finally (9) in its broadest perspective, act to protect the public interest, welfare and safety.

On the basis of the rationale specified, the following investigative committees were formed:

A. Department of Environmental Regulation Committee:

Chuck Littlejohn, Chairman, DER

Anwar Wissa, Consultant to DER

John Garlanger, Consultant to DER

Abby Foster, Consultant to DER

Mr. Reves, Consultant to DER

Jack Goodridge, SFWMD

Patrick Gleason, SFWMD

B. South Florida Water Management District:

Abe Kreitman, Chairman

Richard Slyfield

Ron York

Harry Cedergren, Consultant to SFWMD

William Clevenger, Consultant to SFWMD

C. Martin County:

John Holt, Commissioner

Ed Greenamyer, County Engineer

D. Florida Power and Light Company:

Mid Valley, Inc. - Design Engineer - Resident Engineer

National Soil Service - Geotechnical Consultant

Board of Consultants:

James L. Sherard, Chairman

William V. Conn

Ray K. Linsley

Subsequent to the failure event, a Board of Review consisting of the following members was retained by the owner:

William Swiger, Chairman

John H. Schmertmann

Skip Hendron

Paul Shea

James Mahar - part time geotechnical consultant

Leo Martin - part time geotechnical consultant

In its most general form, the South Florida Water Management District (SFWMD), takes its authority from the Water Resources Act, Chapter 373 FS, Part 4, Management and Storage of Surface Waters, which relates to dams and impoundments and appurtenant works. This part also describes inspections (373.423) and orders relating to remedial measures (373.423(B)) required as a consequence of inspections carried out under 373.423.

In addition, pursuant to an agreement executed on June 8, 1973 between the Water Management District and the Florida Power and Light Company, which relates essentially to the filling of the reservoir and replacement of waters lost to evaporation and seepage, special provisions are described therein requiring approval by the Water Management District of plans and specifications for repairs and construction.

## Foundation Exploration Program

Subsequent to the embankment failure, a Special Board of Consultants as defined above was convened who, in conjunction with the Consultants Review Board, initiated a detailed, indepth comprehensive investigative program the objectives of which were to determine and then quantify the cause(s) of failure and to determine the triggering mechanisms, if any, that set the failure events in motion. Their second objective was to determine and quantify the nature and extent of the repair and rehabilitative procedures necessary to the remainder of the reservoir.

Based on historical precedent in this kind of investigation, there was a general concensus that the exact cause of the failure might never be determined, essentially because all of the "proof" may have been washed away in the flood waters. At best, one could only hope that through careful and detailed investigation, a much more clear and detailed understanding of subsurface conditions could be obtained and that along with other observations would yield one or more "most probable causes". The ability to make such a determination was and is fundamental to the viability of the entire reservoir area because, as a next step, it would be necessary to examine the entire reservoir area in order to determine if any of these conditions exist and to make such modifications to the entire reservoir such that events leading up to a failure from any of these causes could not reoccur. Finally, to install and implement a monitoring network that would accurately display problems that could occur if any of the failure mechanisms began, and to do so in a time frame that would provide the owners maximum reaction time to prevent the reoccurrence of another disastrous and costly failure.

The investigative efforts associated with post failure events were intense, very thorough, and detailed. Briefly described, they included:

- numerous core borings and other exploratory drilling;
- test pits and trenches in the vicinity of the breach;
- the methodical stripping of soil layers within the entire breach and scour area;

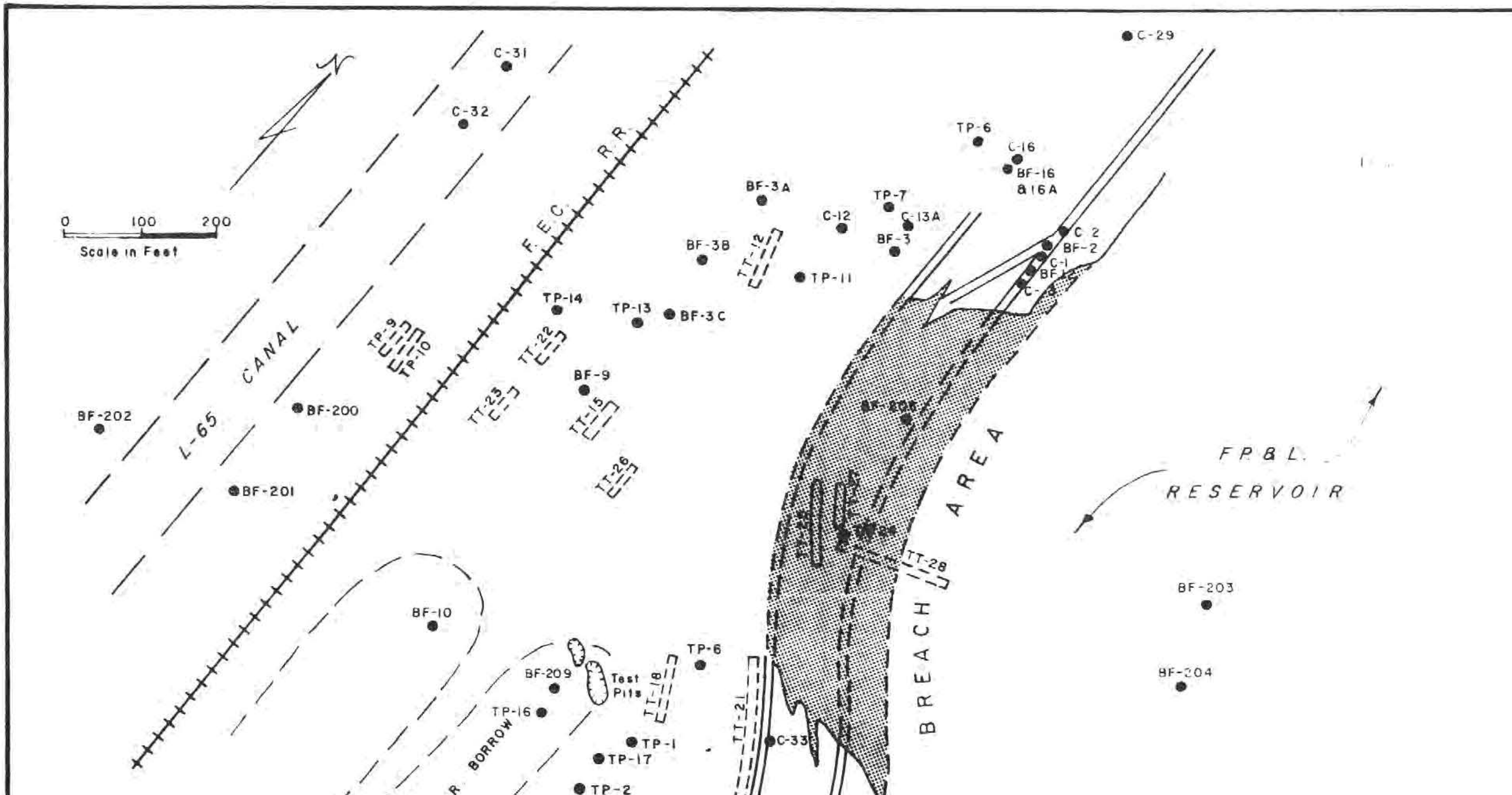
- detailed geologic mapping;
- borings at upstream embankment toe around entire reservoir;
- cone penetrometer tests;
- related quantitative activities such as in-place shear testing and field permeability determinations.

Figure 5-1 shows the location of all of these various activities in the vicinity of the breach.

As an ongoing activity, the results of all of the investigative efforts were studied, collated, and correlated on a daily basis; to a large extent the subsequent exploratory efforts were directed by the results of this ongoing work. This course of action was continued to a logical end point which was reached when a concensus view determined that the data either revealed or did not reveal evidence of relevance or significance to the investigation. During the daily briefings, questions were raised and theories were presented. These thoughts and ideas were than methodically pursued in the field. The final conclusions to this process of field investigative efforts were ultimately distilled down to several probable causes of failure. As assessment of possible and probably causes of failure is elaborated upon elsewhere in this report.

Exploratory Borings in the Breach Area

During the course of the investigation, approximately 18 borings were made. Almost all borings were sampled continuously along their entire depth by various techniques. Four inch diameter soil samples and rock cores, three inch diameter piston and pitcher barrel samples, and standard penetration cores were obtained in order to examine in detail the physical characteristics of the embankment foundation. Logs of these borings are given in




1. CONE SOUNDINGS CF-63 THROUGH CF-90 LOCATED ON THE SOUTH OF BARLEY BARBER SWAMP
2. CONE SOUNDINGS 1U THROUGH 25U AND CF-101 THROUGH CF-109 LOCATED NORTH OF BARLEY BARBER SWAMP. BEGINNING WITH 1U AT THE SOUTH AND ENDING WITH 109 AT THE NORTHERN END OF THE DIKE PARALLEL TO L-65 CANAL

1. CONE SOUNDINGS C-35 THROUGH C-62 LOCATED SOUTH OF BREACH ON TOE ROAD
2. CONE SOUNDINGS CF-300 THROUGH CF-339 LOCATED AT 50' INTERVALS ALONG INSIDE TOE OF EMBANKMENT BEGINNING AT ENG. STA. 416 + 00
3. CF-340 THROUGH CF-367 ARE AT 25' INTERVALS ALONG INSIDE TOE BEGINNING 25' SOUTH OF CF-339

ADAPTED FROM F.P.&L. REPORT

PLOT PLAN – LOCATION OF POST FAILURE SUBSURFACE INVESTIGATION

Figure E-1

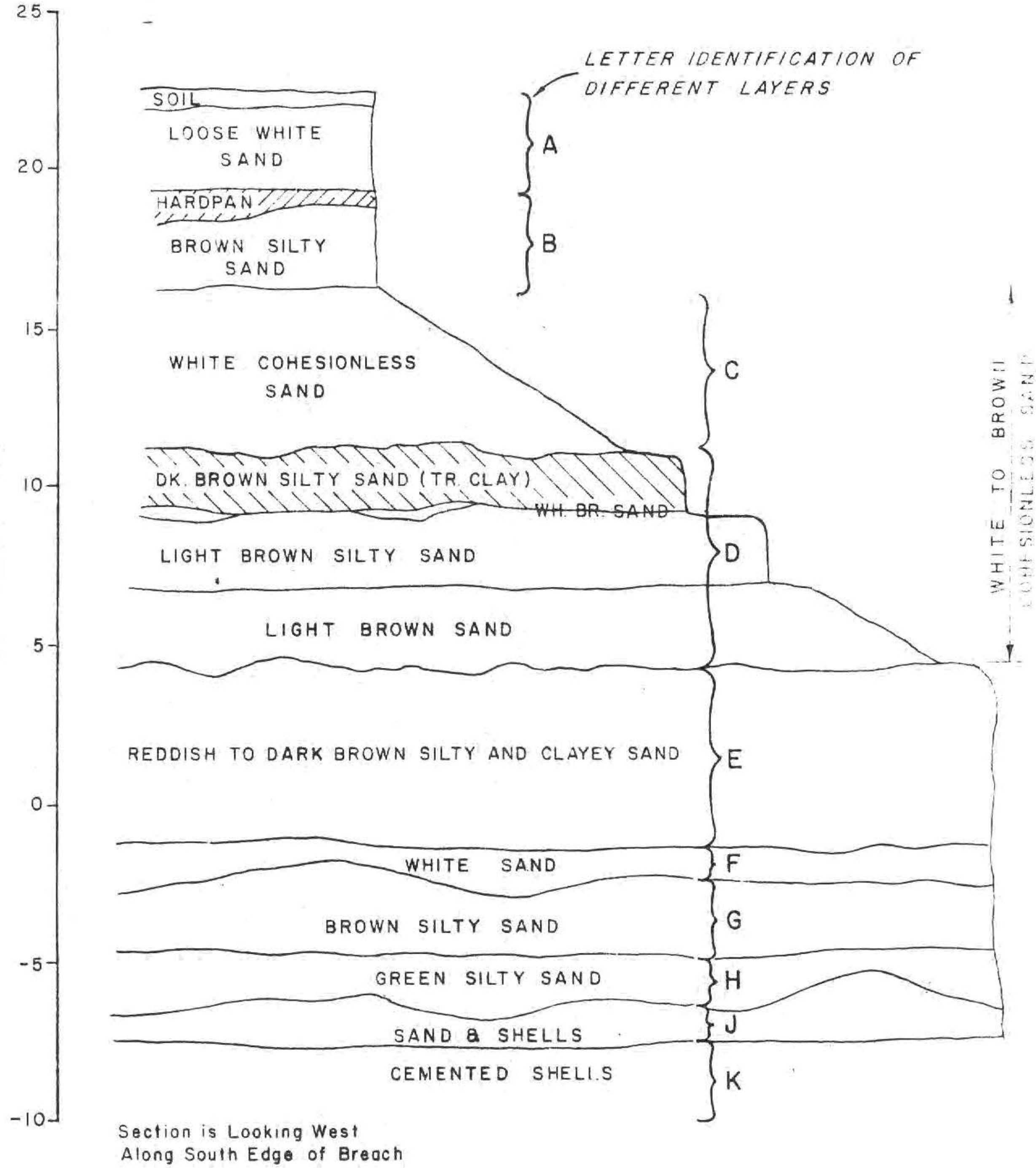
Volume 2 of the Consultants Report and their observations are contained in Volume 1, Section 6.05. A generalized geologic columnar section is shown in Figure 5-2.

As a general statement, these borings confirmed:

1. The very dense nature of the embankment
2. The several soil layers that exhibited very low blow counts, observed during the initial pre-design exploratory program, consisted of red to brown clayey and silty sands and greenish gray silty sands (layers G and H, see Fig. 5-2 ) that immediately overlay the cemented shell beds. These formations extended generally from elevation -3 to -8 throughout the site. Relative to layers G and H, in many of the borings several thin zones, possibly 2-6 inches thick, were noted to be liquified in the sample tubes. In open cut in the breach area these two layers dissected and eroded excessively, caused primarily by release of pore pressure and thence through the mechanism of drainage of water from this very fine grained formation.

   In addition, it was further observed that the formations (at least in open cut) were very prone to the development of pipes.

3. The cemented shell (layers J and K), the top of which is at approximately elevation -5, has a somewhat irregular surface and, in its upper part at least, consists of three units. The top most unit is a weakly cemented to uncemented shell hash




GENERALIZED COLUMNAR SECTION

ADAPTED FROM F.P.& L. REPORT

choked with sand. Cementing is random and variable. This formation consists of shells which are, for the most part, whole and range in size from the smallest size to conch 10 inches in length. It also includes coral heads of a similar size. There are numerous whole bi-valves all of which is indicative of very active, open water, onshore wave action. The middle layer is a very hard, massive, gray black, mud entrained carbonate rock which is in turn underlain by a weakly cemented open fabric shell hash. Examination of these units reveals numerous vertical to high angle fractures and possibly "holes". The significance of these features, viewed in conjunction with the downward gradients shown to exist in the piezometers (D and E) at the downstream toe, lead to the theory that this gradient caused migration of sands and silts from the overlying sediments and into the shell hash leading eventually to settlement, cracking, arching and subsequent failure of the embankment.

An interesting and potentially important sidelight to the exploratory program became evident during the course of our investigations. During or immediately after a boring was made, the samples (or cuttings) were described. These descriptions then became a permanent part of the record which formed part of the basis for subsequent decisions. Assuming that descriptions of the samples were in every case made by competent, trained and experienced professionals, it is interesting to compare the degree of similarity or conflict between descriptions by independent investigators describing the same material. For purposes of illustration, three logs are presented: the first is boring B-139 contained in Volume II to Mid-Valley, Inc. by National Soil Services, Inc. (NSS) dated November, 1973.

This boring is closest to the breach area. The second is Boring BF-1A (NSS) taken as part of the failure investigation and described by National Soil Service. This same boring was described by Water Management District hydrogeologists and captioned BF-1A (WMD). This comparison is presented without further comment in Figure 5-3 except to note the differences that appear when several people examine and attempt to describe a particular static item.

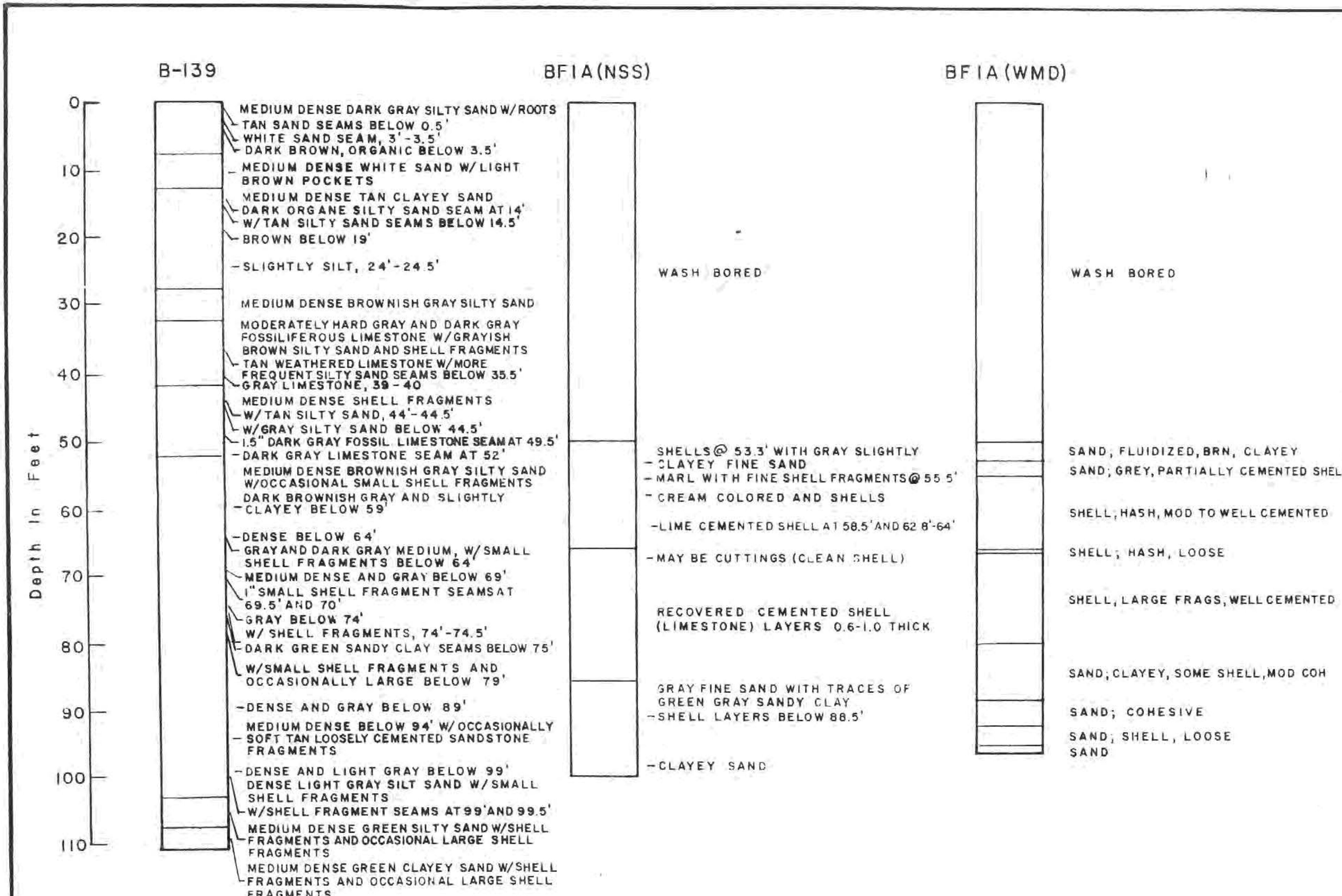
Inspection Trenches and Test Pits

In addition to exploratory borings, approximately 28 test pits and trenches were excavated. The objectives in making these excavations include a) a visual examination of the sediment, its attitude and structure; b) examination for evidence of deformation, structural "defects" or damage, and in general to determine if remnants of the causative agents of the failure were visible; and c) to examine a large exposure of the several formations which provide a far superior view of these sediments than cores taken through them.

The location of the various pits and trenches are shown on Figure 5-1. Basically, they can be separated into three general groups.

1. The area in the vicinity of the downstream side of the southerly breach area.
2. The area in the vicinity of the downstream side of the north breach area and along the scour wall.
3. Test trenches in the scour area designed to view in detail the beds of shell hash which formed the floor of the scour area.

The results of these examinations are described in detail in the FP&L Consultants Report (Volume L, pg. 6-6 et seq). The rationale relating to the placement of the various pits and trenches is also presented. We



DESCRIPTIVE COMPARISON OF LOGS

Figure 5-3

would therefore consider it to be redundant to redescribe these events. The observations referenced were, in our judgment, accurate, detailed and comprehensive. In summary review, these efforts revealed that at the south breach area, between the toe and railroad borrow:

1. No evidence of piping or tunneling was observed.
2. The hardpan layer effectively impeded the flow path of water through it.
3. The original depth of the old railroad borrow was spotted at about 10 feet below existing grade (elev. 8 to 10).

The relative position of the various layers can be seen in Figure 5-2 .

In the downstream area of the north breach:

1. The hardpan layer was not uniformly present throughout the area of investigation.

Test pits and trenches located in the breach area revealed that:

1. The hardpan was present in the area downstream of the main scour zone (between L-65 and the railroad).
2. In those areas not washed away as a result of the failure, the "normal" sequence of sediments were identified.
3. The shell rock was present everywhere and uniformly exhibited three members. The top, a weakly and randomly cemented shell hash, completely choked with sand; a gray-black, dense, massive, very hard limestone overlying a relatively clean, weakly cemented, shell hash. Numerous vertical or high angle fractures penetrating all three members were observed.

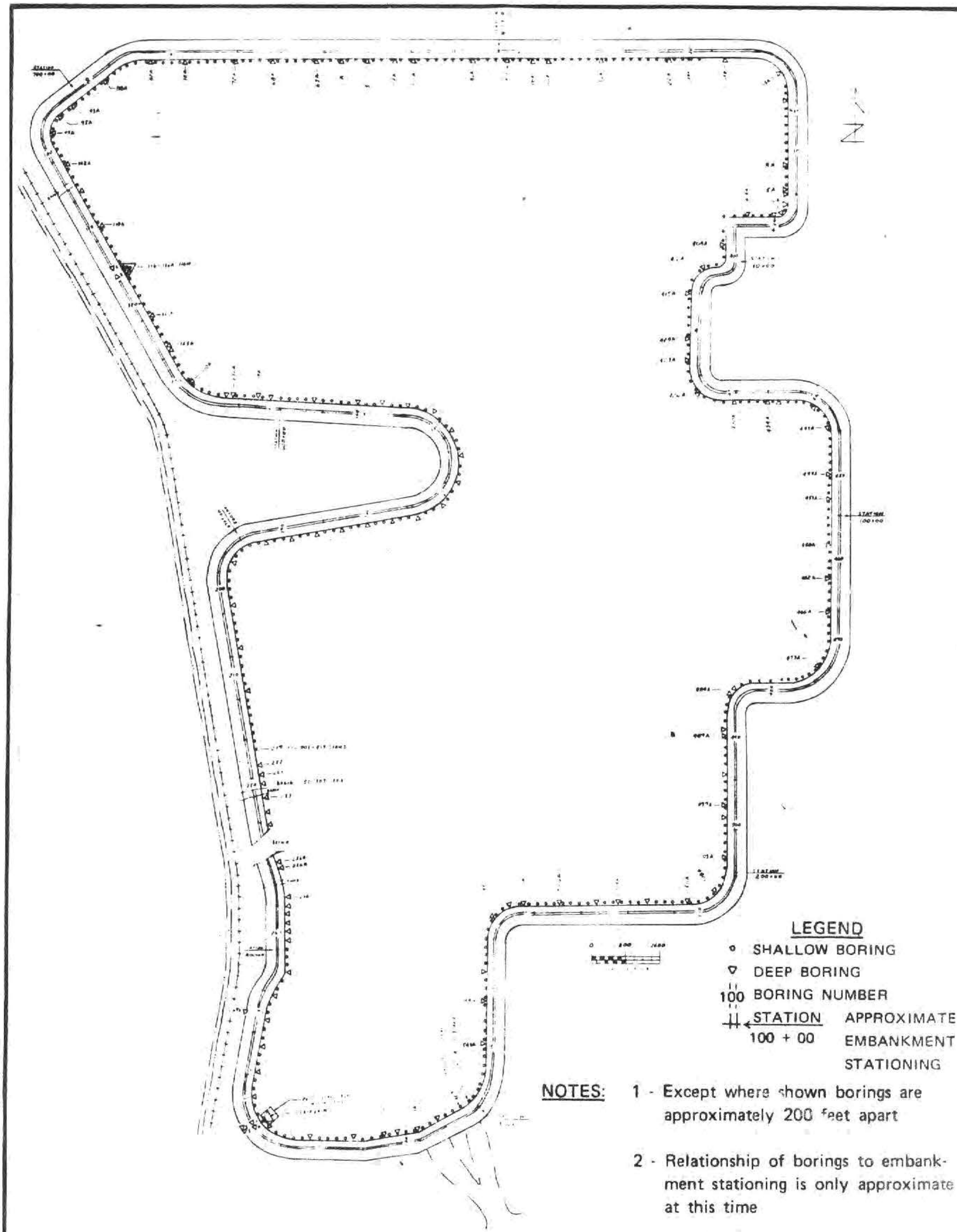
### Geologic Mapping

Another major exploratory effort was concentrated in the breach upstream of the area of major scour. Here all remaining layers of undisturbed sediments were methodically stripped, examined and mapped. This large effort extended across the entire breach area and was carried well below the top of the shell rock. An excellent series of geologic cross-sections which incorporate all previously described borings, test pits, and trenches, as well as the stripping operation, has been summarized in a comprehensive series of geologic profiles and is contained in the FP&L Consultants Report (Vol. 1, Figures 6-3 through 6-18).

