DUKE POWER COMPANY
P. O. BOX 33189
GENERAL OFFICES
422 SOUTH CHURCH STREET
CHARLOTTE, N. C. 28242
TELEPHONE: AREA 704
373-4011

July 7, 1986

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N.C. UTILITIES COMMISSION

North Carolina Utilities Commission
P.O. Box 29510
Raliegn, NC 27626-0510

Attn: Ms. Sandra J. Webster
Chief Clerk

Subject: Safety Inspection of Dams in North Carolina
Per NCUC Order, Docket No.: E-100, Sub. 23
Dan River Steam Station Ash Dikes
File Nos.: SSS-0503, GAH-0202, SSS-0502

Dear Ms. Webster:

Enclosed are three copies of the Law Engineering Testing Company Safety Inspection Report for the Ash Basin Dikes at Dan River Steam Station located in Rockingham County. One report includes original photographs and the other two contain copies of the original photgraphs.

This inspection was performed in accordance with the subject NCUC Order dated October 11, 1976 with the scope being consistent with the "Recommended Guidelines for Safety Inspections of Dams" released by the Department of the Army, Office of the Chief of Engineering in May, 1976 and as supplemented by "Supplemental Hydraulic and Hydrologic Guidelines for Phase I Inspection of Non-Federal Dams" dated June 5, 1978.

The recommendation to perform a soil test boring, at the slight bulge on the raised portion of the outer slope of the primary basin dike along the access road, to explore the subsurface conditions at the bulge will be performed during the fall of 1986 (Reference Recommendation #7, pg. 7-9). The recommended piezometer and observation well will then be installed in the bulge for monitoring purposes. A re-analysis of slope stability of the dikes's section at the bulge will be performed by the end of the first quarter of 1987 if soft soils are encountered and high piezometric and phreatic levels are indicated. The other recommendation will be addressed and handled as appropriate.

Yours very truly,

S.B. Hager, Chief Engineer
Civil/Environmental Division

S G Crews

By: S.G. Crews, Supervising Design Engineer

3 filed
1 clerk
1 Creasy
1 electric

TRM/ww

Enclosures

cc: C.L. Ray, Jr.
A.R. Hollins, Jr.
R.L. Dick
J.R. Hendricks
C.D. Hatley
F.C. Hayworth
Central Records (w/atta.)

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LAW ENGINEERING
TESTING COMPANY

DUKE POWER COMPANY

DAN RIVER STEAM STATION
ASH BASIN DIKES
ROCKINGHAM COUNTY, NORTH CAROLINA
LETCo. Job No. CHW 5475

SECOND FIVE-YEAR INDEPENDENT CONSULTANT INSPECTION
AS REQUIRED BY
NORTH CAROLINA UTILITIES COMMISSION

JUNE, 1986

**LAW ENGINEERING TESTING COMPANY**
geotechnical, environmental & construction materials consultants
501 MINUET LANE
P.O. BOX 11297 • CHARLOTTE, NORTH CAROLINA 28220
(704) 523-2022

June 20, 1986

Mr. S. B. Hager, Chief Engineer
Duke Power Company
Civil/Environmental Division
P. O. Box 33189
Charlotte, North Carolina 28242

Attention: Mr. R. S. Bhatnager, Senior Engineer

Subject: Five-Year Independent Consultant Inspection
Dan River Steam Station
Ash Basin Dikes
Rockingham County, North Carolina
Per North Carolina Utilities Commission
LETCo. Job No. CHW 5475

Gentlemen:

Law Engineering Testing Company is pleased to submit the following report of our independent inspection of the ash basin dikes at the Dan River Steam Station. The inspection was performed in accordance with Duke Power Company's Specification No. SSS-0502-02 "Specifications for Inspection of Facilities as Required by the North Carolina Utilities Commission" dated February 14, 1986 and as authorized by Duke's letter dated March 20, 1986. Our inspection reported herein is the second five-year independent consultant inspection of the Dan River Ash Basin Dikes.

In general, our inspection noted no external, presently visible signs of serious conditions requiring emergency repairs for public safety. Other than routine maintenance, no major repairs appear warranted at this time.

We appreciate the opportunity to provide our professional services to you on this project. Please let us know if you have any questions.