Notes and observations made by District geologists were compiled and compared with profiles cited above. There is no area of relevant disagreement. A detailed study, during both field excavations and a careful analysis of profiles developed, failed to uncover any remnant evidence as to the nature and mechanism of failure.

### Borings at Upstream Embankment Toe

About 500 borings were made at the upstream embankment toe around the entire reservoir. This number equates to a spacing of about 200 feet. See Figure 5-3A for the location of these borings. The original purpose of these borings was to identify the depth to the so-called hardpan layer (Layer B). Consequently about 80% of these borings were 8 to 12 feet deep. The remainder were about 30 feet deep. Each hole was sampled continuously and a continuous record of blow counts was also retained. Any unusual conditions, such as loss of drilling fluid were also noted. The samples were described in the field, and retained for more careful subsequent analysis. A detailed plot of all the findings of the explorations was made and correlations between the drill holes




BORING LOCATION PLAN F.P.&L. MARTIN RESERVOIR

S&W 13550 AS OF 4-12-80

were made, in so far as possible. Correlations were very difficult to make, however, because of the rapid facies changes, except for a few marker beds.

Probably the most startling result of this program was the identification of two unique areas, one at the southwest corner of the reservoir, the other on the west side of the reservoir about a mile north of the Barley Barber Swamp. In each of these locations the drill rod dropped about two feet under its own weight and simultaneously all the drilling mud was lost, at a depth of about 15 feet, or an elevation of about +6 feet. The drillers conclusions was that a void had been encountered. An alternate conclusion is that a very weak and very pervious zone had been encountered. A detailed program was subsequently conducted at the southwest corner, described under testing below (page 5-20).

### Testing

The post-failure exploration investigation included a great deal of soil testing both in the laboratory and in-situ. Most of this testing was in the breach area, but considerable testing was performed on foundation soils along other portions of the dike alignment. The various tests fall into several categories as shown in Table 5-1. A listing of each type of testing is summarized below:

TABLE 5-1

Post-Failure Soil Testing

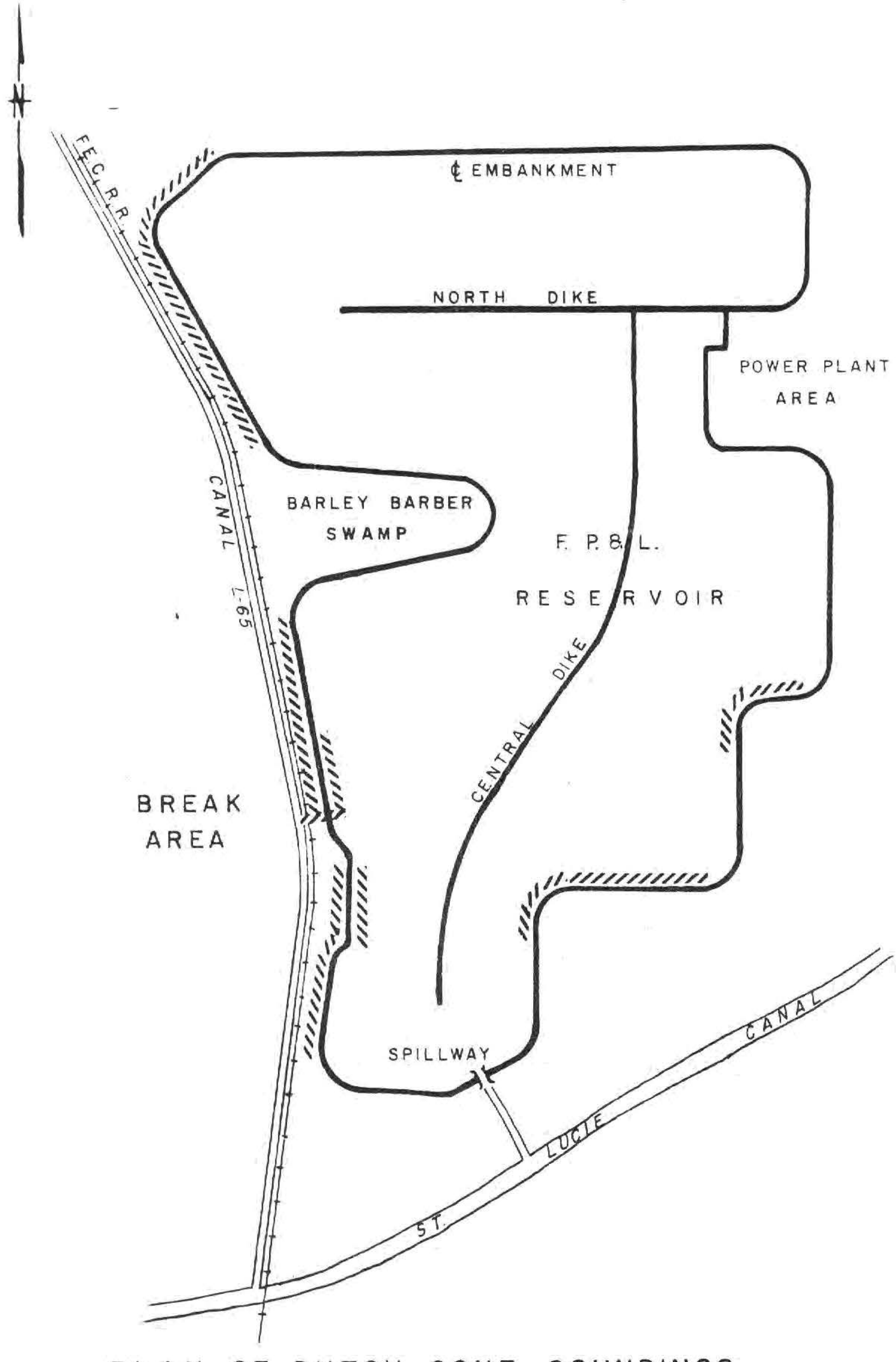
| Type of Testing | Lab or Field | Location | No. of Sample Locations | No. of Depths per Sample Location |
|---|---|---|---|---|
| 1. Cone Penetrometer | | | | |
| Hand Cone Penetrometer | Field | Breach Area | 23 | * |
| Dutch Cone Penetrometer | Field | Breach Area | 14 | * |
| Dutch Cone Penetrometer | Field | Breach to Barley Barber | 28 | * |
| Dutch Cone Penetrometer | Field | North of Barley Barber | 34 | * |
| Dutch Cone Penetrometer | Field | South of Barley Barber | 167 | * |
| 2. Shear Strength Iowa Borehole | Field | Breach Area | 2 | 2 to 4 |
| 3. Density | | | | |
| In Place Density | Field | Breach Area | 6 | 2 to 4 |
| Relative Density | Field | Breach Area | 4 | 2 to 4 |
| Natural Moisture Content | Field | Breach Area | 6 | 1 to 4 |
| Compaction | Lab. | Breach Area | 2 | 1 to 3 |
| 4. Permeability | Field | Breach Area | 1 | 1 |
| 5. Classification | | | | |
| Gradation | Lab. | Breach Area | 4 | 3 to 5 |
| Atterburg Limits | Lab. | Breach Area | 3 | 3 to 5 |
| 6. Standard Penetration Resistance | Field | Around entire reservoir | 500$\pm$ | * |
| 7. Special Grout Test | Field | SW corner of reservoir | 1 | * |

* Numerous, essentially continuous.

Cone Penetrometer (Dutch type) soundings as viewed by District staff were evaluated qualitatively. These soundings gave a measure of the resistance to penetration of a fixed, dimensioned probe as it penetrated the various soil layers. As such, it was possible to develop some insight into the stability of those layers, particularly when viewed in terms of suspect surface features such as boils and shallow depressions which might be interpreted as some form of subsurface movement or deformation related to the reservoir failure, with the potential that other areas of the reservoir site were being similarly affected. As a consequence of this logic process, cone sounding surveys were conducted at closely spaced intervals in the following areas:

1. through the embankment north of the breach.
2. between the embankment and the L-65 canal.
3. along the downstream toe road both north and south of the breach.
4. along the upstream toe of slope south of the breach.
5. two areas north of the pump station.

The general location of these investigations is shown on Figure 5-4.




PLAN OF DUTCH CONE SOUNDINGS

In place shear testing was performed with an Iowa Bore Hole Shear device. This equipment consists of a split cylinder, the sides of which are pressed against the sides of a predrilled hole with a predetermined force. A gradually increasing vertical force is then applied until a shear failure occurs. The device attempts to measure the in-situ soil strength. In principle it is a direct shear device in that it forces a failure on a plane normal to the confining load, and the result is probably a consolidated drained (CD) test.

This testing was performed in the very weak, green silty sand (layer H) and in the brown silty sand (layer G) because of concern generated by the cone penetrometer and blow count analyses. The testing was limited to two drill holes located in the center of the breach area, one near what had been the downstream toe of the embankment, the other on the embankment centerline. This testing was plagued with equipment limitations, and the results are somewhat inconclusive so they should be viewed with some caution. The results are summarized in Table 5-2 as follows:

TABLE 5-2

Soil Strength from Iowa Bore Hole Testing

| Test # | Layer | Location of Drill Hole | Angle of Internal Friction (degrees) | Cohesion (psi) | Tests at Each Depth |
|---|---|---|---|---|---|
| 1 | G | Toe of dam | 42 | 1 | 6 |
| 2 | H | Toe of dam | 22 | 1 | 2 |
| 3 | H | Toe of dam | 13 | 1 | 3 |
| 4 | H | Toe of dam | 9 | 1 | 4 |
| 5 | H | Centerline of dam | 29 | 6 | 3 |
| 6 | H | Centerline of dam | 23* | 12 | 5 |
| 7 | H | Centerline of dam | 29* | 12 | 3 |
| 8 | H | Centerline of dam | 33* | 12 | 1 |
| | H | Average of 2 thru 5 and 7 | 20 | 4 | - |

* Tested at same depth, results varied with consolidation time and rate of shearing.

Several types of density and moisture content tests were made on several strata in the breach area. These are summarized in Table 5-3as follows:

TABLE 5-3

Density Tests

| Sample # | Stratum | Location | Elev. | In Place | | | Compacted | |
|---|---|---|---|---|---|---|---|---|
| | | | | Density (#/cf) | Moisture Content (%) | Relative Density (%) | Max. Density (#/cf) | Optimum Moisture Content (%) |
| 1 | A | Sta 420;120' Upstream | 22-23 | 92 | - | 67 | 101 | 11 |
| 2 | A | Sta 420;120' Upstream | 21-22 | 97 | - | 83 | 100 | 11 |
| 3 | A | Sta 420;120' Upstream | 20-21 | 102 | - | 66 | 101 | 11.5 |
| 4 | B | Sta 420;120' Upstream | 19-20 | 102 | - | - | - | - |
| 5 | E | BF-16 | 0 | 112 | 20 | - | 119 | 10 |
| 6 | E | BF-16 | -3 | 108 | 21 | 94 | 110 | 11 |
| 7 | E | BF-16 | -5 | 100 | 27 | - | - | - |
| 8 | H | Breach Excavation N.Wall | -6 | - | 23 | - | - | - |
| 9 | H | Breach Excavation S.Wall | -5.5 | 101.4 | 24 | 96 | - | - |
| 10 | H | Breach Excavation S.Wall | -5 | 104.7 | 21 | >100 | - | - |
| 11 | J | TT 28 | -5 | 106 | 20 | * | - | - |
| 12 | J | TT 28 | -6 | 103 | 18 | * | - | - |
| 13 | J | TT 28 | -5 | 91 | 22 | * | - | - |
| 14 | J | TT 29 | -6 | 104 | 18 | * | - | - |
| 15 | J | TT 29 | -8 | 95 | 18 | * | - | - |
| 16 | J | TT 29 | -8 | 98 | 21 | * | - | - |

* Relative density could not be determined because soil is a mixture of materials of different specific gravity.

The relative density tabulated in column 7 of the preceding table was determined as follows:

$$\text{Relative density} = \frac{e\ max - e}{e\ max - e\ min} \qquad (4)$$

where,

$e$ = void ratio

The following very interesting conclusions may be drawn from Table .

First, density tests were performed on only two samples from stratum H, the green silty sand, which in the pre-construction (blow count ) and in the post-failure (Dutch Cone Penetrometer) tests was identified as very loose, with very low strength. The density tests, though very limited in mumber, indicate that this stratum has been densified, probably by the load imposed by the dike during the approximate 5 year period since construction of the dike began.

Second, it can be noted that the density of all of the foundation materials beneath the embankment was close to 100 pounds per cubic foot, as is that of the compacted fill from the surface material of which the embankment is composed.

Consequently, the strength used in stability analyses can be based on consolidated shear strength tests and danger of liquefaction of this material is minimal.

During the post-failure exploration, a unique opportunity existed to measure directly the permeability of the open sh ll stratum (L) below the cemented shell (K). Two methods were employed by the FP&L Consultants in this determination which gave somewhat different results.

Both methods employed the same field set-up, but calculated the permeability by different methods. The field set-up consisted of two parallel trenches about 25 feet apart cut through the cemented shell and the open shell strata. Seepage entered the bottoms of both trenches since the water levels in both were at about elevation -10 and the static water level was at about elevation +20. After determining this background inflow rate, a water level differential was established and the increased flow rate into the lower trench determined. During the time of this water level differential, dye was introduced into the higher trench, and the travel time to the lower trench was measured.

In the first method, the permeability was calculated by the following formula:

$$k = \frac{v' \, n}{i} \qquad (5)$$

where,

$k$ = permeability in ft/min.

$i$ = hydraulic gradient

$v'$ = actual velocity in feet per minute through the voids in the soil as contrasted with v which is the apparent velocity through the entire soil and void section.

$n$ = porosity = $\frac{V_{voids}}{V_{total}}$

$V_v$ = volume of voids/cubic foot soil

$V_t$ = total volume/cubic foot soil or 1.0.

In the field test, the time for the dye to pass the 25 feet between the trenches was 51 minutes, therefore $v' = 25/51 = 0.49$ feet per minute. The difference in water surface elevations between the trenches was 3.8 feet, thus $i = 3.8/25 = 0.152$ ft/ft. The porosity was assumed to be

0.35. Consequently $k = 0.49\ (.35) \div 0.152 = 1.1$ ft/minute. This result gave the permeability of the shortest path by which the dye could pass between the trenches.

The second permeability determination was based on a flow net analysis through the entire stratum, the thickness of which was unknown. Generally the horizontal and vertical permeabilities are different, so the flow nets were drawn on transformed sections, and the permeability was determined from the formula:

$$\Delta q = K \frac{N_F}{N_H} \Delta H \qquad (6)$$

where,

- $\Delta q$ = increased flow in cfm per foot of trench caused by the head differential between the parallel trenches, which was measured as 6.4
- $\Delta H$ = head differential between the parallel trenches.
- $N_F$ = number of flow lines in the flow net of the transformed section.
- $N_H$ = number of equi-potential lines in the flow net of the transformed section.
- $K$ = permeability in feet per minute of the transformed section.

The total increase in discharge ($\Delta Q$) is equal to the $\Delta q$ value, as determined above, times the length of the trench which causes the increase. The flow net used in the analysis, however was constructed on a two dimensional, vertical section, whereas the actual flow is three dimensional. Consequently the total flow increase must be $\Delta Q$ times an adjustment facto. This adjustment factor is obtained from a horizontal flow net, and is equal to the total number of flow lines divided by the number of flow lines which will pass on the shortest path route straight between the trenches. (See Figure 5-5) Thus by Formula (6) this factor is 13 ÷ 6.5 = 2, and,

$$\bar{K} = \frac{6.4}{3.8\ (2)\ (45)} \frac{N_H}{N_F} = 0.0187 \frac{N_H}{N_F}$$

As noted above, this permeability is for the transformed section; the true horizontal and vertical permeabilities are as follows:

$$K_H = \bar{K} \left(\frac{K_H}{K_V}\right)^{1/2} \text{ and} \tag{7}$$

$$K_V = \bar{K} \left(\frac{K_V}{K_H}\right)^{1/2} \tag{8}$$

Since the actual thickness of the open shell stratum and the value of $K_H/K_V$ were not known, flow nets in a vertical plane for various combinations of these values were drawn. Figure 5-5is illustrative of these flow nets. The results of the permeability determined by these analyses are given in Table 5-4 as follows:

TABLE 5-4

Flow Net Permeability Determination in Open Shell Stratum

| $K_H/K_V$ | D (ft.) | $K_H$ (ft. per min.) | $K_V$ (ft/min) |
|---|---|---|---|
| 4 | 5 | 0.10 | 0.025 |
| 4 | 10 | 0.06 | 0.015 |
| 1 | 10 | 0.06 | 0.06 |
| 25 | 5 | 0.13 | 0.005 |
| 25 | 2.5 | 0.23 | 0.009 |

Several observations can be drawn from this analysis. First, the assumed ratio $K_H/K_V$ has only a minor effect on the permeability. More significantly, the permeability determined from the flow net analysis is much less than that determined from the dye measurement. Such a result is not surprising since the path taken by the dye was the most direct one and, as such, would yield a higher calcuated permeability. The dye was first observed in the receiving trench opposite one end of the source trench. No record was given of the arrival time in the remainder of the trench. Longer travel times (over longer flow paths) would have resulted in lower values of v' and lower values of permeability. In conclusion, the 0.23 feet per minute value of horizontal permeability by the flow net analysis appears most valid. The significance of the permeability of this strata is related to one of the failure modes in the following discussion.

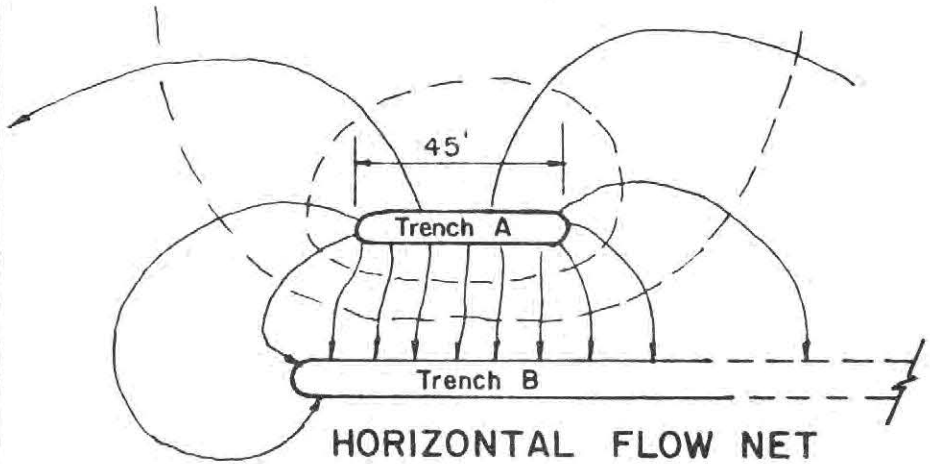
A special grout program was performed in the southwest corner of the reservoir where the "void" was discovered. This special grout test consisted of two phases. The first phase was the addition of 8 to 10 holes along the upstream toe, on the embankment crest and on the downstream toe in an attempt to further define the "void". This effort demonstrated that it was very difficult to correlate any feature, over as short a distance as 25 feet. Some of the holes resembled the first one, some did not.

The second phase of the program was to drill a grid of 30 holes, each about 30 feet deep, on a 6 foot by 8 foot pattern, inject a red dyed grout into each hole, under a gravity head. After waiting a few days for the grout to set, the grouted area would be surrounded with well points and dewatered. If the dewatering was successful, the entire mass would be exhumed and inspected. If the dewatering proved unsuccessful, the project would be abandoned. To date (April 16) only the dye injection has been completed.

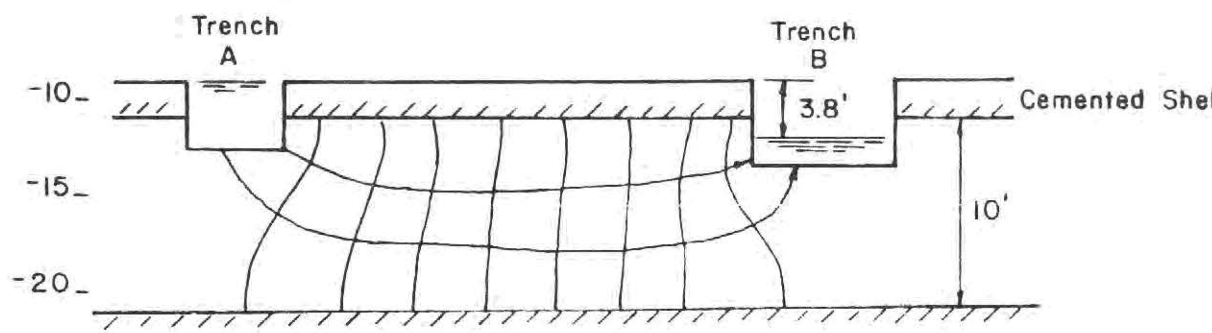
Investigation of Railroad Borrow Pits

Aerial photos taken prior to reservoir construction show the proximity of the railroad borrow pits to the breach (see Figure 5-6). Consequently, it was deemed important to explore several questions concerning these features. First, how deep were they; and second, how close were they to the downstream toe of the slope in the breach area?

During the failure event, a large quantity of sand was deposited in a fan shape to the north, west, and south of the breach, filling and obliterating the north end of these borrow pits; hence, it was impossible to determine their pre-failure outline (see Figures 1-5 and 5-7). The FP&L contract drawings contained aerial photos which appeared to show the relationship between the borrow pits and the embankment alignment, but earlier photos suggest that the borrow pits were of somewhat larger




HORIZONTAL FLOW NET




TYPICAL VERTICAL FLOW NET

FIELD PERMEABILITY DETERMINATION
TYPICAL FLOW NETS BETWEEN TRENCHES

FROM FLORIDA POWER & LIGHT CO.
REPORT ON BREACH OF EMBANKMENT
MARTIN PLANT COOLING RESERVOIR
VOL 2

Figure 5-5

Several observations can be drawn from this analysis. First, the assumed ratio $K_H/K_V$ has only a minor effect on the permeability. More significantly, the permeability determined from the flow net analysis is much less than that determined from the dye measurement. Such a result is not surprising since the path taken by the dye was the most direct one and, as such, would yield a higher calculated permeability. The dye was first observed in the receiving trench opposite one end of the source trench. No record was given of the arrival time in the remainder of the trench. Longer travel times (over longer flow paths) would have resulted in lower values of v' and lower values of permeability. In conclusion, the 0.23 feet per minute value of horizontal permeability by the flow net analysis appears most valid. The significance of the permeability of this strata is related to one of the failure modes in the following discussion.

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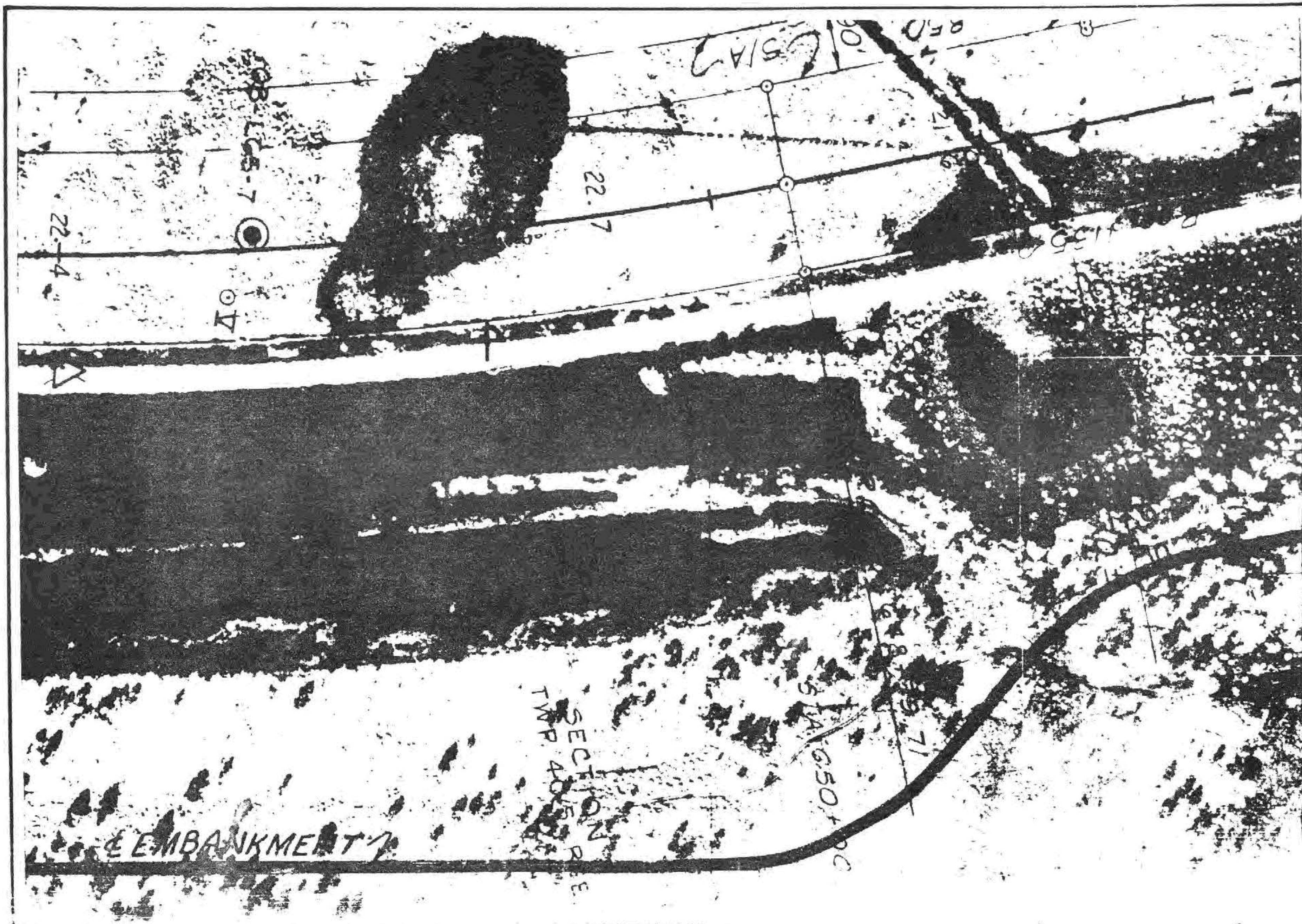



Figure 5-6


10-31-79
FPL
COOLIN
WATER
RESERVOI
FEC
RAILROAD
L-65 BORROW CANAL
DIKE
L-65
LEVEE
L-65
EASTERN
LEVEE
FPL
FEC
RAILROAD

Figure 5-7

areal extent. The discrepancy appears to be due to the fact that floating aquatics covered the northern end of the pits in recent years, and later photos gave misleading indications of their northern limits. An outline from one of the older photos (ca. 1966) was drawn as an overlay on Figure 1-5. This exercise indicates the closest point in the borrow pit is about 145 feet from the downstream toe of the dike.

To determine the depth of the easterly pit, three methods were employed. The pit was dewatered and visually inspected; cone penetrometric measurements were made in the pit area 240 feet south of the breach; and a trench was cut through the north end of the pit. These methods confirmed that the bottom of the pit was in the dark brown silty sand (Layer $D_1$), probably the top of this layer, at an elevation of about 13 feet at the north end; possibly deeper to the south.

The FP&L investigating team believed that these borrow pits extended even further to the north into the breach, and that the bottom of the original pit would have been even closer to the dike than indicated above.

Related Phenomena

Shortly after the embankment failure, the SFWMD evaluated several possibilities that had received some degree of initial, rather widespread notoriety. In the following discussion these "unnatural" causes are singled out for discussion separate from an assessment of probable structural causes for the failure which appear elsewhere in this report.

Over a period of years up to the time just prior to the failure event, there had been a series of labor disputes at the project site with the usual walkouts and slowdowns. Conceivably, deliberate destruction by by involved parties could have been a possibility and thus was considered by many to be a distinct possibility. Sabotage by explosives was ruled out for several reasons: 1) no trace of wire or related apparatus was found in the area; 2) no sounds of explosion were heard by custodial employees or others on the day the failure occurred, and 3) with heavy traffic late into the day on the perimeter road at the site, no drilling or related activity necessary to place charges was possible.

On October 27th, less than a week prior to the event, District Control Structure 153 had purposefully been kept open by vandals who tampered with the electrical controls, as reported in detail elsewhere. By this act borrow canal L-65 dropped abruptly in stage from elevation 19.2 feet to 14.1 feet. At that time it was assumed this was the work of fishermen who frequently tamper with District facilities to gain the advantage of fish accumulation. Therefore it was considered that this might have been a deliberate attempt to lower the water in borrow canal L-65. As explained previously, the water level between the FP&L dike and the L-65 east levee is controlled by culverts with flashboard risers in the east levee. Had the flashboards been removed during the time when L-65 was low, the railroad borrow pits might have been drained and the hydraulic gradient through the FP&L dike been increased significantly. See Figure 1-3. An additional key to such an event would have been the position, or in fact the complete removal, of flashboards in the culvert riser system.

In view of the above, a search was conducted to locate the riser which had obviously been washed out when the flood water broke through the railroad bed and maintenance berm separating the two bodies. By use of survey triangulation and metal detectors, both sections of the double barreled culvert risers Number CP-9 were located five feet beneath the deposited embankment material (then completely filling the L-65 borrow canal).

Upon inspection of both risers it was determined that they were at elevation 18.5 feet and the welded brackets locking the boards in place showed no signs of tampering. One bracket was still in place and the other, although broken loose, showed,through very close inspection, no signs of deliberate removal. The conclusion here is that the borrow pit water could not have been lowered below elevation 18.5 feet through vandalism. Further evidence against a lowering of the water level in the borrow pits is contained elsewhere in this report. Therefore, this possibility was eliminated.

Because of the very unstable nature of some of the strata beneath the dike, the possibility of liquefaction had to be given a high priority. The mechanism of liquefaction is explained elsewhere in the report. Briefly, it is a process by which the superimposed load on a very loose, sandy soil is transferred from the soil skeleton to the pore water. Since water cannot support a shear force, the shear strength is suddenly reduced to zero as the soil particles rearrange themselves to a more compact configuration. Liquefaction requires some sort of a triggering mechanism. A strong vibration or a sudden pressure change could act as such a trigger. The most probable such events would be an earthquake, an explosion, a mechanically induced vibration, such as could be created by a passing train, or a sudden drop in the water table.

In addition to the above investigation, some effort was expended in determining if any seismic activity was noted in the vicinity. Although no independent verification was made by this committee, it was reported at the first meeting of the FP&L Consulting board of November 30, 1979, that they had looked into the possibility of seismic activity. Mr. Swiger, the moderator, stated they could find no records of shocks such as earthquakes or quarry blasts on the night of October 30.

As suggested above, the passage of a railroad train might provide the trigger mechanism. Though this is theoretically possible, were it to have occurred, it would have failed with the passage of the first train after the reservoir was filled, some 18 months prior to the actual failure. If vibration from the train had caused rearrangement of the loose soil particles the passage of each train would make the sand more dense and less subject to liquefaction.

A third consideration was given to a cause of embankment deterioration commonly experienced in other parts of the country. Burrowing into the downstream face of the embankment by animals may have been of sufficient magnitude to precipitate piping and subsequent collapse. Inasmuch as none of the many inspection reports made during the term of operation ever reported this condition, nor did post-failure inspection by this committee indicate any such holes in other areas, this possibility is completely discounted

In view of the above it was concluded that no deliberate or otherwise "unnatural" phenomena could have caused the failure.

Aerial Photography

The use of photography from preconstruction and subsequent periods has been an invaluable aid in conducting this investigation as well as in providing a monitoring tool for post-failure activity by FP&L.

Sources of very valuable aerial photography include:

1. Sequential black and white prints of the L-65 borrow canal alignment route furnished originally to the District by the U.S. Army Corps of Engineers. These included the subsequent reservoir failure area.
2. Aerial maps from this same source that were originally used in construction documents, and which were printed to accurate scale and included significant survey reference for use in accurately locating desired features. Figure 5-6 .
3. A large number of black and white, as well as colored photos, taken at all stages of construction (access to which was provided by FP&L). These were very valuable in showing features subsequently obliterated by construction or water. Figure 5-8.
4. A large infrared aerial mosaic of the entire project site and contiguous area provided by FP&L. This, by its unique color tones, identified such things as relative moisture, density of vegetation, and disturbed soil otherwise less obvious in black and white photography. Figure 5-9 .
5. Photos taken from the District helicopter within twenty-four hours after failure which were used in gaining additional insight into location and direction of flow through and downstream of the breach area. See Figure 5-10.
6. Photos taken by special flights from District aircraft which enabled this committee to monitor progress of the initial stages of breach area repair. See Figure 5-11 .

In addition to close up ground photography, these photos have been a major factor in accomplishing an otherwise insurmountable task of

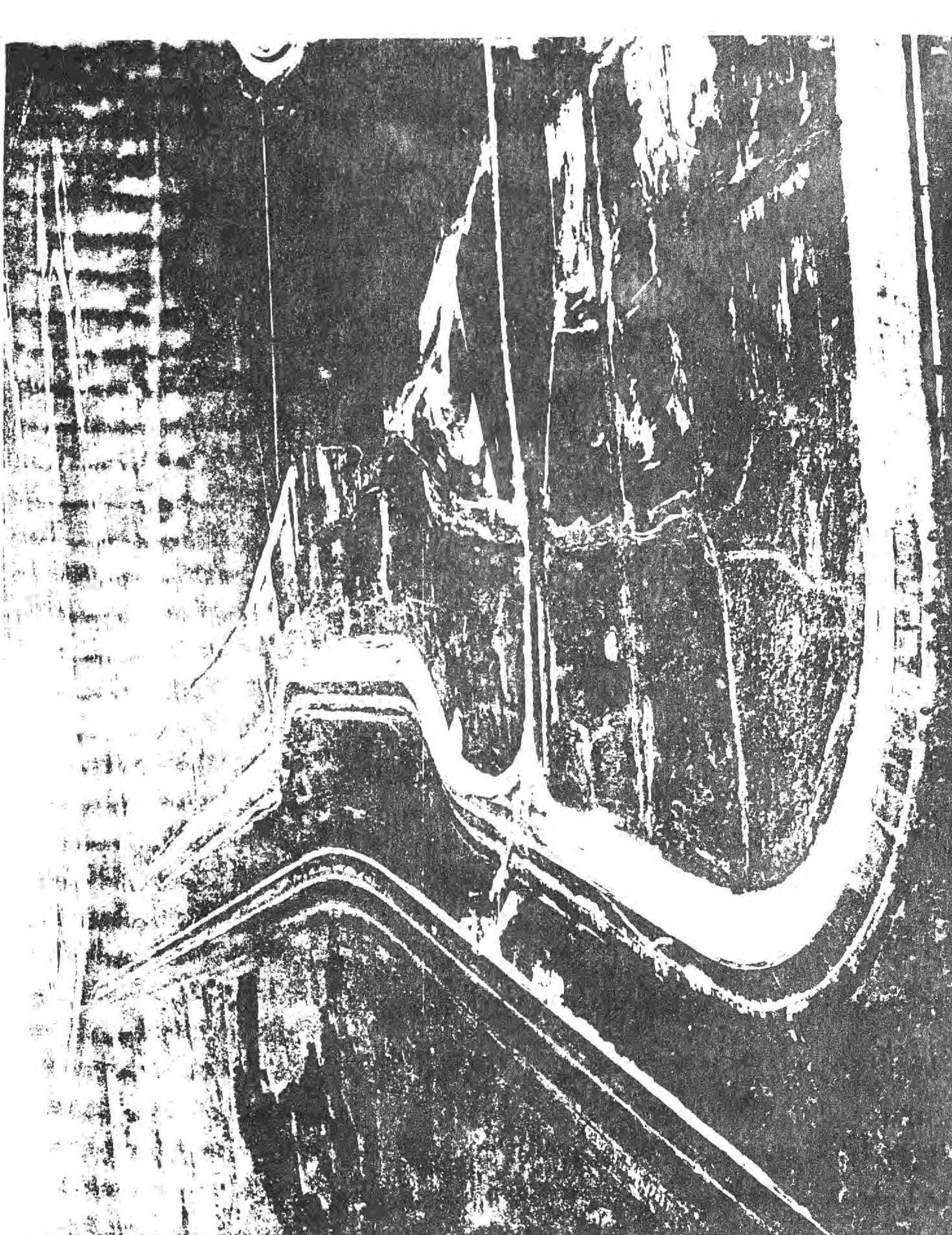


Figure 5-8

Jack Stock photo - give credit!

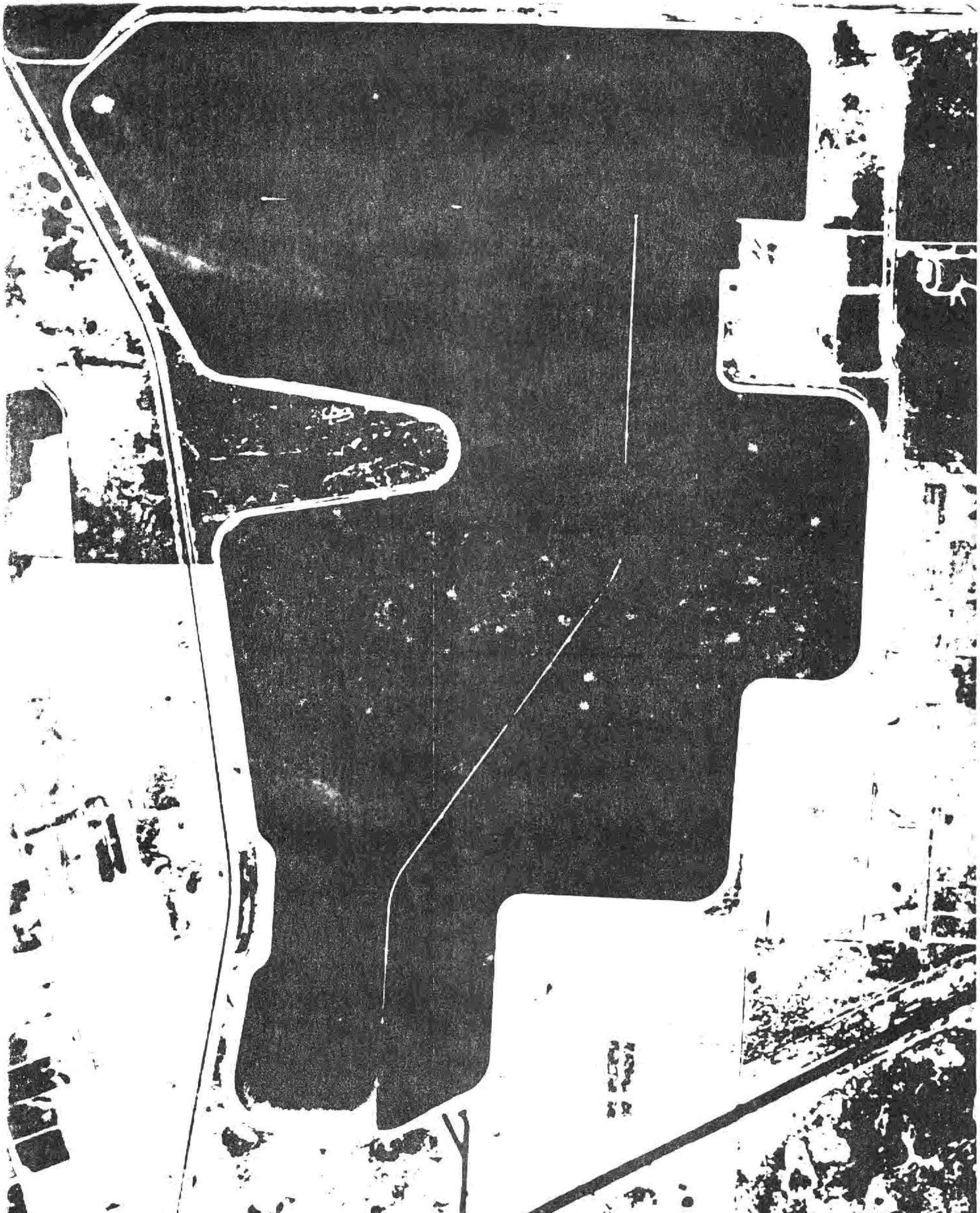


Figure 5-9

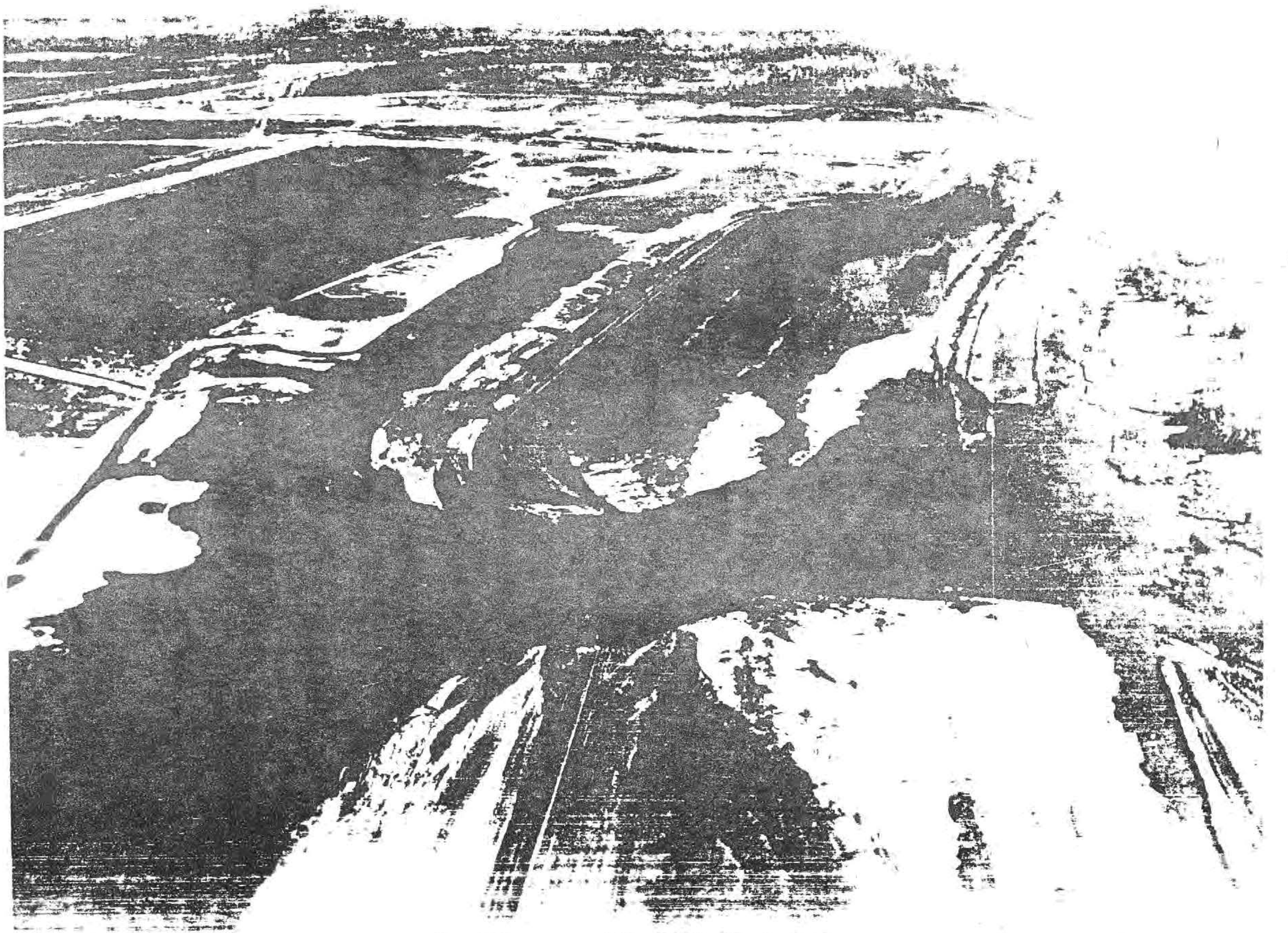


Figure 5-10

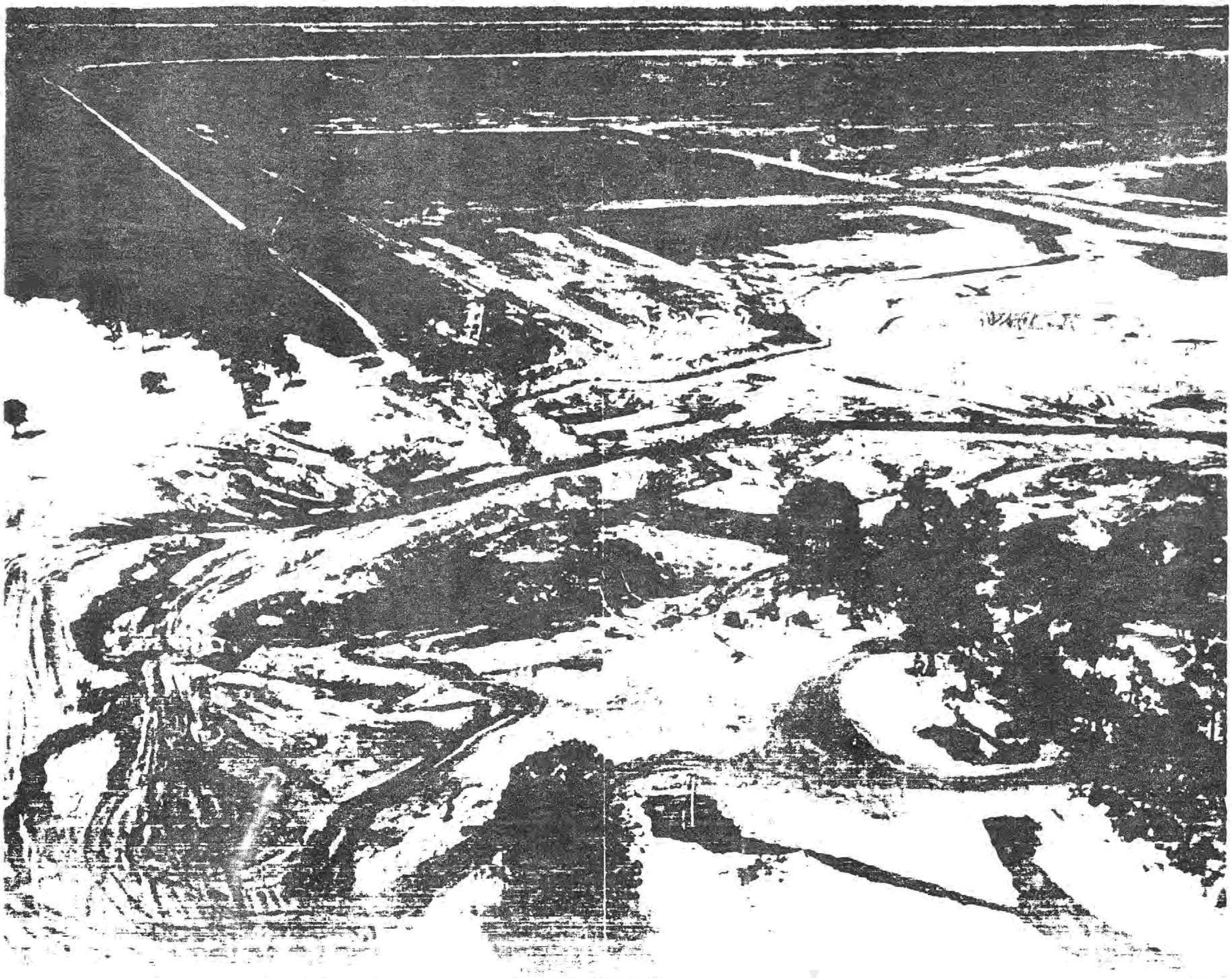


Figure 5-11

communication involved between field and office personnel. With two of the District's principal consultants located as far away as California, photography has been invaluable in providing timely and accurate information.

In all, photography played a unique part in the assessment of all aspects of this investigation and related work.

Analysis

Surface Hydrology

Initially this investigation required that as much pertinent historical data as available be reviewed. These data are displayed chronologically in Composite Data Chart (Figure 4-4A; 4-4B). This chart more easily identifies related events that were not otherwise obvious. The most obvious example was the discharge at District Structure 153 with the simultaneous conditions of rainfall and seepage related to it.

The historical data compiled included rainfall from January, 1972 through December, 1979. This period included the reservoir failure event. All data was not available during these eight years. The data chosen for display was:

1. Rainfall in inches per day recorded at Port Mayaca approximately five miles away. Weather records by FP&L at the reservoir site covered a much shorter time period and there has been some question of their reliability.
2. The discharge at District Structure 153 in cfs taken from District records.
3. The high and low headwater elevation of borrow canal L-65 at Structure 153 recorded by the District and used for the period beginning one month prior to reservoir filling. These were

plotted using only a monthly high and low except during the period proximate to the failure date.

4. Reservoir water levels taken from monthly FP&L records for the entire period beginning at filling and continuing to failure.
5. Computed reservior seepage taken from FP&L reports in cubic feet per second.

In addition to the above, certain milestone dates were added to flag any significant effect the particular item might have. They were dates of construction, commencing and completing filling the reservoir, vandalism at Structure 153, and the date of reservoir failure.

The benefit of the information displayed on the Composite Data Chart is derived chiefly by noting trends over an extended period of time, by scanning chronologically to identify anomalous relationships. For example, a difference in normal discharge at Structure 153 becomes apparent during the last two years, 1978 and 1979. During this period, a consistent discharge can be identified as not previously occurring. Easily identified is the rainfall during any precise period which the reader is free to interpret. Detailed reference and use of this tool is found in various other portions of this report.