Very truly yours,

LAW ENGINEERING TESTING COMPANY

Fred C. Tucker

Fred C. Tucker, P. E.
Senior Geotechnical Engineer

Clay E. Sams/ce

Clay E. Sams, P. E.
Geotechnical Consultant

FCT/CES:tmc

Attachments

DUKE POWER COMPANY

DAN RIVER STEAM STATION
ASH BASIN DIKES
ROCKINGHAM COUNTY, NORTH CAROLINA
LETCo. Job No. CHW 5475

# SECOND FIVE-YEAR INDEPENDENT CONSULTANT INSPECTION AS REQUIRED BY NORTH CAROLINA UTILITIES COMMISSION

JUNE, 1986

BY


LAW ENGINEERING TESTING COMPANY
CHARLOTTE, NORTH CAROLINA

REPORT PREPARED BY

Fred C. Tucker

Fred C. Tucker, P. E.
Senior Geotechnical Engineer
Registered, N. C. 8160

NORTH CAROLINA PROFESSIONAL SEAL 8160 ENGINEER FREDERIC C. TUCKER

Clay E. Sams

Clay E. Sams, P. E.
Geotechnical Consultant
Registered, N. C. 4459

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# 1.0 INTRODUCTION

## 1.1 General

This report presents the results of the second independent consultant inspection of the ash basin dikes at the Dan River Steam Station. The independent inspection is performed at five-year intervals as required by the North Carolina Utilities Commission (NCUC) for facilities operated by Duke Power Company in North Carolina and not licensed by the Federal Energy Regulatory Commission (FERC) and not covered by the North Carolina Dam Safety Law of 1967.

The previous independent inspection was performed in 1981 also by Law Engineering Testing Company. The results of that inspection were presented in a report dated September 8, 1981 (LETCo. Job No. CH 4581A).

In this current report, emphasis is placed on noting the development of any new conditions or changes in old, previously reported conditions. The previously reported conditions are recounted only where there is a change or where it is of particular interest or of use in describing the overall condition of a specific project structure. Liberal use is made of photographs to minimize descriptions. The photographs are used to illustrate general conditions of project structures in overall views and specific conditions in close-up views.

## 1.2 Purpose and Scope

The purpose of this dike safety inspection and report is to identify, within the limitations of surficial field inspection and office review of available data, records and operating history, any actual or potential deficiencies, whether in the condition of the project works or in the quality or adequacy of project maintenance, surveillance, or methods of operation, that might endanger public safety. The objective is to recommend immediate action for public protection where necessary, further studies and analyses where required, and acceptance of the present condition of the dikes if the engineering data and inspections so justify.

A review was made of all available relevant reports on the safety of the development. These include reports by or for Federal or State agencies, submitted under NCUC regulations, and reports of inspections performed by Duke engineers. A detailed systematic visual inspection of the project works was performed. A relatively detailed photographic record was made of the visible conditions of the principal project works. Review was made of all available relevant data concerning the stability and operational adequacy of the project works. Based upon results of the above work, an engineering opinion is given of the general condition and adequacy of the dikes, as well as an assessment of the quality and adequacy of maintenance, surveillance, and methods of project operation for the protection of public safety.

The purpose and scope of this inspection and report are consistent with that outlined in Duke Power Company's Specification No. SSS-0502-02, "Specifications for Inspection of Facilities as Required by the North Carolina Utilities Commission" dated February 14, 1986.

## 1.3 Authorization

This NCUC Five-Year Independent Consultant Inspection was authorized by Messrs. S. B. Hager, Chief Engineer, and R. S. Bhatnagar, Senior Engineer, of Duke's Civil/Environmental Division, in their letter dated March 20, 1986.

# 2.0 PROJECT INFORMATION

## 2.1 Location, General Description and Relevant Historical Information

The Dan River Steam Station is located on the Dan River in Rockingham County in north central North Carolina. The power plant is situated on the north side of the Dan River, approximately one mile northeast (downstream) of the State Highway 14 bridge over the river, and on the southeast side of the city of Eden. The ash storage basins and dikes are located immediately east and northeast of the power plant. The project location is shown on Figures 1 and 2 of our 1981 report; these figures are included for reference in Appendix A.

The facilities of concern in this inspection are the earthfill dikes which impound the ash basins, and the outlets for the basins. The older ash storage facilities are located next to the river and consist of an upper, primary basin (west end) and a lower, secondary basin (east end) which are formed and enclosed by approximately one mile of earthfill dikes. These basins have evolved to their present configurations as a result of past expansion and raising of the dikes.

The approximately 550-ft long intermediate (divider) dike which separates the primary basin from the secondary basin was constructed over ash. A re-analysis of this dike using updated soil shear strength parameters was made by Duke Power engineers in 1984; the re-analysis yielded an unacceptably low safety factor against circular arc shear failure of the "downstream" slope. Therefore in 1985 a berm was constructed on the lower (secondary basin) side of the intermediate dike to increase the factor of safety of the downstream slope.