Another important element related to the failure mechanism is the effect of the sudden lowering of L-65 borrow canal on October 27, just 3½ days prior to the failure, and the dike tailwater elevation at the time of the failure.

As shown on Figures 1-1 and 1-3 , a narrow strip of land lies between the east levee of L-65 and the FP&L dike. The surface and ground water elevations in this strip are controlled by culverts through this east levee because each culvert has a riser with stop logs welded in place. The top elevation of the stop logs adjacent to the failure was at elevation 18.5 . Had the culvert control been removed either by vandalism or because the culverts blew out water levels might have dropped considerably and a large volume of water might have been discharged into the L-65 borrow canal. Therefore, a water budget was developed for the borrow canal for the period that it was being emptied, 12:15 PM on October 27 through 9:30 AM on October 29, and for the period that the borrow canal was being refilled up to the time of the failure, 9:30 AM on October 29 through 11:30 PM on October 30. In each of these periods the following formula for the water budget prevailed:

Change in storage = Inflow-Outflow

For the first period (canal emptying) the canal stage fell from elevation 19.2 to 14.2. The volume in the borrow canal between these elevations is 327 acre-feet, calculated from the geometry of the borrow canal. The discharge of S-153 was 468 acre-feet during this 45¼ hour period, as calculated from the Corps of Engineers' gate rating curve for one gate open 0.75 feet, one open 1.00 feet, and from the water stage recorders. The inflow estimate was based on the calculated inflow before the vandalism. The inflow estimate is subject to more error in calculation than the change in storage or the outflow calculation. It was based on the assumption that the inflow was the same as that during the previous automatic gate cycle on October 26-27. The inaccuracy is introduced by the fact that the time the gates were open could not be read accurately from the chart recordings where 1 hour equals 0.1 inch. On October 26 the two gates were open 0.8 feet from 7:20 PM to 9 PM and 53.3 acre-feet discharged. The gates were closed from 9:10 PM on October 26 until 1:15 PM on October 27, a period of 16 hours, when the canal again refilled. Therefore, the inflow rate was 53.3 acre-feet ÷ 16 hours or 40 cfs.

Using this inflow rate, the calculated change in storage, and outflow rate, the water budget expressed in acre-feet would be as follows:

$$-327 = 40\frac{(1.98)(45.25)}{24} - 468 = 149-468 = -319$$

The imbalance in the equation is 8 acre-feet or 2%, well within the accuracy of the calculations.

During the second period of 38 hours from 9:30 AM on October 29 to 11:30 PM on October 30, the canal stage rose from 14.2 to 16.3. The change in storage was 129 acre-feet. The outflow was zero. The inflow was $40\frac{(1.98)(38)}{24} = 125$ acre-feet. The water balance equation:

$$129 = 125 - 0$$

The imbalance in the equation is 4 acre-feet or 3%.

The surface area of the railroad borrow pit was about 20 acres. Had the culvert controls been removed and the stage in the borrow pit been lowered 2.5 feet to elevation 16, about 50 acre-feet of additional surface water inflow would have entered L-65 in the first period. The water balance equation would have been as follows:

$$-327 = 149+50 - 468 = -269$$

The imbalance would have been 58 acre-feet or 18%.

Thus, it appears very unlikely from a hydrologic standpoint that the tailwater of the dike was lowered prior to the failure. Additional discussions elsewhere in this report tend to conform this conclusion.

### Subsurface Hydrology-Aquifer Characteristics

During the course of pre-design exploratory investigations a number of aquifer tests were performed at various sites within the reservoir area. The essential purpose of the pump tests was to develop estimates of anticipated seepage conditions, to evaluate horizontal and vertical components of permeability of and between the various formations, and to gain some insight into site specific as well as regional perspectives of subsurface hydrology, particularly underflow and leakance.

Pumping tests were conducted in the sediments above and below the limestone zone, existing generally about elevation -5 to -10. Piezometers were installed both above and below the limestone formation in order to arrive at most reasonable values of permeability in these two zones and to develop some insight into the relative imperviousness of this layer; that is, to assess the degree of hydraulic communication between the sediments above and below the limestone. Basically, this was accomplished by alternately pumping the shallow zone and reading the deeper piezometers and then pumping the deeper zone and observing piezometers in the shallow zone.

The results of the testing clearly indicate that the hydrologic regimen in (and presumably) around the reservoir site is complex and difficult to analyze due in part to the somewhat non-homogeneous, discontinuous nature of the layers of sediment and the clear indication that some of the tests provide evidence that in places the deep piezometers respond when the shallow zone is pumped and, conversely, that the shallow piezometers respond when the deep sediments are pumped. In at least one of the aquifer tests, however, the cemented limestone appears to be an effective impermeable barrier. Independent District analysis of the data concurs with that of FP&L and their consultants.

District analysis and comparison of results with those reported by National Soil Consultants indicates reasonable concurrence that the limestone has a relatively low value of permeability and further, the limestone beds do not act as a hydraulic boundary between the upper sediments and those water bearing formations lying below the limestone. This conclusion was reached upon evaluating aquifer performance utilizing several analytical techniques. It is therefore concluded that the sediments lying above the shell beds (layers J and K) have rather low permeabilities, that the sediments communicate hydraulically and are thus considered to be "leaky" aquifers; and that the ratio of horizontal to vertical permeabilities confirm the vertical drainage (delayed yield) component to the flow regime. The permeabilities derived by the pre-design exploratory work and confirmed by the District by independent analysis, reviewed in conjunction with calculated storage coefficients, provide added confirmation to this rather low permeability and, in addition, confirm the somewhat semi-confined nature of the various sediments. The low permeabilities also provide insight into the areal extent of the cone of depression that would be formed when the shallow aquifer is pumped. Due to low aquifer coefficients and the stress placed on the system, District analysis clearly indicates that the cone of depression would have steep gradients towards the pumping wells and further that the diameter of the cone would be quite small

Throughout the entire reservoir post-failure period, the coincidence of the vandalism that occurred at S-135 and its effect on the stage in

L-65 and the failure event was never too far out of focus of all of the various investigators.

In order to evaluate this event in terms of triggering or participating in the embankment failure, an analysis was made to determine if the lowering of L-65 could have caused a lowering of groundwater levels in the vicinity of the downstream toe of the embankment and if this were shown to be possible, the stress that this event may have placed on the hydrologic regimen in the vicinity of the embankment; and finally, assuming this possibility, whether such an event could have participated in the failure.

The mechanism chosen for analysis is taken from Professional Paper 708, Ground Water Hydraulics, S.W. Loh man, Page 40,41, "Aquifer Tests by Channel Methods, Line Sink or Line Source". In making this analysis two different conditions were evaluated: (1) The release of water from storage is instantaneous and in proportion to the decline in head; and the drain discharges at a constant rate, and (2) an analysis in which the head abruptly changes by a constant amount and the discharge from the formation declines slowly. In both analyses, several Theis type assumptions are made, including that of no recharge. The net effect of the assumptions upon which the mathematical derivations are based is that the solution may be considered a worst case solution. The essential reason being that of no recharge. Obviously, just prior to the failure, water impounded in the reservoir provided an infinite constant recharge. In addition, the stage, and thus groundwater level in the area between L-65 and the FP&L reservoir on its west side, was controlled by the L-65 east levee and its secondary inlet controls as described previously. These controls held surface and ground water elevations in this area including the water levels in the railroad borrow pit, at elevation 18.5.

In addition to the factors cited above, a mathematical analysis was made of the impact of the lowering of stages in L-65. The analysis indicates that, assuming the dike failed 80 hours after the vandalism occured, the lowered canal stages could have lowered water levels in the vicinity of the railroad borrow pit no more than about 0.25 feet. As discussed above, the recharge boundary existing at that time precludes any possibility that the impact of the canal vandalism extended to the downstream toe of the embankment and, in fact, the District interprets the results of this analysis to suggest strongly that the impact of the canal lowering on adjacent groundwater levels did not extend eastward more than a small fraction of that distance toward the westerly shore of the railroad borrow pit.

In summary the District therefore concludes that the low permeability of the several formations, viewed in conjunction with an analysis of L-65 acting as a drain in the immediate vicinity of a significant recharge boundary, could not possibly have played a role in the failure event.

Analysis of Piezometer Measurements

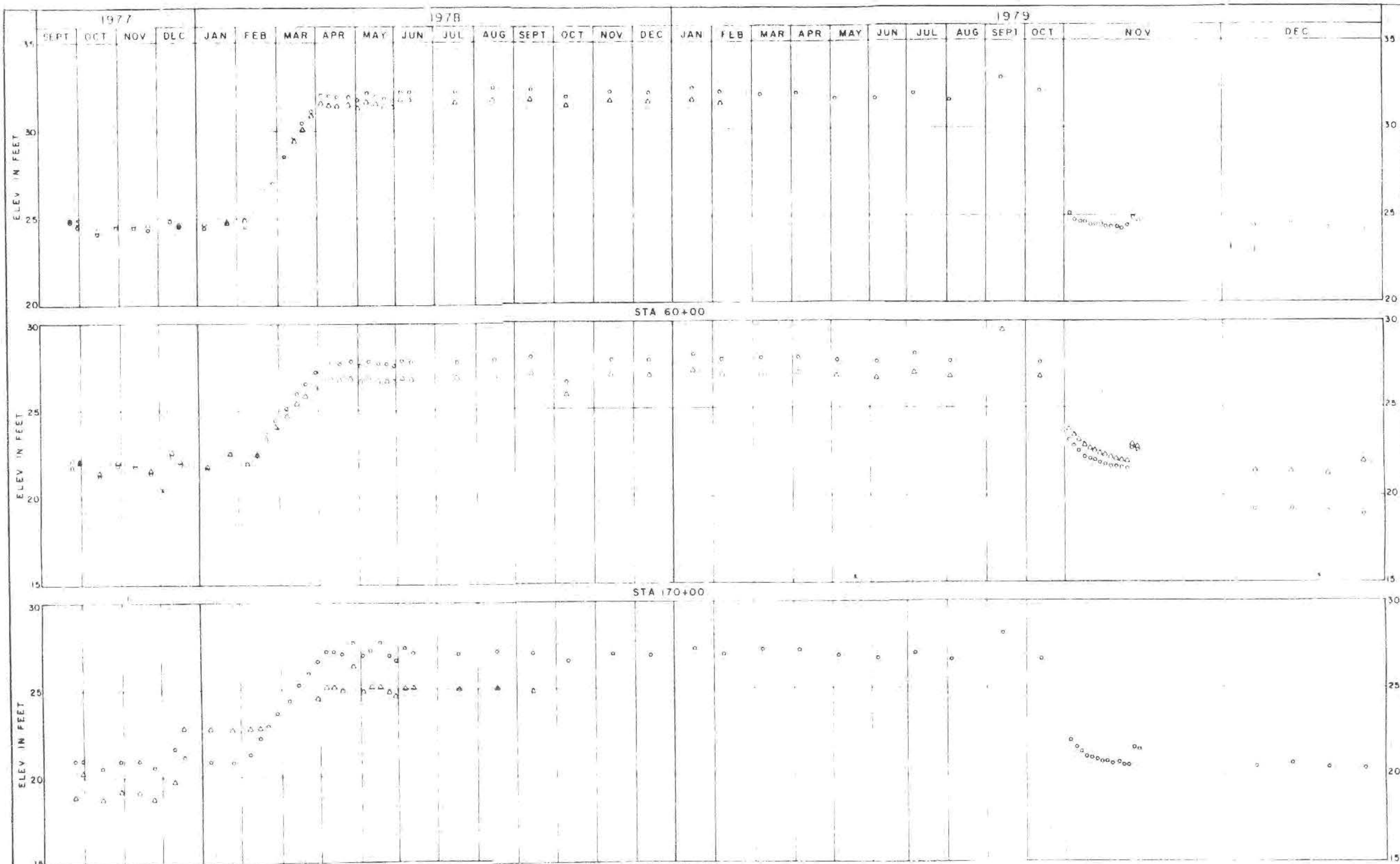
In other parts of this report the District Committee had discussed briefly the importance of the piezometers and also made reference to problems associated with them. Considering its intended use and recognizing the potentially vital nature of the data that are derived from this system, a number of observations concerned with the system as installed are appropriate.

1. Observations of water levels measured in the network were begun on October 23, 1977. Filling of the reservoir was begun on or about February 4 and completed to elevation 37 on April 4, 1978. Pre-construction water levels were about elevation 19. Water levels during this period were taken at a frequency that varied depending on the specific event involved. For example, prior to filling, initial measurements were taken semi-monthly. During the period of filling, and for a short period thereafter, water levels were read on a weekly basis. Thereafter, measurements were made once a month. Post-failure water levels were taken on at least a daily basis for a considerable period of time.
2. An examination of the record reveals that all piezometers within the hydrologic area of influence of the rising pool stage of the reservoir responded immediately to increasing water elevations. For purposes of further study a detailed plotting of Piezometers D and E, located beneath the downstream toe, was constructed. These piezometers measure water levels above and below a rather hard, cemented limestone that was considered to have some ability to hydraulically separate (confine) waters above and below this zone. In other words, this thin limestone layer acted to separate, in general, a water table aquifer from a leaky artesian system. An analysis of the data from the pair of piezometers, indicates by inspection that both "aquifers" responded immediately to rising stages within the reservoir; and, although each of these two water-bearing formations generally had slightly different heads, both responded to the same degree. Thus, a plot of these two

piezometers parallel each other, rising and falling in hydraulic sympathy to one another.

3. The D and E piezometers were plotted for their entire period of record and are shown in Figure 5-12A & 5-12B. An inspection of pairs of piezometeric heads reveals what can be considered to be anomalous conditions. For example, the potentiometric gradient from transect to transect changes. That is, in some transects the water level above the shell is higher than water levels measured below the cemented shell. Thus the gradient would be in a downward direction. This can be seen in the D & E piezometers at station 460+00. Conversely, at all other stations (60, 170, 240, 370, 670, 800) the water levels measured in the deep piezometer (E) stand higher than water levels measured in the shallow wells (D). In this latter situation the potentiometric gradient can be considered to be in an upward direction. The reason for this change in potentiometric gradient most probably relates to subtle changes in permeability of the various soil layers in the vicinity of the piezometers and the position of the regional hydraulic gradient.

4. The direction of the gradient does, however, take on meaning when evaluated in terms of probable causes of failure. One of the theories that had been proposed suggests that a downward gradient could drive the very fine sand overlying the shell intc the underlying open fabric of the shell rock; and, by degrees, sediment would be removed by upward "stoping" and deposited in the shell until finally the reservoir embankment could be breached and failure could occur. The mechanics of such a



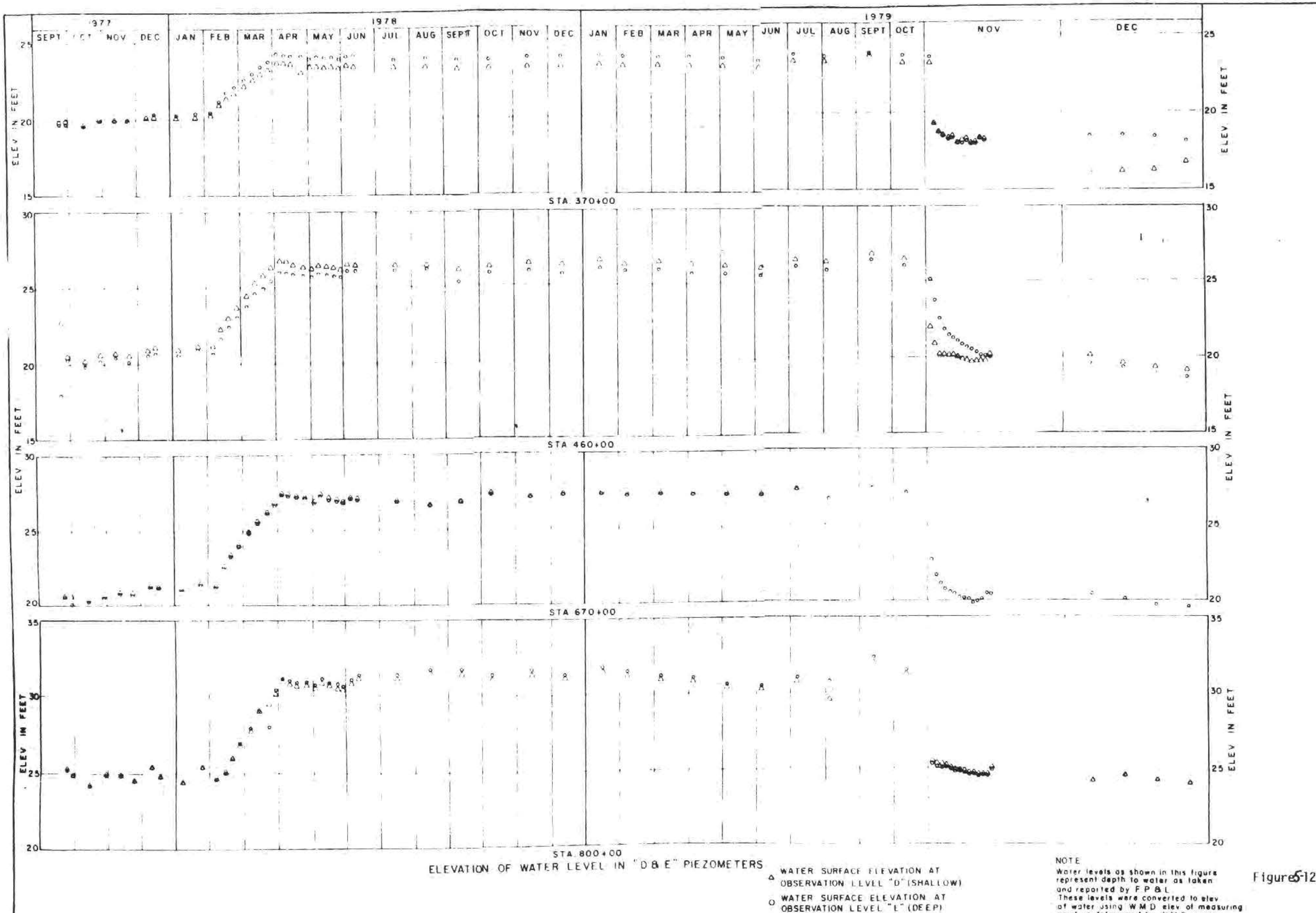

ELEVATION OF WATER LEVEL IN "D & E" PIEZOMETERS

△ WATER SURFACE ELEVATION AT OBSERVATION LEVEL "D" (SHALLOW)

○ WATER SURFACE ELEVATION AT OBSERVATION LEVEL "E" (DEEP)

NOTE:
Water levels as shown in this figure represent depth to water as taken and reported by F P & L.
These levels were converted to elev. of water using W M D elev of measuring point as determined by W M D survey.

Figure 51




ELEVATION OF WATER LEVEL IN "D & E" PIEZOMETERS

Figure 5-12-B

failure mechanism must take into account the differential head between these two formations. Obviously, the greater the head in a downward direction, the higher the velocity and thus the better the ability to transport the sand grains from their in-situ position into the shell rock. Continuing this line of reasoning, one must recognize that there is a minimum velocity (time travel relationship) where the forces available are insufficient to move the sand grains. Further detail concerning this mechanism is covered elsewhere in this report.

Underseepage Conditions in the Failure Area

Because of the concensus concerning the relevance of seepage through the embankment foundation to the failure, great importance was attached to this subject. As one might expect, no piezometers were located in the vicinity of the failure; consequently, the exact flow and pressure conditions cannot be known. Analyses were made, however, based on two sets of conditions - one average, one extreme. The average condition was defined by the average pressure at each similar piezometer location from four of the seven transects around the reservoir. The piezometers on the high ground areas on the northeast corner of the reservoir were not included in the average. The piezometers at station 370 were also omitted from the average because the downstream toe readings appeared so different from all others.

As discussed above, the reference points of many of the piezometers were in error. The studies, however, were made before the reference point corrections were obtained. The error from this fact was small, because the reference point errors in the averaging operation tended to cancel each other. Table 5-5 shows the effect of the reference point errors.

TABLE 5-5

Effect of Reference Point Errors on Average Piezometric Pressures (July 17, 1978 July 5, 1979 October 10, 1979)

Average Pressure in Feet

| Piezometer | Uncorrected | Corrected |
|---|---|---|
| A | 35.0 | 35.0 |
| B | 28.7 | 28.9 |
| C | 28.6 | 28.6 |
| D | 27.0 | 26.3 |
| E | 26.6 | 27.0 |
| F | 23.3 | 23.3 |
| G | 25.0 | 24.8 |
| H | 23.3 | 23.0 |
| I | 22.0 | 21.8 |

The extreme condition was more difficult to define. Uncorrected readings indicated downward piezometric differentials through the cemented shell layer at the downstream toe of the embankment at three locations, with a maximum value of one foot. It was reasoned that if a one foot downward differential could exist at one location, greater differentials could exist elsewhere. Consequently, it was assumed that an extreme case could involve a downward differential of 2½ feet through the cemented shell at the downstream toe of the embankment. After the reference point elevations of each piezometer had been accurately surveyed, however, it was revealed that downward gradients at the downstream location were present at only one station with a maximum differential of only ½ foot. Nevertheless the assumed extreme case still appears reasonable because whatever the failure mechanism was, it was worse at the location where the breach occurred than at those locations where failure did not occur. Table 5-6 shows the corrected and uncorrected piezometric differential through the cemented shell layer at the downstream toe of the dike.

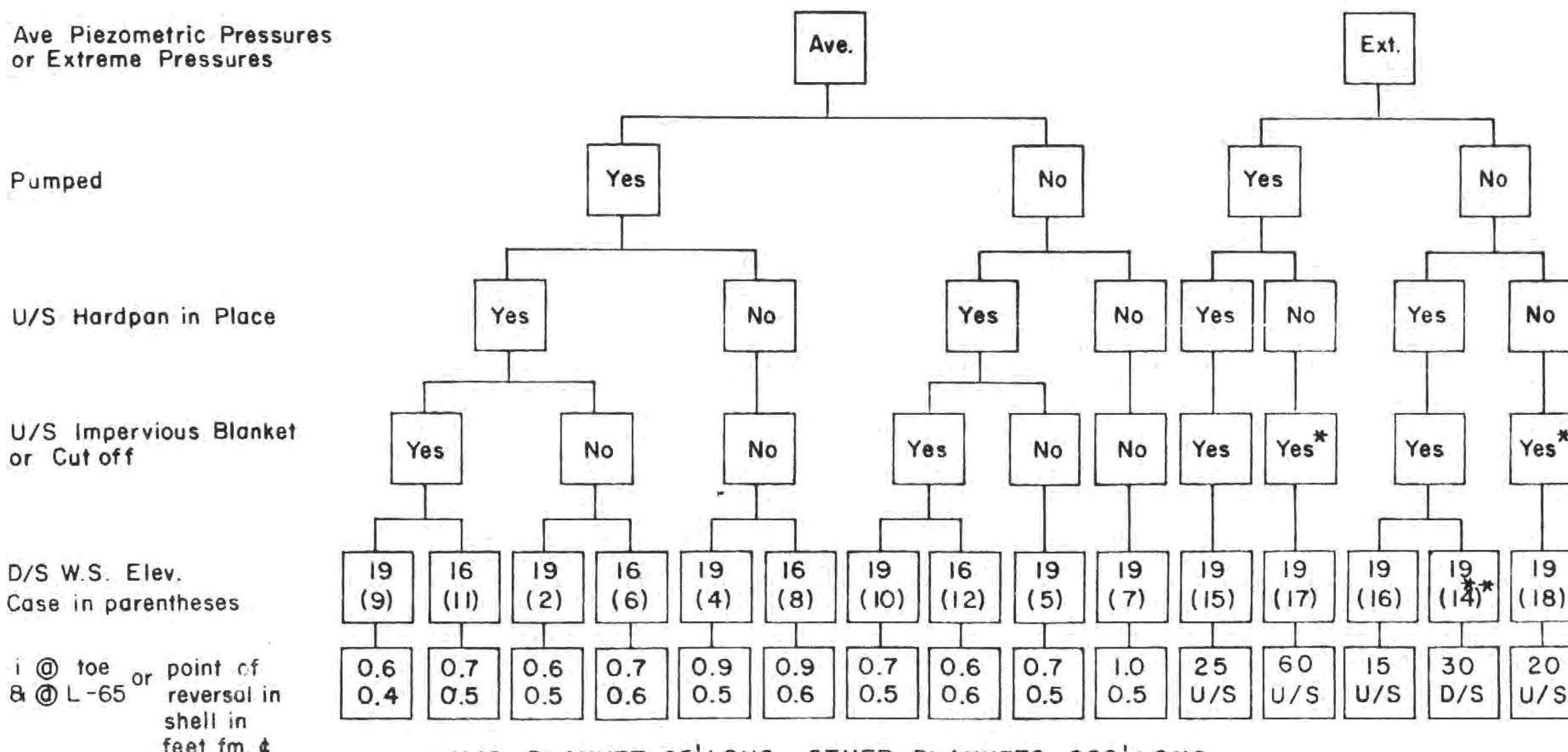
TABLE 5-6

Piezometric Differential through Cemented Shell
Downstream Toe of Embankment
(July 17, 1978, July 5, 1979, October 10, 1979)

| Station | Corrected | | Uncorrected | |
|---|---|---|---|---|
| | Differential in feet | Direction | Differential in feet | Direction |
| 240 | 2.1 | up | 0.1 | up |
| 170 | 1.2 | up | 1.0 | down |
| 60 | 0.6 | up | 0.6 | up |
| 370 | .45 | up | 3.2 | up |
| 670 | .0 | — | 0.2 | down |
| 460 | .44 | down | 0.5 | down |

Flow nets were constructed to demonstrate the flow and hydrostatic pressure distribution in the embankment proper and its foundation. These flow nets are included in the appendix. Fig. 5-13summarizes the various cases studied and the conditions prevailing with various remedial works under various assumed conditions. Cases 1, 3 and 13 represent conditions in the breach area is they might have been prior to the failure. Cases 1 and 3 are based on the assumption that the piezometric pressures were the same at the breach as the average around the rest of the reservoir. Case 1 is based on the assumption that the hardpan layer was present; case 3 is based on the assumption that the hardpan was absent. Case 13 is based on the extreme case, previously discussed, with a 2½ foot downward head differential through the cemented shell layer at the downstream toe, with hardpan present.

# POST FAILURE FLOW NET ANALYSES
# WITH REMEDIAL WORK IN PLACE




* U/S BLANKET 65' LONG, OTHER BLANKETS 250' LONG.
** CASE 14 IS THE ONLY CASE WITH NO D/S DRAIN.

Figure 5-13

The thickness and permeability of the various materials in the dike and its foundation are summarized in Table 5-7 as follows:

TABLE 5-7

Soil Permeabilities and Thickness
Assumed in Flow Net Analyses

| Stratum | Description | Permeability (cm/sec) | Thickness (feet) | Elevations (feet) |
|---|---|---|---|---|
| Embankment | -- | $10^{-5}$ | -- | +50 to +20 |
| 1st | fine sand | $10^{-3}$ | 3 | 20 to 17 |
| 2nd | hardpan | $10^{-6}$ | 4 | 17 to 13 |
| 3rd | silty sands, fine sands etc. | $10^{-4}$ | 20 | 13 to -7 |
| 4th | cemented shells w/openings | $10^{-6}$ | 2 | -7 to -9 |
| 5th | porous shells | 1 | 10 | -9 to -19 |

The flow nets demonstrate that critical conditions could have existed for two cases - piping of the sand layer beneath the hardpan into the railroad borrow pit or piping of the sand through the cemented shell layer. These failure mechanisms and possible remedial actions are discussed in greater detail later in the report.

Embankment Stability Analysis

The presence of a very weak layer (H) about 25 feet beneath the base of the embankment caused concern that failure could have occurred through this material. This suspicion was further enhanced because of the fact that the design did not recognize the presence of this stratum nor did the design theoretical failure surfaces pass through it. Various methods and strength assumptions were made by both the WMD and the FP&L consultant investigators, but all reasonable analyses indicated that the failure was not due to this mechanism involving layer H. The results are presented here, nevertheless, to demonstrate that another mechanism must have been responsible.

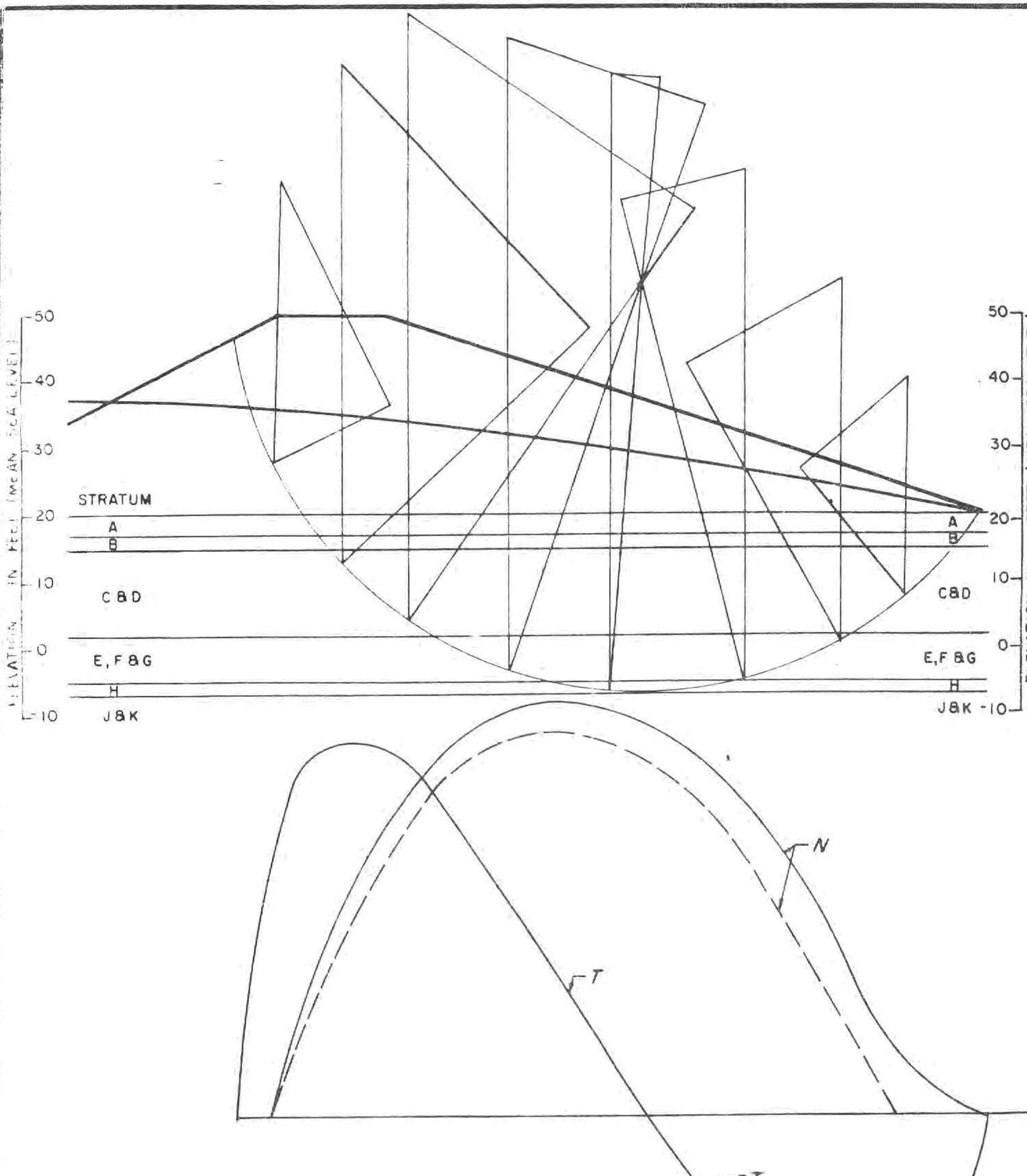
The stability of the dike was analyzed by both the Swedish circle and the sliding wedge methods. Obviously, the results of any stability analysis vary with the assumed soil characteristics and hydrostatic pressure distribution. Since most of these parameters are imperfectly known in the failure area, various assumed values were employed.

Since all analyses were based on assumed values, it was not deemed necessary to find the most critical failure case, though several near critical ones were analyzed. Fig. 5-14 and 5-15present typical cases for the two methods employed. Table 5-8 summarized the various cases analyzed and some of the assumptions inherent in each. Table 5-9 summarizes the remaining soil characteristics used in the analyses.

TABLE 5-8

Soil Characteristics Used in Stability Analyses

| Layer | Thickness in Feet | Elev. | Angle of Internal Friction in degrees | Cohesion | Unit Dry Weight in #/cf | | Moisture Content in % |
|---|---|---|---|---|---|---|---|
| | | | | | Moist | Saturated | |
| Enbankment | -- | 50 to 20 | 35 | 0 | 125 | 131 | 14 |
| 1 | 3 | 20 to 17 | 33 | 0 | - | 118 | - |
| 2 | 2 | 17 to 15 | 33 | 0 | - | 137 | - |
| 3 | 13 | 15 to 2 | 33 | 0 | - | 125 | - |
| 4 | 7 | 2 to -5 | 33 | 0 | - | 125 | - |
| 5 | 2 | -5 to -7 | various | 0 | - | 118 | - |

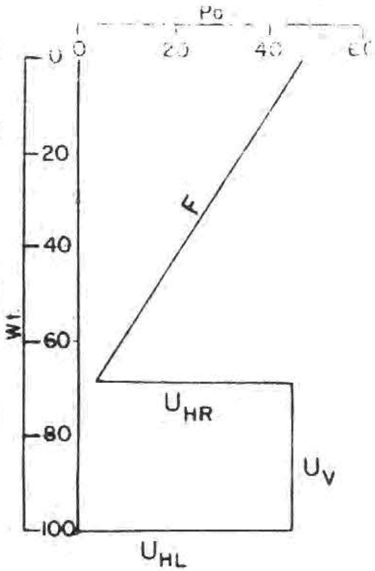



——CASE 1
– – –CASE 2

SAFETY FACTOR = $\frac{\Sigma \text{ N Tan } \phi}{\Sigma \text{ T}}$

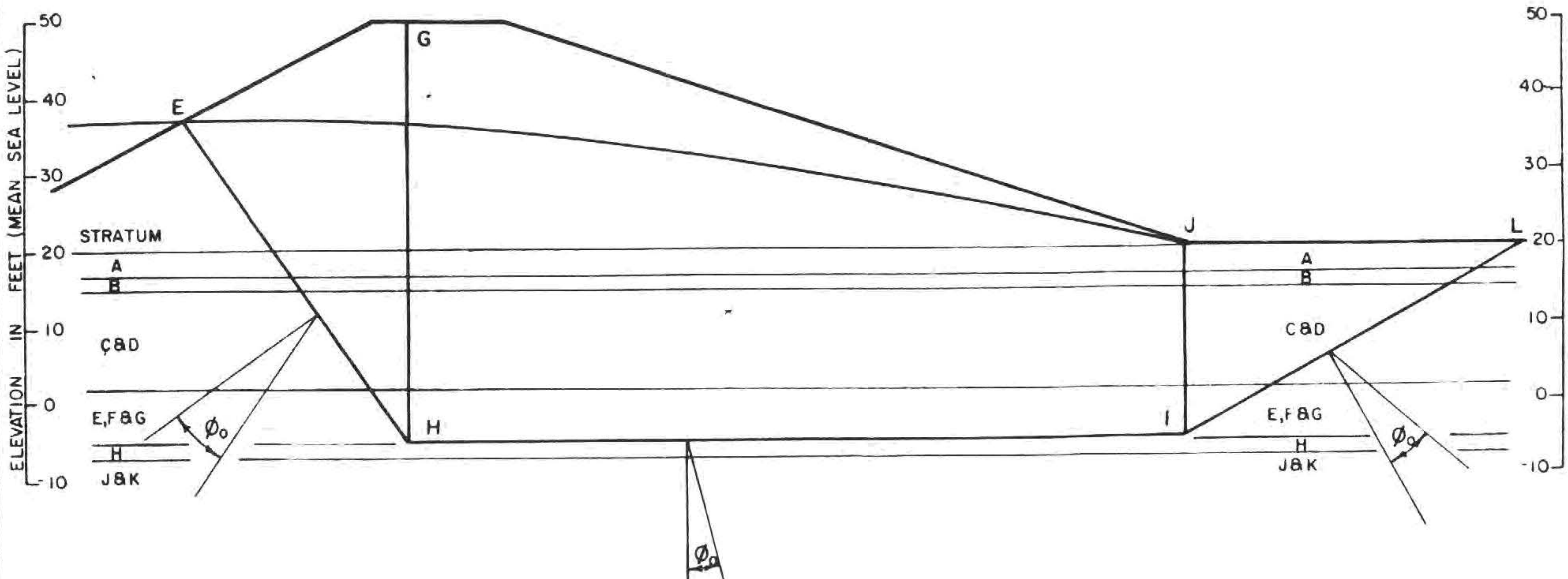
## TYPICAL STABILITY ANALYSIS
( SWEDISH CIRCLE METHOD)

Figure 5.14




CASE 7

| BLOCK OR WEDGE | WT. (Kips) | $\phi_W$ | $U_{HL}$ | $U_{HR}$ | $U_V$ | TRIAL SAFETY FACTOR | Tan $\phi_0$ | $\phi_0$ | F | $P_A$ | $P_{PW}$ | $P_{PB}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ACTIVE WEDGE EHG | 101.3 | 33.3° | 45.6 | 42.2 | 31.9 | 1.7 | 0.3858 | 21.1° | 82 | 48 | | |
| PASSIVE BLOCK GHIJ | 427.0 | 25° | 42.2 | 32.6 | 207.5 | 1.7 | 0.2743 | 15.3° | 60.3 | | | 50.7 |
| PASSIVE WEDGE JIL | 56.5 | 32° | 32.6 | 32.6 | 57.3 | 1.7 | 0.3676 | 20.2° | | | 0 | |




## TYPICAL STABILITY ANALYSIS

SLIDING BLOCK METHOD)

Figure 5-15

TABLE 5-9

Stability Analysis Summary

| Case | Layer H Angle of Internal Friction in degrees | Hydrostatic Pressure | Safety Factor |
|---|---|---|---|
| | | Swedish Circle Method | |
| 1 | 25 | A | 1.58 |
| 2 | 25 | B | 1.30 |
| 3 | 0 | A | 1.05 |
| 4 | 0 | B | 0.81 |
| 5 | 11 | B | 1.00 |
| | | Sliding Block Method | |
| 6 | 25 | A | >2.0 |
| 7 | 25 | B | >1.7 |
| 8 | 10 | B | 1.00 |

A - Hydrostatic pressure between hardpan and cemented shell as measured in the piezometer transect at Station 370, one mile south of the break, about elevation 25. This case corresponds roughly to the flow net No. 1 in the Appendix.

B - A high hydrostatic pressure between hardpan and cemented shell of elevation 30. This case corresponds roughly to flow net No. 3 or 13 in the Appendix.

The FP&L consultants also studied the stability of the embankment, but obtained somewhat different results, partly because of less conservative assumed soil properties and partly because their analyses took into consideration the effects of restraining forces perpendicular to the analysis plane. Though a complete comparison between the two approaches does not seem warranted because this type of failure is not considered probable, it should be pointed out that the FP&L consultants used angles of internal friction of 35 to 42 degrees compared with 33 to 35 degrees for the various soils shown in Table 5-8and used by this Committee in the analysis described above.

The factor of safety calculated by FP&L which corresponds roughly with Cases 2 or 7 (Table 5-9) is about 1.9, whereas that which corresponds roughly with Cases 1 or 6 is about 2.5

In order to interpret the results presented in Table 5-9,several comments are appropriate. First, it is more likely that hydrostatic pressure condition A prevailed because the hardpan was observed upstream of and in the south side of the break. Second, the angle of internal friction of the weak layer (H) was probably on the order of 20 to 25 degrees as determined by the FP&L investigations (see Table 5-2 ). The stability analyses were made prior to receipt of the soil test results. This very loose material appears to be very prone to liquefaction when subject to a suddenly imposed load. During liquefaction of any soil, all the load on that soil is transferred from the soil skeleton to the entrapped pore water, and since water cannot support shear, the angle of internal friction drops to zero. Thus, the analysis indicates that had liquefaction occurred, the stability of the embankment would have been critical.

Though cases of dam failures from liquefaction are rare, and are never easy to prove, one dam in central California is believed to have failed from this cause and the State of California is presently considering abandoning another dam very similar to the FP&L facility because of this potential problem. Liquefaction, however, does not occur without a triggering mechanism. A large shock wave created by a mechanism such as an explosion or an earthquake is the most likely cause of such a triggering mechanism. But in this case, no such action was present. Moreover, the limited post-failure density testing in the breach area (see Table 5-3 ) appears to indicate that the suspect layer (H) is no longer loose; i.e., the relative density is about 100%.

In spite of all the calculations, the most convincing evidence against a shear failure is the fact that the natural soils (layers G and H) are still present in the breach area in the locations through which the failure surface would have to have passed. Consequently, all stability analyses are rendered academic, and are included in this report only for the sake of completeness.

# CHAPTER 6

## EVALUATION OF FAILURE MODES

Many possible failure mechanisms have been postulated by almost everyone even remotely associated with the project or affected by the failure. Consideration has been given to every such thesis. In all honesty, however, even after the expenditure of a great deal of money, after thousands of man-hours of painstaking investigations, after the removal of thousands of cubic yards of soil in the breach area, and the sifting and testing of countless soil samples, it must be admitted that THE one failure mechanism cannot be positively identified. On the other hand, many of the proposed mechanisms can be discarded as not possible because they do not fit all the evidence. It is the opinion of the District's Committee that only two failure mechanisms are in agreement with all the physical evidence and with accepted engineering principals.

<u>Piping Tunnel to Old Railroad Borrow Pit</u>

The most likely failure mechanism is piping or erosion of the embankment foundation, probably from layer C, a white cohesionless sand, into the old railroad borrow pit. This piping action could have been going on for many months, during which time sand was being deposited below the surface of the borrow pit, hidden from view by floating aquatics, which are clearly seen on aerial photographs. The piping progressed backwards from the borrow pit, forming a tunnel beneath the embankment until it reached close to the upstream toe of the embankment. Then suddenly, on the night of October 30, 1979, the thin layer of soil separating the tunnel and the reservoir broke through and a torrent of water rushed through the tunnel at a very high velocity. The torrent quickly eroded the soft sand forming the sides of the tunnel, until the tunnel was ~~to~~ so large that the overlying embankment collapsed into it.

The flow through the breach so formed widened and deepened, eroding soil from behind the soil cement, which collapsed in place. As the breach enlarged, the rate of discharge increased, but the velocity of flow through the breach remained generally constant, on the order of 15 - 20 feet per second.

In order for the above scenario to take place, a hydraulic gradient would have to have been present which would favor piping. Unfortunately no record exists of the piezometric levels at this location. It is possible, however, to transfer the measured levels to the breach location, assuming that these values could have existed there, but also recognizing that more critical values could also have been possible. As discussed previously, flow nets were drawn based on the average piezometric pressures. Case 1 in the Appendix, is such a case, which demonstrates that a near critical condition could have existed at the edge of the railroad borrow pit. A critical condition is defined as one in which the hydraulic gradient at the exist point is 1.0 or 100%. Case 1 has an exit hydraulic gradient of 0.7 or 70%. This analysis is two-dimensional, whereas actually the flow is three dimensional and a further concentration of the flow net will occur in the horizontal plane, further increasing the true hydraulic gradient. Additionally, this flow net was drawn on the basis of the average piezometric values, whereas, the actual situation could have been more (or less) critical. The flow net also shows a high gradient into the toe drain. The toe drain, however, is located in the relatively impervious brown, silty sand (layer B), which is less susceptible to piping.

Several arguments have been advanced against this failure mechanism. These will be examined in detail.

First, it is argued that, because there was no appreciable leakage emerging at the downstream toe if the embankment at this location, the water pressure head in the upper part of the foundation sands could not have been much greater than the ground surface level. This assumption is not true because the impervious layer B would have greatly retarded such seepage. While it may be true that the topmost black organic portion of this layer may have been penetrated by the toe drain, Test Trench 21 intersected several feet of the lower portions of this layer, which was exposed in the full length of the trench.

It is further argued that the hydraulic gradient was too low. The flow nets in the appendix, previously referenced, show that the gradient at the exit point could easily have been 100% and the average between the borrow pit and the downstream toe could have been (26.5 - 19)÷150 = 5% or greater. A 5% gradient is an emperical value considered necessary by many engineers, in order to produce a piping failure. Dr. Schmertmann, in the report of the FP&L Review Board, also developed a theoretical analysis which indicated that this gradient is the level required to produce a piping failure.

Another argument is advanced that no tunnels were observed in layer C beneath the hardpan in Test Trench 21. While this is true, it is not entirely unexpected since TT 21 is not located in the area where the highest hydraulic gradients would have occurred. It is somewhat surprising, however, since the hydraulic gradients would have been fairly high in TT 21 also.

Another argument is that in order to collapse a dam, a large quantity of material would have to have been removed and such a quantity would have been easy to spot. The answer to this argument is that the eroded material could easily have been "hidden" in the railroad borrow pit beneath the floating aquatics. Calculations show that the northernmost 100 feet of the eastern pit contained space for over 2,000 cubic yards of soil.

Erosion of a tunnel five feet in diameter between the edge of the borrow pit and the upstream toe of the dike would have removed only 220 cubic yards.

Sand Piping Downward Through Cemented Shell Layer

The only other possible failure mechanism involves a much more unique set of circumstances. It also involves erosion of the foundation beneath the embankment and its subsequent collapse. According to this scenario, erosion would have been occurring for many months, and the final collapse of the embankment on October 30, 1979 would have been only the final phase of that process.

The key to this mechanism also involves the distribution of piezometric pressures beneath the embankment, and again it must be pointed out that these pressures were not known at this location. In this case, the pressure distribution would have to have been similar to that recorded at the piezometer transect at station 460 where the pressures above the cemented shell layer (K) were higher than those below this layer, beneath the entire embankment. An average pressure differential of 0.4 foot was recorded beneath the downstream toe at station 460, during the period when the reservoir was full. During this same period, the pressure differential across the cemented shell at the embankment center line was downward an average of 1.2 feet. If such downward differentials could exist at one location, it is reasoned, they could easily have existed, and even been exceeded at another location. Hence, Case 13 in the Appendix was assumed to represent such a possibility.

With the 2.5-foot downward differential of Case 13 through this one or two-foot thick layer, a very high gradient would have been present. Numerous small holes have been observed in the cemented shell layer during the post failure exploration. With such a large downward gradient, the fine sand above the cemented shell could easily have been transported through the cemented shell and into the very open layer (L) beneath. Sandy material above the cemented shell would then settle downward and leave a void beneath a competent layer above the shell. This void would have progressed downstream until it reached the vicinity of the downstream toe. Under such a condition the head loss in the void or tunnel would have been minimal, and a large hydraulic head would have existed at the downstream toe, which could have caused a blowout , allowing a flow of water to pour freely from the reservoir.

An agrument against this mechanism is that no downward gradients through the cemented shell of the order used in flow net Case 13 were observed at the downstream toe. The answer to this agrument is that, since no measurements were made at the breach, the pressure distribution could have been anything. Since a downward gradient was observed, even at only one location, downward gradients could have been present elsewhere. Moreover, flow net Case 13 is a plausible case; it is certainly possible to have occurred and hence, it cannot be discounted.

A secondargument against this mechanism is that, were it valid, it is most likely that a similar mechanism would have been occurring elsewhere in the 19 miles of embankment. Evidence of such an activity would be cracking of the road on top of the embankment, evidences of collapse failures in the upstream soil cement slope protection or sink holes adjacent to the upstream toe in the reservoir. None of these evidences have been positively identified. The argument is true as to the lack of evidence of collapse failure in the embankment. On the other hand, numerous depressions are present in the reservoir, but because of the activity associated with the original construction, it is not possible to determine if these depressions were caused by the construction or by downward collapse. Some apparent sinks near station 380 were observed which had bushes in the bottom of them. Hence, it appears that these might be genuine sinks. **Moreover, the most recent exploration did reveal two areas of very weak zones or voids beneath the embankment.**

Improbable Failure Mechanism

There are seven other postulated failure mechanisms which for one reason or another could not have occurred, or are so improbable as to warrant no further serious consideration. Each will be mentioned together with the reasons why they were rejected. The order in which they are presented is of no significance.

Piping to Toe of Embankment - This mechanism would consist of leakage in the strata immediately below the base of the embankment, emerging at the downstream toe with sand being carried by the escaping water. The sand particles so emerging would leave a void or tunnel, which in time would have progressed upstream, beneath the embankment toward the reservoir. Eventually the tunnel would reach the reservoir or would enlarge sufficiently to the point at which time the roof of the tunnel would collapse, cracking the dike and allowing the reservoir to flow through the crack. Such a piping tunnel could form, it is argued, in layer A or C.

This failure mode was rejected because no seepage was observed at the toe of the dike during the 18 months of reservoir operation up to 4 p.m. of the evening of the failure. It is inconceivable that a leak would begin at 4 p.m., and form a tunnel which would progress approximately 160 feet beneath the embankment in 7 hours, when no change in conditions had occurred from those that existed for the preceeding 18 months, during which no seepage had occurred.

Piping to L-65 - This mechanism is similar to the last one except the seepage would have emerged in the L-65 canal, presumably below the water surface, and since no one was looking for it there, quite an accumulation of sand from layer C could have been carried by the escaping seepage and deposited in the canal during the 18 months of reservoir operation. Flow net Case 1 in the appendix show a fairly high exit hydraulic gradient into the canal. The average gradient between the reservoir and the canal, however, is about $(37 - 14) \div 750 = .03$ or 3%, even when the canal was at its lowest level, or 2½% when the canal was at its normal level.

There are several arguments which render such a mechanism most improbable. First, there are miles of embankment just as close to the canal as at the point where the breach occurred with very similar subsurface conditions. Had this been the mechanism some evidence of piping would be expected somewhere else but none was observed. Second, the old railroad borrow pit is closer to the reservoir than the canal; the path to the railroad borrow pits had a higher hydraulic gradient and hence would have been far more susceptible to failure. Finally, the hydrauilc gradient to the canal by accepted criteria is insufficient to cause a failure, even ~~then~~ when the canal was lowered.

It is inconceivable that piping could have occurred in the loose, green, silty sand of Layer H for two reasons. First, the top of this layer at the canal location is at an elevation of -6, about 9 feet below the bottom of the canal, which is at an elevation of +3. Even more importantly, the green, silty sand is present on the axis of the embankment in the breach area and in the canal section. Had a pipe developed from the canal to the reservoir, it would have removed all of this material all the way from the breach to the canal.

The only layer through which a tunnel or tunnels could have developed between the reservoir and the canal is layer F, a clean, white sand, which in all areas of the excavation was very subject to tunneling. For the reasons outlined above it seems highly unlikely that even this layer could have supported a pipe over the great distance under the low gradient that existed.

Construction Deficiency - Numerous accusations have been made by employees or former employees of FP&L that construction irregularities existed, and that these caused the embankment to fail. These charges have been rather vague and hard to substantiate. They fall into four categories - poor compaction, poor demucking under the embankment, organic material in the embankment, and "cement in washouts". It is not clear what is meant by the last item, but any ill effects from the other three items would have shown up shortly after the reservoir was completed. The stability of the embankment above ground was very conservative. Many deficiencies in the construction could have been made without endangering the structure. Moreover, an embankment of this kind becomes ever more stable with the passage of time; consequently, charges of poor construction are considered invalid in explaining the failure.

Failure of Soil Cement - It might be argued that wave wash, especially during Hurricane David in early September, 1979, could have damaged the soil cement slope protection allowing waves to attack the underlying embankment.

This mechanism is considered impossible for a number of reasons. First, the soil cement facing was inspected many times after the hurricane and no damage noted. Second, had even minor damage in the soil cement gone unnoticed and had wave wash occurred to the embankment, the latter is far too massive at the water line elevation for erosion to progress to the downstream face without affecting the physical condition of the crest road. This condition ~~whould~~ would have been noticed during routine inspections by supervisory and other personnel who traversed this road daily for some considerable period of time prior to the failure.

Sabotage - FP&L has been having some labor problems, and has experienced considerable sabotage damage elsewhere in the past during times of more serious labor unrest and strikes. Consequently, attempted sabotage is always possible. It seems most unlikely because a structure as massive as a dam is not easy to destroy. To do so would have required considerable expertise and either considerable explosives, the detonation of which would have been noted, and/or power equipment. This area is not open to the public and access to it is through the power plant area which is patrolled and fenced. No evidence of cut fences was found and the gates are guarded 24 hours a day. Even with power equipment it would have been necessary to dig a trench 13 feet deep and almost 100 feet long, including removal of the 6-ft. thick soil cement layer, in the few hours between dark and about 11 p.m.

Liquifaction - The green, silty sand, layer H, has all the characteristics of being subject to liquifaction. Testing of this material before construction and after the failure indicated that is is a prime candidate for such a phenomenon. Under other circumstances liquifaction could not be ruled out. In this case, however, it cannot be considered seriously. First, there was no triggering mechanism to induce liquifaction, which is a process in a loose sand of transfer of the load being carried by the intergrannular soil particles to the water as the soil particles are rearranged into a more dense state. Had the canal lowering been such a trigger mechanism, the failure would have occurred three days earlier, when the lowering took place. Second, had the green sand liquified, a sliding failure would have occurred, and this layer would no longer be present on the embankment axis and in the canal bank.

Shear Failure - As stated previously, the presence of the very weak green, silty sand, layer H, is of concern not only as a candidate for liquifaction, but also as a candidate for a conventional shear-type failure. An analysis of the possibility of this type of failure has been presented previously, along with the arguments why it is considered impossible to have caused the failure.

# CHAPTER 7

# CORRECTIVE MEASURES

Preceding parts of this report detailed the activities concerned with a comprehensive testing and exploration program that was designed to detail to the extent possible, the physical features related to all reasonable modes of failure. These potential failure modes were subsequently distilled down to two probable causes. The process of evaluation and analysis that resulted in this reasoning process was carried out by the two teams of FP&L consultants and by the District's investigative committee on an independent basis. Throughout this rather intensive process, a number of workshop meetings were convened for the express purpose of exchanging ideas, receiving comments and suggestions, and to argue the logic of each of the possible failure mechanisms.

There were many reasons why this process of analysis was necessary. One of the most compelling was the recognition that, as is common in this type of event, the exact nature of the failure is rarely uncovered as all of the primary evidence would be obliterated in consequence of the failure event. Therefore, one could only determine, based on best available evidence, a series or probable causes. In planning for the repair of the breached section, as well as for the entire remainder of the reservoir, the essential philosophy was adopted that any recommendations concerned with the rehabilitation of the entire reservoir would, of necessity, include modifications to the embankment and appurtenant areas that would protect against all probable causes of failure and that the modifications would be designed such that none of the probable causes identified could cause a repetition of the failure.

Phased Reconstruction Activities

On the basis of this reasoning, a series of workshop meetings was held throughout the investigative process. In attendance at all of these meetings were representatives, and their consultants, representing the following teams and agencies: FP&L staff, Mid Valley, Inc. (Design Engineers), FP&L Board of Consultants, FP&L Board of Review, Martin County, Governors Committee, WMD Committee, Corps of Engineers. The essential thrust of these meetings and some of the highlights are as follows:

1. Meeting of November 30, 1979. A workshop meeting was convened after completion of a substantial portion of the field work and associated analysis. At this meeting an extensive series of possible failure modes was presented, discussed and evaluated. An analysis of all possible failure modes has previously been presented.
2. Meeting of December 14, 1979. The main thrust of this meeting concerned technical discussions relating to five probable causes of failure. This was a distillation of the possible causes of failure previously discussed. Preliminary plans relating to reconstruction in the breach area were reviewed. These repairs were designated as Phase 1 and 2. Phase 1 consisted of filling in the scour area and the railroad borrow pits back to original grade. Phase 2 consisted of the repair of the embankment to original specifications. The essential distinction between these two phases of work was as follows: Phase 1 required placement of fill back to original grade in the scour areas and the several railroad borrow pits. Compaction was accomplished by the equipment placing the fill. Compaction beneath the embankment was required to be 95% of maximum density.

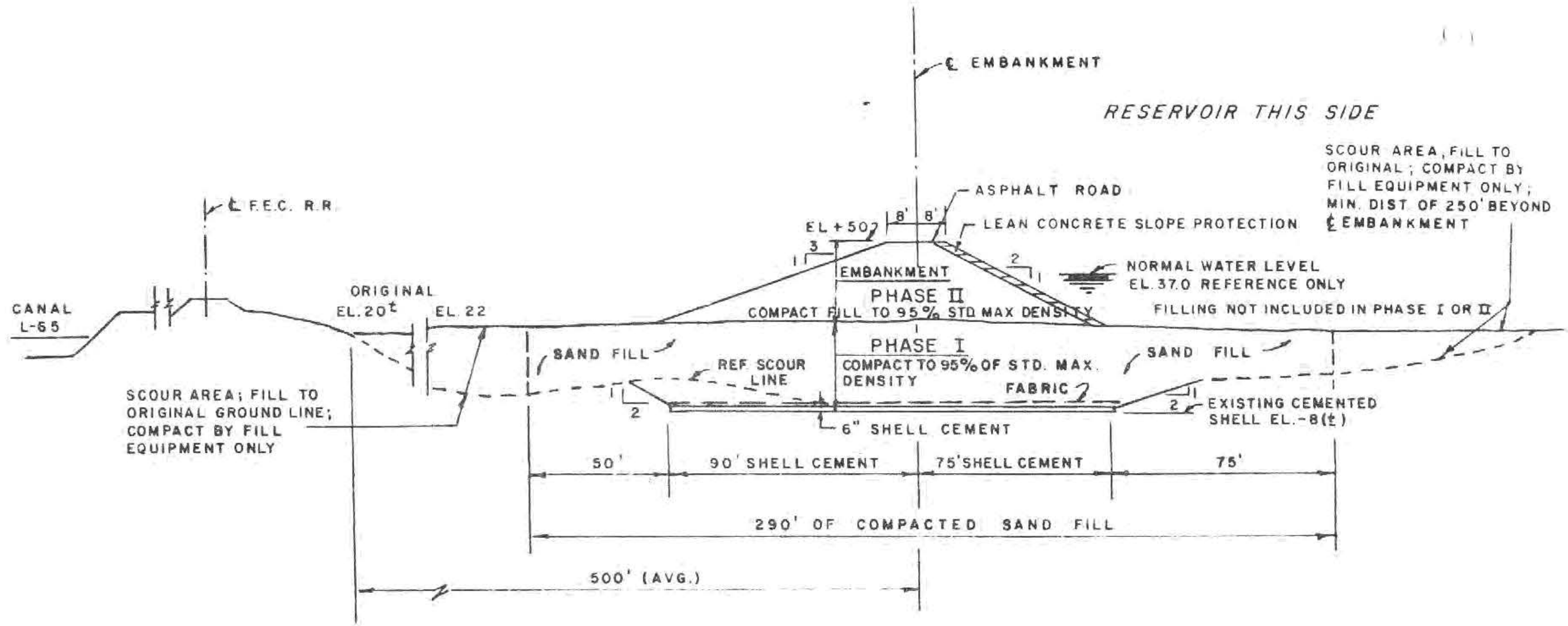
Phase 2 fill placement for the embankment in the breach area required compaction to 95% of maximum density. Figure 7-1 presents a typical east-west section through the breach and scour area. The placement of fill, comprising Phase 1 and 2 repairs, and the relationship between these two elements of work is shown. Figure 7-2 is a plan view of the area that puts this work into areal perspective.

During the workshop of December 14, preliminary discussion was also directed to rehabilitative procedures to the remainder of the reservoir.

3. On December 29, 1979, a workshop meeting was held to consider the final details of the Phase 1 and 2 repairs and to receive final comments and suggestions by the concerned agencies and consultants. These discussions were based on plans to do the work as submitted previously by the engineers. A final action coming out of this meeting was conceptual concurrence, by the several groups represented, that these repairs are technically sound and could go forward subject to the approval of the several governmental agencies involved.

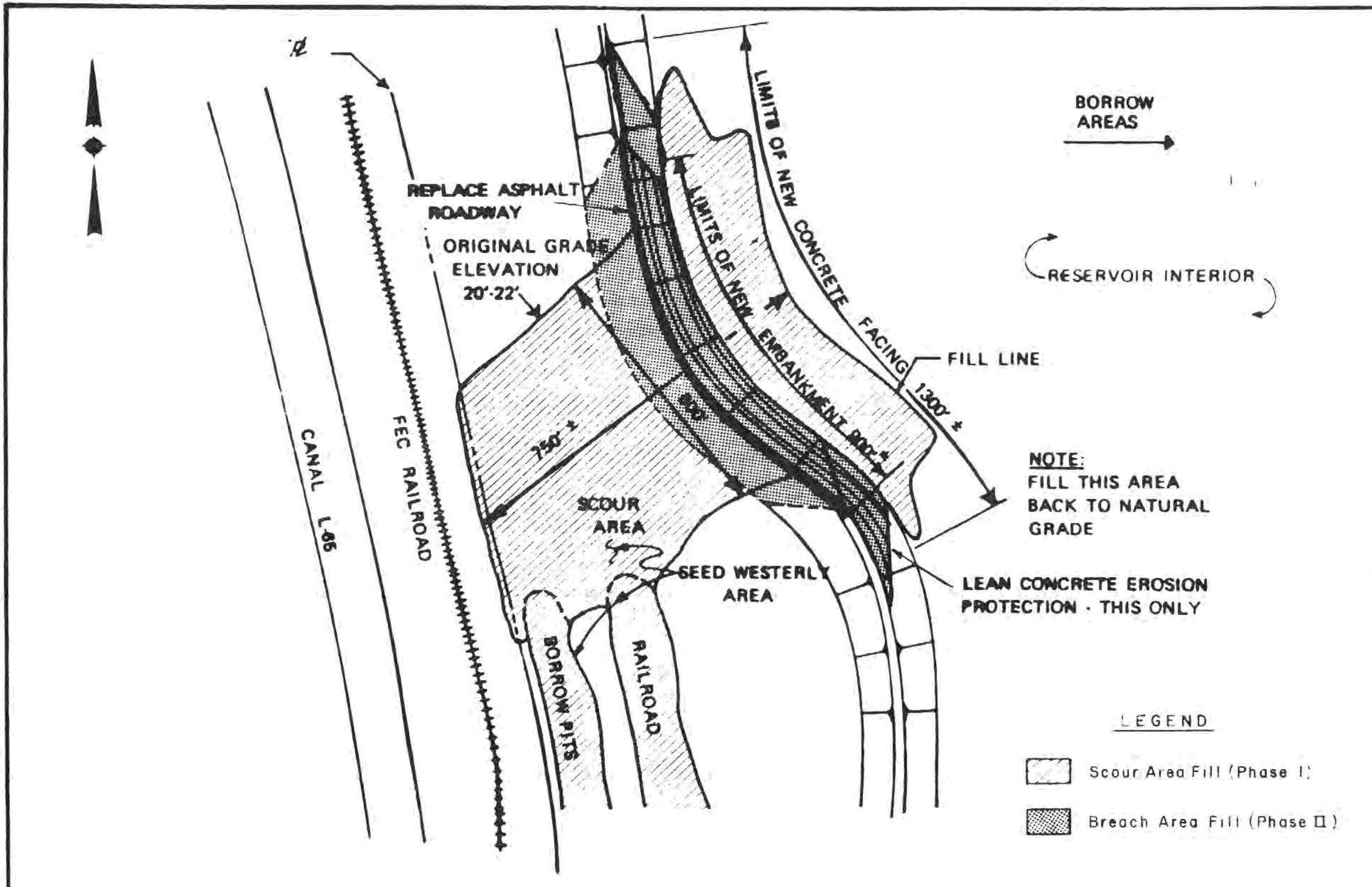
   A careful examination of the embankment early in the investigation revealed that the embankment itself was extremely stable, competent and conservatively designed. On the other hand, we have previously pointed out in this report that the foundation had serious defects which ultimately resulted in the cause of failure.

   FP&L proposed to replace the foundation and embankment in the




## PHASE I AND PHASE II REPAIRS

ADAPTED FROM FPBL REPORT




PHASE I AND PHASE II REPAIRS

breach area with a homogeneous compacted fill which would result in a far superior foundation than that which originally existed. They also proposed to refill the scour area and the old railroad borrow pits with similar material which also would have far greater stability than the original soil configuration. The District Committee agreed with the logic of this reasoning and therefore conceptually endorsed this plan.

On January 11, 1980, the WMD Governing Board on recommendation by staff approved Phase 1 and 2 reconstruction. The approval was formalized by the District with the issuance of the Order Requiring Remedial Measures executed on January 21, 1980.

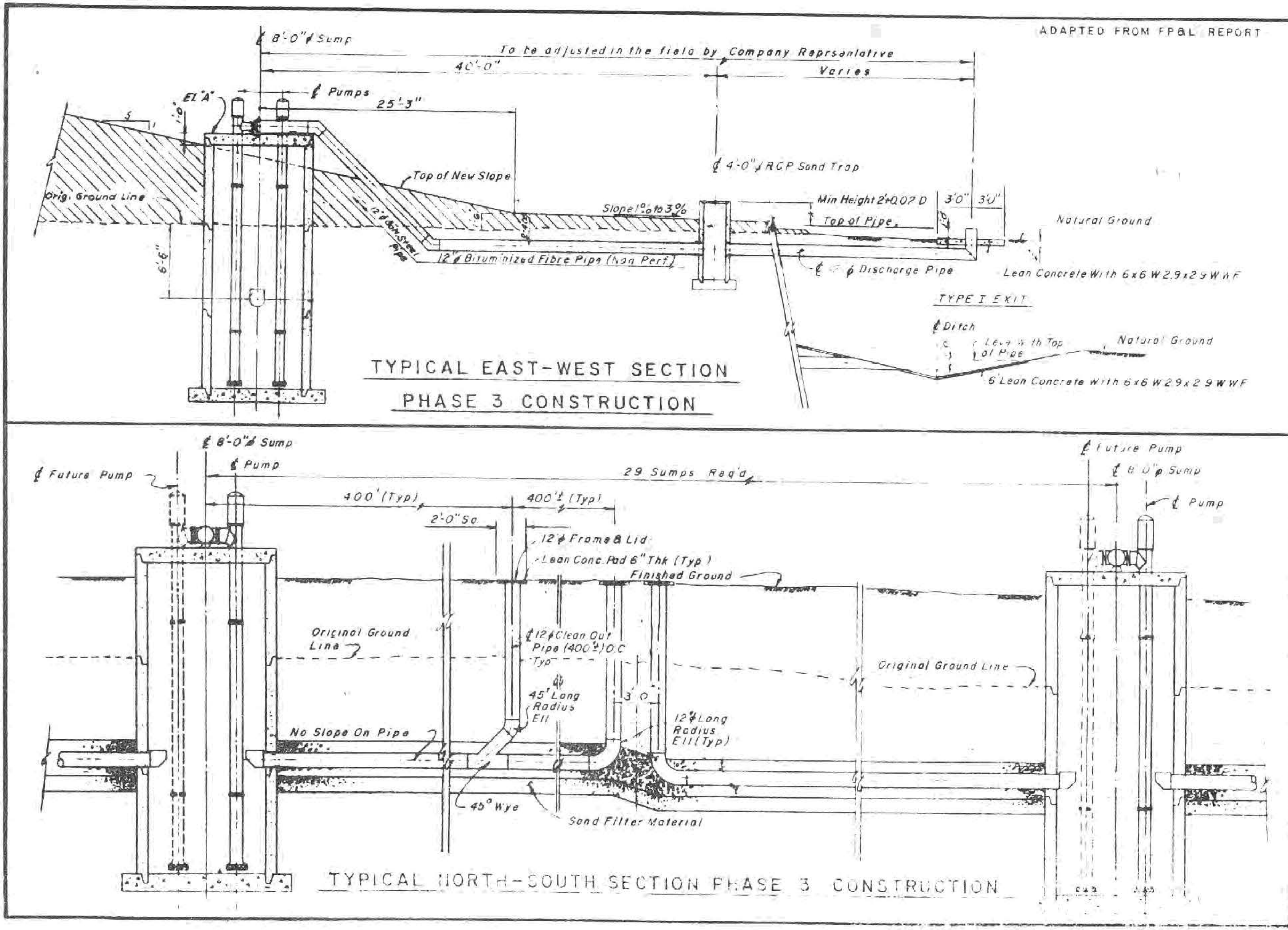
4. A workshop meeting was convened on February 6, 1980 to receive comments regarding the FP&L Consultants Report which a) traced the history of the design and construction and proceeded through to the failure event, and b) detailed the post-failure investigations, and c) presented design criteria and recommendations regarding the details of remedial repairs to the remainder of the reservoir. These proposed remedial repairs were designated as Phase 3.
5. On the basis of the comments and suggestions received, a meeting was convened on February 27, 1980 to finalize design details concerned with Phase 3, incorporating both upstream and downstream repairs.
6. Based on the results of the upstream toe drilling program, FP&L consultants recommended a change in the upstream remedial works. The results of the exploration and the consultants latest revision were presented to the District investigative team in a briefing on April 4 and in a workshop on April 10, 1980.

In an earlier part of this report, a brief summary of Phases 1 and 2 was given. Phase 3, which covers the rehabilitation of the remainder of the reservoir, incorporates those elements of work that are designed to eliminate a failure from occurring as a consequence of any of the failure mechanisms described earlier as "probable". As a consequence of the decision to adopt this philosophy, there is a significant degree of redundancy (safety) built into this remedial work. The results of the investigation as to the cause of failure by FP&L and by the District were not in complete agreement. FP&L presented four "possible" causes of failure, whereas the District postulated but two "probable" causes of failure. The two District causes were included among the four of FP&L. The latter included two additional failure modes, 1) piping emerging at the downstream toe of the dike and 2) piping into the L-65 borrow canal. Since these are indeed "possible" causes, strict semantic usage reveals no disagreement.

The major elements of the remedial work will be described, followed by an explanation of their efficacy in preventing failures from each of the FP&L possible causes. Figures 7-3 and 7-4 present graphically the details of the proposed works.

Upstream Modifications

1. During the course of the post-failure investigation, it was observed that vertical shrinkage cracks of one sort or another had developed over large segments of the soil cement. As originally conceived, the essential function of the soil cement was to stabilize the embankment against erosion and to dissipate the energy created by wind driven wave action. As discussed below, however, it was deemed necessary to make the soil cement as impermeable as practical in certain areas. Thus, part of the remedial work would consist of sealing all significant




TYPICAL EAST-WEST SECTION
PHASE 3 CONSTRUCTION

TYPICAL NORTH-SOUTH SECTION PHASE 3 CONSTRUCTION

Fig. ~~7-5~~ 7-4

vertical cracks and further, of repairing and resealing all horizontal separations between layers of soil cement that occurred as a consequence of the rapid drawdown brought on by the embankment failure. A disagreement exists between District and FP&L as to the efficacy and the necessity for the sealing of the vertical shrinkage cracks. The District position is that the sealing will not be permanant and also that it is not necessary.

2. Throughout significant portions of the reservoir area, numerous relatively impervious and cohesive layers are found at various depths. These layers have important beneficial effects with regard to embankment stability, and a shallow cement-bentonite cut-off wall would be placed at the upstream toe of the soil cement to intersect these shallow layers. Because the deeper cemented shell layer was not uniformly present around the entire reservoir, the vertical cutoff was proposed to be constructed to two different depths. Where the cemented shell is deeper than 20 feet, the depth of the cutoff would be 8 feet; where the cemented shell is shallower than 20 feet, the depth of the cutoff would be 15 feet, as proposed by FP&L. The District did not agree with the shallower cutoff, but believed it should be constructed to a uniform depth of 15 feet. Consequently, the conclusions and recommendations made at the end of the report relate only to this deeper cutoff wall. A typical section through the embankment and details of the upstream cutoff wall as proposed by FP&L are shown on Figure 7-3.

Downstream Modifications

1. The downstream modification would consist of two parts, the addition of embankment of the slope and an extensive pumped toe and chimney drain system. The added embankment would be in the form of a flattening of the slope from 3 to 1 to 5 to 1 below elevation 33 and to 4 to 1 between elevations 33 and 37. The pumped sump system could consist of a horizontal perforated

pipe encased in a graded sand filter. This drain pipe would completely circumscribe the reservoir. At distances varying from approximately 2300-4500 feet and averaging 3300 feet apart, a pumped sump would intercept the drain pipe. Pumps would control water elevations in the sumps between prescribed elevations. The point of discharge would be either to the toe ditch, back into the reservoir or into the Barley Barber Swamp. Associated with the system, a chimney drain extending up the downstream slope would direct water in the embankment and convey it to the collector drain where it would be transported to the pumped sumps. As a final element of this system, a filter fabric would completely surround the collector and chimney drains in order to prevent migration and removal of material from the foundation or drain system. Typical details of this system can be seen in Figure 7-3.

Justification of Modifications

The most probable cause of failure, piping beginning below the surface of the railroad borrow pit and progressing upstream beneath the embankment, can be prevented simply by filling the borrow pit. In fact, this filling has already been completed.

The second probable cause of failure, piping of sands from the foundation downward through the cemented shell layer, can be precluded by preventing a downward hydraulic gradient through this layer beneath the embankment. Several methods can be employed which will help to meet this latter objective. Some of these will also prevent the other two "possible" failure modes postulated by FP&L. The effectiveness of each modification separately and in combination can be seen on the flow nets in the Appendix.

As discussed previously, Case 13 represents what might be termed a potential failure prone condition. The hydraulic gradient is downward under the entire embankment. Case 14 represents the same physical conditions, except that the upstream soil cement has been tied by a cutoff wall to an impervious layer, the upstream soil cement has been sealed, and the railroad borrow pits have been filled. The result of these actions is to move the point at which the gradient changes from downward to upward through the shell (referred to as the point of reversal) from a location downstream of the downstream toe to a location about 30 feet downstream of the centerline of the dike. Moreover, downward gradients through the shell beneath the embankment are substantially reduced.

Case 15 represents the condition with all the remedial works in place and functioning. In this case the point of reversal has been shifted still further upstream. Thus, even for the extreme conditions inferred by the piezometers at station 240, piping through the cemented shell layer will be precluded by the proposed remedial works. As an added redundancy, in case the above described piping should still occur, a tunnel form in the foundation at the base of the embankment, or beneath a shallow impervious layer, the tunnel would be intercepted by the chimney drain, or the embankment might crack and the crack would be interrupted by the chimney drain. The FP&L consultants developed some interesting, if unique, theories concerning the required capacity and stability of this chimney drain. Without going into the details of these theories, they are based on the full reservoir hydrostatic head being applied to the filter fabric on the upstream side of the drain. The resulting high velocity flow from such a condition, it is argued, would wash fine materials onto the surface of the fabric, partially sealing it, whereupon the tunnel or crack would seal. This argument appears reasonable, but not capable of being verified. Since the chimney drain is a seconary or tertiary line of defense, it is considered unncessary to prove its effectiveness.

It is also possible that a tunnel could begin to form beneath a layer immediately below the base of the upstream cutoff wall and progress downstream in the manner described previously. It could not progress beyond the point of reversal because downstream of this point the hydraulic gradient would progress upward. Under such a condition a higher piezometric head would be introduced beneath the upstream portion of the embankment. Under this changed condition, however, the point of reversal would move downstream, permitting the tunnel to progress still further downstream until it was beneath the downstream toe. Calculations indicate a failure condition if this scenario were to occur with the 8 foot deep cutoff and a reservoir at elevation 37. On the other hand, a 15 foot deep cutoff would preclude such a failure.

Piping of the foundation emerging at either the downstream toe of the embankment or in the L-65 borrow canal are the other "possible" failure modes noted by FP&L. The first of these possible types of failure would be prevented by the proposed remedial works, as the hydraulic gradient would be concentrated around the downstream pumped drain pipe and reduced at the downstream toe of slope. The gradient at the L-65 borrow canal is so low that it is inconceivable how this mode of failure could occur. Piping at the drain pipe will be protected by two redundant means -- the graded filter surrounding the pipe, and the filter cloth. Each of these means alone should be capable of preventing piping.

## Future Actions

### Phase Four

Phase 4 comprises the filling of the reservoir. At this point in time, the pool stage of the reservoir has not been fixed. Under any circumstances, all relevant parties to these meetings firmly recognize that any matters concerned with the refilling would require separate application and approval by the WMD Governing Board. As a final note to the entire subject area of reconstruction and refilling, FP&L has consistently stressed the need and urgency of starting to generate power from one unit of their facility around September 1, 1980.

### Monitoring

Since earliest post-failure times, monitoring has always been considered an integral and indeed vital and fundamental part of Phase 3 reconstruction. The District considers it essential that this activity include many items associated with the operation of the facility. Chief among them are:

1. Those parameters related to developing a mass water balance for the reservoir.
2. Water levels in the piezometers
3. Operating characteristics of the pumped sumps
4. Measurements of the volume of sand entering the sumps
5. The results of the periodic inspections (i.e., weekly, monthly, quarterly, etc.).

Because of the complexity of the system and, recognizing that at least initially and extending for some period of time when an operating history will provide high confidence levels, it will be necessary to engage in a substantially more intensive monitoring program. Moreover, some additions and/or modifications to the system initially approved may become necessary. The future configuration of the system and the monitoring schedule will depend on the experience and analysis of data gained during early operational periods. It will be necessary to display these data in a manner that is lucid, succinct and amenable to analysis and correlation. On the basis of all of the above, District staff considers the entire subject area of monitoring to be a "stand alone" item with its own design, justification, and analytical report and to be subject to separate approval. At this point in time the final plans concerned with instrumentation and reporting have not been finalized. The basic elements as described above, however, have been defined. In summary form these consist of the following elements:

1. The various parameters that form the basis of determining inflow - outflow calculations of the reservoir have been shown to be poorly defined or imperfectly calculated. A review of the entire methodology leading to higher confidence levels in this basic equation and/or measurements that form the basis for solving the mass water balance equation is required.
2. A revamping of the program for taking water level measurements in the various piezometers and the maintenance of the piezometers, all of which have been reviewed in an earlier section of this report, will be required.
3. The operating characteristics of the pumped sumps, particularly the timing, frequency and duration, and quantification of the pumping cycle will require careful definition. The quantity of water pumped as measured against time, and the frequency and duration of pumping is a fundamental quantity that reflects gross seepage from the reservoir. A significant anomaly noted in these measurements can be a fundamental and vital factor in assessing impending problem areas.
4. A methodology and program for measuring the volume of sand in the sand traps is considered to be a highly critical factor in determining the efficacy of the downstream drain system. In this regard, a program to flush out and measure sand that may accumulate in the perforated pipe on the downstream drain system will be required.
5. The series of periodic inspections covering all aspects of the system, as described in an earlier section of this report, appears adequate and must be implemented. These inspections

should be continued at the several levels of intensity as specified; thorough and comprehensive periodic investigations and review of these data as well as results of the consultants own inspections should continue at least for some significant period of time after refilling of the reservoir.

As previously indicated, District staff does not consider the entire subject area of a monitoring program to be an essential and inseparable part of the Phase 3 repair work. All of Phase 3 work as outlined above, but omitting the monitoring portion, can proceed without compromising the efficacy of the repair and rehabilitation of the reservoir. We therefore concluded that for several reasons, the monitoring program can be considered as a "stand alone" item of work and that it is not essential to incorporate acceptance of this program along with the remainder of the Phase 3 work.

It is, however, a fundamental necessity to have concurrence and acceptance of every detail of a monitoring program by the District prior to any consideration for placing any water whatsoever in the reservoir. Staff would urge and recommend, therefore, that separate approvals be required for the monitoring program - followed by separate considerations concerned with refilling the reservoir - and that both of these matters, if necessary, can be considered after other elements of the Phase 3 work have been approved.

## CONCLUSIONS

The conclusions that follow are designed to comment on specific elements of the design, construction, proposed rehabilitation and operation of the reservoir. Although deservedly critical in some specific areas, from a broader perspective, we consider the entire project to be conservative in design and construction and well executed.

1. The pre-design exploratory program that formed the basis of reservoir design was comprehensive, and generally well done.
2. Design stability analyses that were performed may not have been realistic in that they did not recognize the potentially critical weak zones in the foundation. However, more carefully developed post failure studies conclusively demonstrated that the several sensitive soil horizons underlying the embankment could not compromise the stability of the reservoir.
3. There appears to have been no demonstrated consideration by the various entities involved in the design as evidenced by the failure to examine in detail the areal extent and depth of the old railroad borrow pits adjacent to the failure area particularly as regards the proximity of the northerly extent of the borrow to the downstream toe.
4. The embankment construction was in general compliance with accepted engineering practice.

5. Construction records, at least those that were available to the District Committee, were not consistently informative.

6. The 250 foot setback from the centerline of the embankment in the reservoir area, as required by the plans, was not always observed,

7. There were several shortcomings in the piezometer network as designed, constructed and utilized:

    a. Gross errors in measuring datum

    b. Incorrect depths

    c. Absence of minimum standards of maintenance

    d. Density of network as installed was not adequate to address its intended use

8. The failure to resolve adequately the problem associated with the wet downstream toe road probably resulted in a less than desirable visual inspection in that area.

9. The format of periodic inspections, however, were generally good.

10. The reporting forms developed by the designers were excellent, however, as implemented, parts of this reporting process were eventually compromised.

11. The post failure investigations were extremely comprehensive and thorough. They were conducted by professionals of the highest caliber and reputation.

12. The District's Investigation Committee received the highest level of cooperation and responsiveness from FP&L and their consultants.

13. The vandalism that occurred at S-153 resulting in a dramatic lowering of L-65 was not related to the failure event.

14. Although generally upward under the downstream half of the embankment, in at least one location a downward hydraulic gradient existed under the entire embankment through the cemented shell layer, which could cause downward migration of sand leading to ultimate failure.

15. A comprehensive and detailed series of flow nets constructed specifically for the failure investigation indicated the existence of potentially critical flow gradients in the vicinity of the downstream toe and the northerly end of the railroad borrow pit.

16. The District concludes that the failure occurred due to one of two mechanisms. However, we agree that the two additional mechanisms proposed by the FP&L Consultants, though unlikely, are considered possible.

17. There are five other mechanisms that were studied and evaluated that are so improbable as to warrant no further consideration.

18. The two failure mechanisms considered to be most probable based on all available evidence are:

    a. A piping failure to the north end of the railroad borrow pit.

    b. Deep piping to the open shell hash resulting ultimately in tunneling of the foundation beneath the embankment, and subsequent blowout at the toe of slope.

19. The other two failure mechanisms advanced by the consultants as being possible include:

    a. Piping of the embankment foundation to the L-65 Canal.

    b. Piping of the shallow embankment foundation emerging at the downstream toe.

20. Repair of the failed area was proposed in two phases by FP&L. This construction will result in an embankment and foundation section superior to any of the rest of the embankment. Repairs consist of:

    a. Placement of fill up to pre-existing natural ground in the breach and scour area with homogeneous material, Phase I.

    b. Reconstruction of the embankment to original specification in the breach area, Phase II.

    This aspect of the reconstruction has been approved by the District and is currently under construction.

21. The plans for the Phase III remedial work as submitted by the FP&L are found to be generally acceptable in concept with one specific exception. The 8 foot depth of the cutoff wall proposed as part of the upstream modification covering portions of the reservoir will not provide the required margin of safety and is therefore not acceptable.

22. Incorporation of the 15 foot depth of the upstream cutoff wall however, will provide the required margin of safety and would be acceptable around the entire reservoir. The design with this exception incorporates several levels of redundancy wherein each element of work acting independently can effectively prevent failure due to several or all of the failure modes identified. Collectively they present a very substantial factor of safety.

23. All exploration work is not yet complete. When all the results of this work are received and evaluated, a lesser requirement for the depth of the upstream cutoff wall may be acceptable, therefore, based on further field data, a selective modification of the depth of this wall may be appropriate.

24. Details of the monitoring network so vital to the future safety of the reservoir have not been completed by FP&L. It is not necessary at this time to consider the entire subject area of a monitoring program to be an essential and inseparable part of the Phase III repair work. Phase III work can proceed without compromising the efficacy of the repair and rehabilitation of the reservoir.

## RECOMMENDATIONS

1. The entire subject area of monitoring is a stand along item with its own design, justification and analytical report and should be subject to approval before approval of Phase IV, filling the reservoir. Therefore FP&L should be required to submit the necessary documentation of a complete monitoring system.
2. Phase III repairs as described in this report should be approved in principal by the District subject to approval by staff of detailed plans and specifications, with appropriate interpertation and application of the results of subsequent field exploration. Staff further recommends that the Executive Director be authorized to issue an order to effect these repairs.
3. Staff should be authorized to continue intensive monitoring of reconstruction to completion to assure that it is done in accordance with plans and specifications and be provided with the authority to implement its decision.
4. Refilling of the reservoir, Phase IV, should be subject to separate District approval and under no circumstances should approval be granted prior to satisfactory compliance with recommendation 1, as specified above, and acceptable completion by FP&L of work in Phases I, II, and III.

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APPENDIX

FLORIDA POWER & LIGHT DIKE

# REVIEW OF SEEPAGE CONDITIONS AND FLOW NET STUDY

By H. R. Cedergren,

Consulting Engineer, Sacramento, California

for

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

West Palm Beach, Florida

January--March, 1980

FLORIDA POWER & LIGHT DIKE

REVIEW OF SEEPAGE CONDITIONS AND FLOW NET STUDY

By H. R. Cedergren, January--March, 1980

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### FLOW NETS

FLORIDA POWER & LIGHT DIKE

REVIEW OF SEEPAGE CONDITIONS AND FLOW NET STUDY

By H. R. Cedergren,
Consulting Engineer, Sacramento, California
for SOUTH FLORIDA WATER MANAGEMENT DISTRICT

January--March, 1980

A. GENERAL BACKGROUND OF PROJECT.

Florida Power & Light Company's Martin County oil-fired power plant project includes a 6700 acre man-made lake for circulation and storage of heated effluent from the plant. With its internal dike system, the reservoir retains the water long enough to cool sufficiently for recirculation in the plant. Since the land in the area is relatively flat (varying from approx. Elev. 20 in the southerly and easterly portions of the reservoir to about Elev. 25 at the northwestern corner) it was necessary to build a completely enclosed dike system 19 miles in circumference to retain the required volume of water. A sand dike was built on the sand foundation containing silt, hardpan, and clayey sand layers, and underlain by several feet of very porous "open-work" shells that overlie very deep beds of shells and sand. A prominent geologic feature of this part of the state is the numerous circular depressions or "sinks" often several hundred feet across, which may be formed in part by concentrated downward seepage of the water trapped in the sinks, with possible piping of the fine soils downward into large pore spaces in the underlying porous shells.

Water was first put into the completed reservoir (constructed between April, 1974 and July, 1977) on February 6, 1978, reached the minimum operating level of Elev. 31 on March 7, 1978, and reached the maximum design operating level of Elev. +37 on April 3, 1978. On the day it failed (Oct. 30, 1979), the water stood at Elev. 37.24, being a little higher than normal as a result of Hurricane David and heavy rains. It stored 80,700 acre-ft. of water at the time it failed.

During the 18-month period in which the reservoir water level stood at or near its planned maximum design level of Elev. +37, appreciable seepage emerged from downstream toe areas (and beyond) and was being observed regularly by F.P.& L. employees who drove the length of the dike and commented upon any "unusual" conditions noted. Total seepage losses prior to the failure were estimated to be in the order of 100 cfs, although reliable estimates were not possible because there was no way to directly collect and measure the seepage, and it had to be estimated from rates of fall of the reservoir with allowances being made for rainfall, evaporation, etc.

In addition to the visual observations of downstream toe areas, readings of water pressures or levels were made periodically in families of 7 to 9 piezometers at seven locations along the 19-mile dike.

Except for a sudden 5-ft lowering of Canal L-65 caused by an act of vandalism three days before the failure, no "unusual" conditions had been reported before the failure. In fact, the toe areas at the failure location had been consistently reported as being drier with less seepage than most areas along the dike.

An old railroad sand borrow pit that extended within a moderate distance of the toe of the dike at the failure point was considered a possible factor contributing to the failure.

Without any advance warning of impending trouble, some time late in the evening of Tuesday, October 30, 1979, a portion of the cooling water reservoir dike began to fail, releasing several billion gallons of water towards the Lake Okeechobee areas of Martin and Okeechobee Counties, causing substantial physical damage to South Florida Water Management District project facilities and other public and private properties and facilities.

Since there is no record of anyone having seen the failure take place, and the failure washed out all evidence of the specific mechanisms of the failure, F.P.& L.'s engineers and their consultants speculated on the most probable cause of failure. They agreed, however, that in all probability it was a piping failure of some kind. They presented five of the most probable types of failure they felt might have occurred.

B. PERTINENT SOIL CONDITIONS.

Figure 1 of F.P.& L's Board of Consultants Dec. 15, 1979 "Report to Florida Power & Light on Martin Cooling Water Reservoir" represents a soil profile at the failure area. It depicts a top layer (2-3-ft thick) of loose white sand over the so-called "hardpan" layer of somewhat compact silty sand (3-4-ft thick), resting on alternating beds of cohesionless to moderately cohesive sands, silty sands and clayey sands for an additional thickness of about 20 feet; a layer of "green" silty sand--very sensitive to shaking and very non-resistant to piping (3-4-ft thick). These sandy and silty formations rest on a thin layer of cemented shells containing vertical tubes or holes, over several feet of very porous shells and shell fragments, that in turn rest on very deep beds of shells and sand.

The dam was constructed with local sandy materials from borrow pits in the reservoir and is a "homogeneous" unzoned dam with an inspection trench along the dam centerline excavated through the upper sand layer to the "hard-pan", or to a depth of three feet, whichever is shallower. It is a sand dam on a sand foundation. The dam had no filter, and the intent was that there would be considerable leakage, and that a monitoring program was envisioned, and was actually carried out.

C. DISCUSSION OF SEEPAGE CONDITIONS.

In the 18-month period during which the reservoir contained water at or near its design level (Elev. +37), the downstream toe and areas beyond were saturated, and "pin boils" and "sand boils" were common occurrences along most of its 19-mile length.

To try to obtain a better understanding of the nature of the seepage conditions in and under the dam, the writer constructed a number of flow nets for basic soil conditions projected into a typical cross section with the information on soil layer permeabilities and depths provided by the South Florida Water Management District engineers and their consultant, Mr. W. Clevenger. They are presented here in the hope they will help those studying the project to have a better appreciation of the nature of the seepage behavior. Flow Nets Nos. 1-20 are given in this report (reduced to half-size) for the conditions summarized in Table 1. Among these flow nets are three (Nos. 1, 3, and 13), which represent potential flow conditions in and near the failure or at other locations around the reservoir. The very complex soil conditions were simplified somewhat to contain five distinctly different horizontal layers as given in Table 2. A range of soil conditions and design features are represented in the flow net study.

Table 1

FLOW NET SUMMARY--FLORIDA POWER & LIGHT DIKE

| Flow net No. | Open sand pit? YES | Open sand pit? NO | Upstream hardpan? YES | Upstream hardpan? NO | Toe drain? YES | Toe drain? NO | Upstream imperv. blanket? YES | Upstream imperv. blanket? NO | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|
| 1 | X | | X | | | X | | X | Very high saturation levels in dam with concentrated uplift pressures and seepage at downstream (d.s.) toe areas; large downward gradients under upstream half of dam (to 200% gradients); fairly large upward gradients in downstream (d.s.) areas (to 80%). |
| 2 | | X | X | | X | | | X | Reduced saturation levels in d.s. part of dam; slightly reduced downward gradients under upstream (u.s.) parts of dam (to 180%); slightly lower upward gradients in d.s. areas (to 60% gradients). |
| 3 | X | | | X | | X | | X | Very high saturation levels in dam with concentrated uplift pressures and seepage at d.s. toe areas (to 100%); slightly lower downward gradients under upstream half of dam (to 140%). |
| 4 | | X | | X | X | | | X | Reduced saturation levels in d.s. parts of dam; slightly smaller upward hydraulic gradients at downstream toe (to 90%); and slightly smaller downward gradients under u.s. part of dam (to 130% gradients). |
| 5 | | X | X | | X | | | X | Same as No. 2, but drain not pumped. Slightly greater upward gradients at downstream toe and canal bottom; raised saturation levels in dam. |
| 6 | | X | X | | X | | | X | Same as No. 2, but canal at Elev. +16 (3-ft drawdown). Increased upward hydraulic gradients in canal bottom. |
| 7 | | X | | X | X | | | X | Same as No. 4, but drain not pumped. Slightly greater upward gradients at downstream toe areas; raised saturation in dam. |
| 8 | | X | | X | X | | | X | Same as No. 4, but canal at Elev. +16 (3-ft drawdown). Increased upward hydraulic gradients in canal bottom. |
| 9 | | X | X | | X | | X | | Addition of impervious upstream facing and blanket greatly lowers saturation levels in dam and d.s. upward gradients; virtually eliminates downard hydraulic gradients under dam. |
| 10 | | X | X | | X | | X | | Same as No. 9, but toe drain not pumped. Moderate rise of saturation levels in dam, and slightly greater upward gradients at downstream toe areas. |
| 11 | | X | X | | X | | X | | Same as No. 9, but canal at Elev. +16 (3-ft drawdown). Increased upward hydraulic gradients in canal bottom. |
| 12 | | X | X | | X | | X | | Same as No. 9, but drain not pumped and canal at Elev. +16. Slightly increased upward hydraulic gradients at downstream toe areas and canal bottom. |

(Continued)

Table 1

FLOW NET SUMMARY--FLORIDA POWER & LIGHT DIKE (CONTINUED)

| Flow net No. | Open sand pit? YES | Open sand pit? NO | Upstream hardpan? YES | Upstream hardpan? NO | Toe drain? YES | Toe drain? NO | Upstream imperv. blanket? YES | Upstream imperv. blanket? NO | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|
| 13 | X | | X | | | X | | X | This flow net was made to conform to a differential head of 2.5 feet on the cemented shell layer under the downstream toe; otherwise conditions are similar to those for Flow Net No. 1. The flow conditions are generally similar to those in Flow Net No. 1, except for the concentrated downward hydraulic gradients in the cemented shell layer under the downstream toe. Downward hydraulic gradient in cemented shell layer is about 1.3. |
| 14 | X | | X | | | X | Soil cement facing sealed and tied to "hardpan" with narrow cutoff. | | Upstream seal of soil cement slope protection layer and tying the soil cement to the "hardpan" layer by means of narrow cutoff eliminates downward gradients in cemented shell layer. |
| 15 | | X | X | | X (pumped) | | Same as for No. 14 | | Adding pumped toe drain further improves seepage conditions in foundation and in dam. |
| 16 | | X | X | | X (unpumped) | | Same as for Nos. 14 & 15 | | Not pumping toe drain allows slightly higher saturation level in dam and slightly higher upward gradients downstream from the dam. |
| 17 | | X | | X | (pumped) | | 65 ft wide | | Conforms to planned construction in locations where dike is on land at about Elev. +20 and there is no hardpan upstream. |
| 18 | | X | | X | (not pumped) | | 65 ft wide | | Same as No. 17, but toe drain is not pumped. |
| 19 | | X | | X | (pumped) | | No treatment | | Pertains to areas where ground level (original) is at Elev. +27, and no upstream blanket or slurry wall is planned, but soil-cement riprap is assumed to be sealed; and a toe drain will be constructed. |
| 20 | | X | | X | (not pumped) | | No treatment | | Same as No. 19, but toe drain is not pumped. |

NOTE: Flow nets Nos. 17-18 were developed for features incorporated in design after Work Shop held at site on February 6, 1980.

Table 2

Soil Permeabilities and Thicknesses Assumed for Flow Nets

| Layer | Description | Rel. k | Thickness | Elevations |
|---|---|---|---|---|
| 1st | fine sand | 10 | 3 ft | +20 to +17 |
| 2nd | "hardpan" | 0.01 | 4 ft | +17 to +13 |
| 3rd | silty sands, fine sands, etc. | 1.0 | 20 ft | +13 to -7 |
| 4th | cemented shells with some openings | 0.01 | 2 ft | -7 to -9 |
| 5th | porous shells | 10,000 | 10 ft | -9 to -19 |

The compacted sand embankment was assigned a relative permeability value of 0.1 (in relation to the 3rd foundation layer, the silty sands, etc., with an assumed average permeability of $1 \times 10^{-4}$ cm/sec, and a relative value of 1.0.)

Two initial flow nets that were constructed for potential conditions prior to the failure were developed in such a way as to make their equipotential lines conform to the average pore pressures recorded in the piezometers at Stations 170, 240, 460, and 670, which were all relatively similar in height of dam and levels of pore pressures measured. Those at other locations were not included in the "averages" used, either because the dam height was less, or the downstream piezometers were giving lower readings than the general levels being recorded at those four locations. These two flow nets, Nos. 1 and 3, established the basic flow conditions that influence all of the flow nets.

Table 3 gives piezometer readings at Stations 170, 240, 460, and 670 on July 17, 1978 (soon after initial stabilization), July 5, 1979 (an intermediate time), and October 10, 1979 (just prior to the failure). It also gives averages of the three times for all piezometers for each location, and averages of the readings at the four locations.

Since the piezometer readings provided the primary control over equipotentials, the flow nets do not necessarily conform exactly to the assumed relative *k* values given in Table 2.

Table 3

SELECTED PIEZOMETER READINGS--FLORIDA POWER & LIGHT DIKE

| Location | Date | A | B | C | D | E | F | G | H | I | NOTES |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sta.170 | 7/17/78 | 35.5 | 30.45 | 30.4 | 26.95 | 25.85 | 23.5 | 25.25 | - | - | |
| | 7/5/79 | - | - | 30.68 | 27.11 | 26.2 | 24.03 | 25.64 | - | - | |
| | 10/10/79 | - | - | 30.6 | 26.9 | 25.7 | 22.9 | 24.9 | - | - | |
| | Average | 35.5 | 30.5 | 30.6 | 27.0 | 26.0 | 23.5 | 25.2 | - | - | |
| Sta.240 | 7/17/78 | 34.25 | 29.4 | 28.55 | 26.9 | 27.3 | 24.55 | 24.8 | - | - | |
| | 7/5/79 | 34.33 | 29.36 | 28.52 | - | 27.18 | 24.39 | 24.68 | - | - | The height of the dike is approximately the same at Sta.170, Sta. 240, Sta. 460, and Sta. 670, and the readings are relatively consistent. |
| | 10/10/79 | 34.4 | 28.6 | 28.3 | - | 26.8 | 23.7 | 24.1 | - | - | |
| | Average | 34.3 | 29.0 | 28.4 | 26.9 | 27.0 | 24.0 | 24.5 | - | - | |
| Sta.460 | 7/17/78 | 34.0 | 27.85 | 29.6 | 26.45 | 26.1 | 23.35 | 24.9 | 22.1 | 21.05 | |
| | 7/5/79 | 34.88 | 27.90 | 27.79 | 26.45 | 26.00 | 23.32 | 24.94 | 22.12 | 21.17 | |
| | 10/10/79 | 35.0 | 28.1 | 26.8 | 26.5 | 26.0 | 23.3 | 24.9 | 25.5 | 21.0 | |
| | Average | 35.0 | 28.0 | 27.0 | 26.5 | 26.0 | 23.3 | 24.9 | 22.3 | 21.1 | |
| Sta.670 | 7/17/78 | 34.6 | 26.9 | 27.8 | 27.95 | 26.95 | 22.55 | 25.1 | 24.1 | 22.65 | |
| | 7/5/79 | 34.7 | 27.49 | 28.33 | 27.42 | 27.5 | 22.80 | 25.31 | 24.30 | 22.73 | |
| | 10/10/79 | 34.6 | 27.2 | 28.1 | - | 27.2 | 22.1 | 25.0 | 24.0 | 22.6 | |
| | Average | 34.6 | 27.2 | 28.2 | 27.5 | 27.3 | 22.5 | 25.2 | 24.1 | 22.7 | |
| AVERAGE OF ABOVE | | 35.0 | 28.7 | 28.6 | 27.0 | 26.6 | 23.3 | 25.0 | 23.3 | 22.0 | These values used in the flow net study. |

NOTE: Piezometer readings at Sta. 60 and Sta. 800 are generally higher than at the four above locations; however the ground elevation is about 5 feet higher at these two locations than at the others. Readings at downstream toe areas in the piezometers at Sta. 370 are about 2 ft lower than those at the four locations, and were not included. Water level readings in A piezometers at Sta. 170, Sta. 240, and Sta. 670 may only represent the bottoms of these piezometers; however the one at Sta. 460 has definite readings above its bottom, so these readings may represent the water level.

By examining the flow nets it can be seen that the "porous shell" layer between Elev. -9 and -19 has a large influence on the seepage patterns. Because of its very high relative permeability, the shell layer causes a pronounced downstream spreading of uplift pressures, as is evidenced by equipotential line No. 1 which goes beyond canal L-65. Also, the emergence of seepage out of the lower part of the downstream slope of the dam, together with the general shape of flow nets Nos. 1 and 3, suggests that saturation may have risen to relatively high levels in the dam. Such a rise could be caused by the free flow of water under the upstream soil-cement facing that could produce nearly full reservoir head under the upstream half of the dam, irrespective of whether the soil-cement is pervious or impervious.

In order to show the possible influence of the upstream "hardpan" layer, Flow Net No. 1 (sheet 1) was constructed for the assumption that such a layer extends to the borrow pit 250 feet upstream from the centerline of the dam, and Flow Net No. 3 (sheet 1) was developed for the assumption that this layer does not exist.

Flow Nets Nos. 1 and 3 indicate that very large downward gradients could have existed in the hardpan layer under the upstream half of the dam (up to 140 to 200 percent), with smaller downward gradients in this layer extending appreciable distances downstream from the centerline of the dam. These conditions are with no downstream toe drain or other control measures installed. Also, rather large concentrations of seepage and uplift pressures could have existed at downstream toe areas, and as already noted, rather high saturation levels in the dam. Exit gradients at the sand borrow pit at the failure area were in the order of 70% with the hardpan layer extending 250 feet upstream from the dam centerline, and 80% without the hardpan upstream from the dam. Uplift gradients in the shallow drainage ditch just beyond the toe are in the range of 80 to 100 percent (larger at this point than in the borrow pit because of the flow-retarding effects of the hardpan layer). And, at the L-65 canal, uplift gradients could have been in the order of 45 percent to 50 percent, with its water level at Elev. +19, and up to 80 percent with the water level in the canal down to Elev. +14, as was caused by the sabotage action.

The flow nets also show that without any new remedial measures installed, significant downward gradients could have existed in the soil formations above the "porous shell" layer under at least the upstream half of the dam.

The hydraulic gradients indicated at the sand borrow pit are those obtained from flow nets for two-dimensional flow, whereas actual gradients at this location (and similar ones) would be substantially increased by converging three-dimensional flow; hence the real conditions were probably more severe than those indicated by the flow nets. The actual distance from the toe of the dam to the sand borrow pit seems to be unknown, and this could also have an effect on the actual gradients before the failure.

Flow Nets Nos. 2 and 4 show potential comparable conditions (relative to Nos. 1 and 3), with the F.P.& L.'s Consultants' proposed toe drain and berm in place, being pumped as they propose. To show the potential value of pumping the toe drain, Flow Nets Nos. 5 and 7 are with the toe drain installed but <u>not being pumped</u>, and Flow Nets Nos. 6 and 8 are with the L-65 canal down at Elev. +16, as may occur during extended dry periods.

By examining Flow Nets Nos. 2, 4, and 5 to 8, it is seen that the toe drain can effectively lower saturation in the downstream part of the dam (less when the drain is not pumped), but the drain would apparently have little influence on downward gradients in the hardpan layer under the upstream part of the dam, or in the cemented shell layer under the formations upstream from the dam. In developing the flow nets with the toe drain in place, it is assumed that there will be no un-filled sand borrow pits downstream from the dam. Eliminating borrow pits results in a "trapping" of water that could have been escaping more freely into open pits excavated below the "hardpan" layer; hence uplift pressures downstream from the dam are increased. For this reason, flow nets constructed with the hardpan layer downstream from the dam show larger uplift gradients in downstream areas than would develop without the trapping effect of the hardpan. Even with the toe drain installed, upward gradients in these areas are moderately large (ranging from 60% to 100%).

Because of the large downward gradients the flow nets indicate might exist in critical places with the new toe drain in place, several flow nets were constructed (Nos. 9-12) to show the possible benefits of placing a very impermeable facing on the dam, and a highly impermeable blanket on the reservoir floor to a significant distance upstream from the dam. They assume that the upper clean sand layer is removed before the blanket is placed, and that a nearly totally impermeable blanket is constructed. For these conditions it is seen that this treatment, when added in addition to the proposed toe drain, virtually eliminates downward gradients under the dam. It would slightly reduce upward gradients downstream from the dam. Water levels within the dam would be substantially lowered.

A few individual readings in piezometers D and E, near the downstream toe, indicated the possibility that rather substantial downward gradients could exist across the cemented shell layer below the downstream toe of the dam. Since this is an undesirable condition, Flow Net No. 13 was developed for the assumption that a 2.5-ft differential downward head exists across the cemented shell layer under the downstream toe. Also, Flow Net No. 14 shows the benefits of an impervious cutoff tied into an upstream hardpan layer and into a sealed soil-cement riprap, and Flow Nets Nos. 15 and 16 show the added value of a downstream drain, pumped and not pumped. Using the average piezometer values in Table 3, a downward head of 0.4 ft. had been used in developing the initial flow nets (Nos. 1 and 3), but this represents a less severe condition than is depicted by Flow Net No. 13.

During the Work Shop held at the site on February 6, 1980 the South Florida Water Management District's consulting panel recommended that in addition to the proposed toe drain (modified somewhat from initial designs) upstream cutoff or blanketing systems be used to provide added factor of safety because of the unusual conditions at this project. Also they had recommended sealing of joints and cracks in the soil-cement riprap to reduce flow into the dam. The Plans and Specifications dated March 5, 1980 incorporate both of these ideas. At locations where a substantial hardpan or clay layer exists under the top sand layers, a slurry wall cutoff will be constructed into the hardpan or clay layer, and tied into the soil-cement with a water tight membrane. Flow nets Nos. 15 and 16 approximately represent this condition. By examining Flow Nets Nos. 1, 2, and 14-16 it can be seen that the presence of a substantial hardpan layer under and upstream from the dam has a generally beneficial effect in moving the location of downward gradients in the cemented shell layer a substantial distance from the upstream toe of the dam, so that if any cavities did form they would not endanger the foundation of the dam. The conditions represented by Flow Net No. 13 have already been noted. Also, the added benefits of a wide impermeable blanket as illustrated by Flow Nets. Nos. 9-12 have already been discussed.

In areas where the upstream hardpan is missing, large downward gradients in the cemented shell layer over the very porous shells tend to develop under the upstream toe as illustrated by Flow Nets. Nos. 3, 4, 7, and 8. In such areas (which will be located by an extensive augering program) the Plans and Specifications dated March 5, 1980 call for the construction of a 65-ft wide impermeable membrane. Flow Nets Nos. 17 and 18 depict the potential seepage conditions at such areas, with this treatment in addition to the planned toe drain. It can be seen that substantial downward gradients (around 170 to 180%) have been estimated from the flow nets (see Table 4), but they are shifted upstream by about the width of the membrane.

The Plans and Specifications dated March 5, 1980 indicate that in areas where the adjacent ground level is at or above Elev. +27, F.P.& L.'s designers do not plan to require an upstream slurry wall cutoff or upstream impervious membrane. At such locations, where a substantial upstream hardpan layer exists, the hydraulic gradients can be approximated by multiplying those in Flow Nets Nos. 5 and 6 by 60% (the ratio of 10 feet of head to 17 feet of head). To obtain an indication of possible conditions at locations where the ground level is at Elev. +27 and there is no upstream hardpan layer, Flow Nets. Nos. 19 and 20 were developed. By examining these flow nets and Table 4 it can be seen that gradients in downstream areas are generally moderate, but large downward gradients can still exist in the cemented shell layer under the upstream toe. At locations where the ground is at or near Elev. +20, as represented by Flow Nets.Nos. 7 and 8, these gradients were in the order of 280 to 300%. Those given in Table 4 for Flow Nets Nos. 19 and 20 indicate gradients in the range of 160 to 180 %, which is about 58% of those where the ground is at Elev. +20. As already noted, they might have been expected to be around 60% of those at the locations with lower ground elevations. In view of these studies it would seem prudent that some kind of upstream treatment be considered for the entire dike system.

Table 4
FLORIDA POWER & LIGHT DIKE
SUMMARY OF ESTIMATED HYDRAULIC GRADIENTS
FROM FLOW NET STUDY

| Flow Net No. | u.s. hardpan | | Open d.s. pits | | d.s. land Elev. | Upstream blanket or cutoff | Sealed soil-cem. | | d.s. toe drain | | | Downstream canal | | | Est. Max. downward gradient in shells$^{x}$ | | Est. Max. upward gradient at d.s. toe area - % |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Yes | No | Yes | No | | | No | Yes | None | pumped | Not pumped | Yes | water level | No | upstream % | downstream % | |
| 1 | X | | X | | +20 | None | X | | X | | | X | +19 | | 150 | - | $80^{++}$ |
| 2 | X | | | X | +20 | None | X | | | X | | X | +19 | | 120 | - | $60^{++}$ |
| 3 | | X | X | | +20 | None | X | | X | | | X | +19 | | 200 | - | $100^{++}$ |
| 4 | | X | | X | +20 | None | X | | | X | | X | +19 | | 300 | - | $90^{++}$ |
| 5 | X | | | X | +20 | None | X | | | | X | X | +19 | | 140 | - | $70^{++}$ |
| 6 | X | | | X | +20 | None | X | | | X | | X | +16 | | 150 | - | $70^{++}$ |
| 7 | | X | | X | +20 | None | X | | | | X | X | +19 | | 280 | - | $100^{++}$ |
| 8 | | X | | X | +20 | None | X | | | X | | X | +16 | | 300 | - | $90^{++}$ |
| 9 | X | | | X | +20 | 160' bl. | | X | | X | | X | +19 | | 150 | - | $40^{++}$ |
| 10 | X | | | X | +20 | 160' bl. | | X | | | X | X | +19 | | 150 | - | $50^{++}$ |
| 11 | X | | | X | +20 | 160' bl. | | X | | X | | X | +16 | | 150 | - | $60^{++}$ |
| 12 | X | | | X | +20 | 160' bl. | | X | | | X | X | +16 | | 150 | - | $70^{++}$ |
| 13*** | X | | X | | +20 | None | X | | X | | | X | +19 | | 180 | 125 | $60^{++}$ |
| 14 | X | | X | | +20 | Cutoff | | X | X | | | X | +19 | | 170 | * | $50^{++}$ |
| 15 | X | | | X | +20 | Cutoff | | X | | X | | X | +19 | | 170 | - | $15^{+}, 70^{++}$ |
| 16 | X | | | X | +20 | Cutoff | | X | | | X | X | +19 | | 160 | - | $20^{+}, 80^{++}$ |
| 17** | | X | | X | +20 | 65' bl. | | X | | X | | X | +19 | | 180 | - | $18^{+}, 60^{++}$ |
| 18** | | X | | X | +20 | 65' bl. | | X | | | X | X | +19 | | 170 | - | $24^{+}, 80^{++}$ |
| 19** | | X | | X | +27 | None | | X | | X | | | | X | 180 | - | $12^{+}, 50^{++}$ |
| 20** | | X | | X | +27 | None | | X | | | X | | | X | 160 | - | $13^{+}, 60^{++}$ |

+20, +27, etc. = Elevations; ++ = with hardpan layer downstream from dam; + = no hardpan downstream from dam.

x Refers to cemented layer over open-graded shells.

** Represents the sections prepared for construction.

*** This flow net assumes large vertically downward gradient in cemented shell layer under downstream toe area.

* Sealed soil-cement riprap and upstream cutoff eliminates the downward gradients under the downstream toe area.

NOTE: All gradients are approximate, and are presented to facilitate comparisons; actual gradients will depend on specific conditions at each location.

D. DISCUSSION OF POSSIBLE FAILURE CAUSES.

The December 15, 1979 "Report to Florida Power and Light Company on Martin Cooling Water Reservoir," said that "After review of all the facts available, no evidence was found to support any mechanism of failure, the specific cause of which cannot be determined even after extensive investigation. It is especially difficult to understand the failure developing after 18 months of operation." The report previously said, ". . . the seepage conditions through and beneath the dike stabilized within several months after filling." Presumably the board had in mind the pore pressures being recorded at seven locations along the 19-mile dike in families of piezometers, and general seepage and saturation at and beyond the downstream toe as being observed by project personnel. It is understood that little if any change in toe seepage conditions were noted in the observational program after the first several months.

After some speculation regarding possible effects of sabotage that led to the lowering of the L-65 Canal three days before the failure, potential backward erosion starting at an initial concentrated leak, the liquefaction potential of the "green" silty sand layer over the cemented shells, and some other comments, five most likely modes of failure were selected and discussed. These discussions are summarized here, with some comments that appear appropriate in the light of the flow net solutions.

Possible Failure Type 1. "Piping (backward progressive erosion) of an initial leak just below the dike, in layers A or C, emerging at or near the downstream toe." This mode was considered unlikely anywhere around the dam because seepage velocities were considered too low, and even less likely at the failure location because "the seepage emerging at the toe was considerably less than the average around the reservoir." The drawdown of the L-65 Canal was felt to have had little or no influence on this kind of failure.

(According to Flow Nets Nos. 1 and 3, large downward gradients (up to 200%) could have existed in the hardpan layer under upstream parts of the dam, and upward gradients of 80% to 100% could have existed at the shallow ditch below the dam. These gradients are all large enough to induce soil movement.)

Possible Failure Type 2. "Piping (backward progressive erosion of an initial leak) emerging unseen below the water in the pond created by the old railroad borrow area excavation." The report said that such erosion "could have been going on without being observed" and "The location of the breach is one of the few locations around the length of the reservoir where there is such a pond with any appreciable depth within a few hundred feet The report said though that "such a failure mechanism is difficult to explain, because of the considerable distance and low hydraulic gradients existing."

(While it is true that the "average" gradients between the reservoir and the borrow pit were rather small, Flow Nets Nos. 1 and 3 show that when consideration is given to the possible influence of the extremely permeable shells under the other layers, exit gradients of 80% or more could have existed at the borrow pit, suggesting that moderate amounts of cohesionless sands or silts might have been emerging unseen into the borrow pit because of the heavy growth of vegation).

Possible Failure Type 3. "Piping (backward erosion of an initial leak unseen into L65 Canal)". Because of the great distance from the downstream toe of the dam to the canal, it was felt "inconceivable that backward erosion of a leak traveling all the way to the canal could threaten the dam."

(According to the flow nets, upward exit gradients of around 50% might have existed in the canal bottom under normal conditions with its water level at Elev. +19, and around 70 to 80% at the drawn-down level of Elev. +14 which occurred several days before the failure, suggesting that some piping of soil into the canal might have been possible.)

Possible Failure Type 4. "Sand above the layers of shells and cemented shells could migrate into open voids in the shell layers by downward traveling seepage, causing a loosening of the sand supporting the dam, localized settlement, arching of the dam embankment over the area of settlement and a concentrated leak below the arch emerging at or near the downstream toe, rapidly eroding to failure."

This type of failure was considered because of the large particle-sizes of the shells with respect to the average sand particles, and the tendency for downward seepage at the upstream toe of the dam through the upper sand foundation layers to the shell layer, with sand entering voids in the shell layer. But this kind of failure was considered very unlikely because "most of the voids (in the shells) are already filled with sand." And, "once the sand enters the shell voids by downward migration, it cannot be moved very far within the shells since the velocity of seepage water within the shells is too low to cause the sand to move horizontally." It was suspected that this failure hypothesis could accommodate the trigger action of the lowering of the L-65 canal because of a reduction in water pressure in the shell layer under the upstream slope of the dam, increasing downward seepage gradient and restarting the migration of sand into shell voids. An argument against this kind of failure was said to be the lack of obvious sink holes in the reservoir floor just upstream from the dam toe (at locations other than the failure) where they should also have been expected. Also, the report said there was no apparent settlement of the dike crest.

(Under the unstable seepage conditions created by the reservoir water, it seems possible that appreciable amounts of sand could have been piping downward, provided there was sufficient volume of unfilled voids; however this volume seems to be quite unknown. An argument for this kind of failure might be the concept that under reservoir head, unseen tubes or "pipes" might have been forming in horizontal directions between the upstream toe of the dam and downstream areas, and that when a group of these tubes became sufficiently continuous a through leak developed and rapidly expanded into the breach. While such tendencies may have been quite dormant under the flow conditions produced by rainfall on the area before construction of the reservoir, the added head of 15 feet or more with an unlimited supply of water available to enter any openings in the bottom of the reservoir, it could conceivably set new migration and piping actions in motion. According to the flow nets, horizontally directed gradients in the shell layers would be very low, and it is hard to understand how large amounts of soil could move horizontally in the shells unless some kind of pulsating or liquefying actions were taking place because of intermittent build-up of head at a given location to the point where it could move a slug of soil, with periodic repetition of the action over the 18-month period.)

Possible Failure Type 5. "The green silty sand directly above the shells is an unusual material. It may be in a very loose state" with very low shear strength. An increase in effective stresses in the green silty sand layers causing a collapse of the soil structure and "a sudden increase in pore water pressure and decrease in shearing resistance of this layer" might have reduced the safety factor against a downstream block-type slide to near 1.0, producing a crack through which a large concentrated leak developed through the dam, leading to rapid erosion and formation of the breach. This type of failure was considered less likely than some of the others "because the suspect layer is so deep and such a sliding failure in a sand dam 30 ft high on a sand foundation, with 3:1 slope would be unprecidented."

(Because of the extremely sensitive and erodible nature of this formation, it seems possible that extensive "tubes" might have been eroding in it at numerous places around the reservoir over the 18-month period, and that such tubes in the failure area might have in some way contributed to the failure.)

E. SUMMARY AND RECOMMENDATIONS.

One of the most plausible failure mechanisms might have been a combination of collapse and piping actions leading to formation of a continuous opening from the toe area or beyond (possibly at the borrow pit) to the reservoir (Possible Failure Types 2 or 4). If such a tube did form gradually over the 18-month period of water storage, even a small initial leak that started through such a "tube" might have rapidly enlarged to form the breach (possibly aided by some kind of trigger action, as has been discussed by F.P.& L.'s Consultants). Support for this kind of failure mechanism is the extremely erodible nature of most of the foundation soils (see Photographic Supplement to this report for typical evidence of soil erodibility). Openings might have been forming while the eroded or collapsed soil was disappearing into underlying shell voids, or emerging and remaining undetected in the borrow pit or other downstream areas around the reservoir because of the thick vegetation covering many of these areas. Large volumes of soil would not necessarily have had to be removed to allow such actions to have been going on without detection. A 2-in. diameter tube, for example, would

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MAPS

Sheet 5

contain 0.22 cubic feet of soil per linear foot, or about 7 cubic feet for a 300-ft long tube extending from a downstream area to the reservoir side of the dam. Larger amounts of soil would be produced by larger diameter tubes.

Since the exact cause of the failure has not been pin-pointed beyond the probability that it was a piping failure of some kind, I feel that a very conservative approach ought to be used with this project. The flow nets suggest that upstream treatments consisting of some combination of cutoff, soil-cement sealing, and blanketing could provide an additional margin of safety over that with the toe drain alone. I therefore recommend that such treatments be given very serious consideration in developing remedial measures to be provided the entire dam before water is put in the reservoir.

Respectfully submitted:

Harry R. Cedergren

Harry R. Cedergren, February-
March, 1980

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Sequim, ...
206-683 ...

# Woodward-Clyde Consultants

April 14, 1980

South Florida Water Management District
P.O. Box V
West Palm Beach, Florida 33402

Attention of Mr. Abe Kreitman

Re: FP&L Reservoir

Gentlemen:

This report briefly describes professional engineering services provided in accordance with the agreement between Woodward-Clyde and the District dated December 3, 1979. Opinions on the cause of the reservoir failure are presented, as are recommendations to the District on remedial measures. Comprehensive accounts of the failure, of investigations carried out and of evaluations of numerous possible causes of failure have been reported on by others and will not be repeated herein.

I visited the project site five times, at which times I made foot inspections of the entire breach area, and foot, auto and helicopter inspections of the approximately 19 miles of embankment. I participated in numerous conferences with your engineers and in workshop sessions with interested parties and their consultants. Documents generated during the design and construction stages by the owner and his consultants, as well as data generated from the extremely comprehensive post-failure investigations, were reviewed.

As the investigations proceeded, it became apparent that a type of analysis known as "flow mate" was important for assessments of cause of failure, and a specialist in this area, Mr. Harry Cedergren, was engaged by you for this purpose. Results of his work, which was excellent, were incorporated in the reasoning which has led to the conclusions presented in this report.

## Cause of Failure

In my opinion the cause of failure was a phenomenon known as "piping". If the velocity of water seeping underground is sufficient to cause movement of the soil particles underground, it is possible for an open "tunnel" to form, leading to the reservoir. This allows unimpeded flow of water, rapid erosion and failure.

In my opinion, the movement of soil particles in this case is most likely to have been downward into an open-structure "shell" layer at about 30-feet below ground. This would cause the embankment foundation to settle and the more rigid embankment to arch, enabling the formation of an opening through the base of the dam. The sudden rupture probably followed 18 months of slow internal soil movements. It is remotely possible that the sudden drawdown in the L65 canal a few days before the failure could have had a "triggering" effect for the failure which, if so, would probably have occurred soon anyway.

Almost all of the factual evidence available fits this mechanism. One of the large "sinks" prevalent in this area of Florida was nearby on the west side, and it is possible that original formation of the sink was related to a similar mechanism accelerated by the 15-foot head of water ponded by the reservoir.

It is also possible that piping occurred through progressive internal erosion of soil beginning at a seepage point in the nearby railroad borrow and progressing to the reservoir. There is no physical or factual evidence to discount either of these two primary mechanisms.

Also on the list of possible specific mechanisms of failure is similar piping beginning at the downstream toe of the embankment. This one, however, should have been observed before failure and apparently was not.

In view of the above it is my opinion that repair of the breached area and preventive measures for the remainder of the embankment should include reconstruction which will substantially reduce risks associated with any of the mechanisms.

Repair of Breach Area

For the breach area the following remedial work has been done:

(1) Cleaning of the foundation area of debris and loose soil.

(2) Placement of a layer of shell-cement and filter cloth over the cemented shell layer exposed by the failure.

(3) Refilling the foundation area to be occupied by the new dike with 95 percent compacted sand fill.

(4) Rebuilding the dike foundation to the essentially before-failure configuration.

(5) Refilling the railroad borrow pit located near the downstream side of the embankment on the left side.

Final plans and specifications for this portion of the repair job, prepared by the owner's consultants, were reviewed and a recommendation for their approval was provided you in our letter of January 19, 1980.

Repair of Main Embankment

In my opinion the entire main embankment, including the breach area, should be provided with facilities to:

(a) substantially increase the distance water flows need to travel to escape;

(b) eliminate water pressures of the type which could encourage migration of soil particles downward into any open-work shell layers;

(c) depress the water table in the area of the downstream toe of the embankment well below ground to avoid uncontrolled exit of water from the ground; and

(d) collect the water and dispose of it in such a fashion as to prevent movement of underground soil particles.

Plans and specifications to accomplish the above have been prepared by the owner's consultants. A set dated March 5, 1980 and stamped "for approval" were received by me on March 8th and were reviewed immediately afterward. Subsequently changes were made in some of the details of the plans, with respect to work on the upstream side and these were reviewed during a meeting on April 10, 1980. Included in the plans are specific provisions for:

(1) An underground drain consisting of layers of filter cloth, filter sand and drain gravels on the downstream slope of the embankment leading to a 12-inch perforated pipe for collection of leakage water.

(2) A pumping system for disposal of leakage water, including a system for measuring seepage flows and amounts of sand pumped, if any.

(3) Resloping of the downstream portion of the embankment to add weight in the appropriate location to assist the filter-drain system in its function and to increase the width of the base of the embankment.

(4) Filling of all external sand borrow pits which are near the embankment.

(5) A vertical cutoff on the upstream side which is deep enough to provide a suitable barrier to flow of water underground, so that unfavorable pressure gradients or soil movements should not occur. This depth is dependent on the foundation layering and is to be determined by detailed exploration at every location ahead of construction, but is to be of the order of 15 feet generally.

(6) Piezometer installations for monitoring of groundwater levels at typical locations.

Inspection

Preliminary plans for monitoring and inspection procedures, both during construction and during operation, have been discussed during the various workshop meetings from time to time. Since this stage of work is not so urgent, final consideration of it was postponed until later.

Recommendation

I recommend that the plans and specifications dated March 5, 1980 be approved for construction, with some flexibility allowed for your staff to assist in final determination of upstream cutoff depths.

Very truly yours,

W A Clevenger
William A. Clevenger

WAC:sem