There also is a newer ash storage area formed by construction of approximately 2100 ft of earthfill dike across several natural drainage features north of the older basins. This newer area is used primarily for dry storage of the ash. However, in 1982 a 625-ft long dike was constructed perpendicular to the main dike to enclose the eastern portion of the dry storage basin; the enclosed area was used to hold ash dredged from the primary basin. The dredge pond area has been filled to capacity with ash.

All the older dikes (primary, secondary and divider dikes) were designed to have 2H:1V side slopes and 15-ft crest widths. Design crest elevations are 540 ft and 530 ft for the primary and secondary basin dikes, respectively. Maximum dike height is approximately 60 ft above the outside (downstream) toe of the primary basin dike next to the river. These older dikes have no internal drainage. The dikes next to the river have riprap slope protection designed to extend upslope from the toe to elevation 512 ft on the outside slope. There also is a wide rockfill berm designed to have a top elevation of 503 ft at the downstream toe along the western portion of the primary basin dike next to the river.

The new berm on the secondary basin side of the divider dike was designed to have two levels: one 15-ft wide level at nominal elevation 534.5 ft and a 10-ft wide level at nominal elevation 530 feet. The slopes between the two levels and below the lower level were designed to be 3H:1V. (Only one berm level is evident as noted in the Field Inspection Observations.) The soil berm was designed to overlie a 2-ft thick drainage blanket of No. 67 washed stone wrapped with filter fabric; the drainage blanket was designed to be placed directly on the ash foundation. The outlet (toe) end of the blanket drain was designed to have a protective layer of N.C. D.O.T. Class C riprap.

There are several pipe culverts which pass beneath the primary ash storage basin and dike. These culverts provide drainage directly to the river of surface runoff from adjacent areas on the north side of the basin.

The original dike for the dry ash storage basin dike was designed to have 2.5H:1V side slopes and 15-ft wide crest at elevation 560 feet. The major height portions of this embankment were designed to have toe drains, and the maximum height section, which is approximately 40 ft above the toe, also was designed to have an internal blanket drain.

The dredge pond dike constructed in 1982 was designed to have 2.5H:1V side slopes and 12-ft wide crest at elevation 560 feet. The maximum height of the dike is approximately 25 feet. No toe drain or other internal drainage measures were called for in the design of this dike. A riprap-lined ditch extends along the entire downstream toe of the dike.

Drainage from the primary (upper) basin to the secondary (lower) basin is through a drainage tower and a 36-inch diameter discharge pipe of reinforced concrete located at the bottom of the tower. The "primary" drainage tower is approximately 8.7 ft square of reinforced concrete; it has two 4-ft wide open sides which are fitted with removable precast concrete stop logs. The maximum stop-log elevation is 535 ft-MSL. The discharge pipe is approximately 100-ft long and passes through the bottom of the north end of the divider dike.

Drainage from the secondary basin to the river is through a similar reinforced concrete drainage tower which has four 4-ft wide open sides fitted with removable precast stop logs; the maximum stop-log height is elevation 525 ft-MSL. Discharge from this "secondary" drainage tower is through a 175-ft long, 36-inch diameter (RCP) discharge pipe that passes north-south through the bottom of the dike near the southeast corner of the secondary basin.

Drainage of surface runoff from the filled dredge pond is into the dry ash storage basin through two 24-inch diameter reinforced concrete pipes at the right (north) end of the dredge pond dike. The outlet pipes were designed to have an inlet invert elevation of 557 ft-MSL and slope of one percent. The pipes discharge into a relatively short length concrete-paved ditch which in turn discharges into a riprap-lined ditch that leads to the drainage outlet for the dry storage basin.

The drainage outlet for the dry storage basin is a reinforced concrete drainage tower (box) that has a 36-inch diameter (RCP) discharge pipe which empties into the secondary basin. The pipe is approximately 600 ft long and passes north-south through the bottom of the dry storage basin dike. The drainage tower is 8.33 ft by 9 ft in plan and has two 5-ft wide open sides and an open top. These openings are covered with bar screens. The stop-log height in the open sides currently is about the same as noted in the last 5-year independent inspection; the elevation was reported as 533 ft-MSL in the 1981 report, however, field observation (see Photo 4-29) suggests that it may be 1 to 1.5 ft lower than this. (Elevation of open sides is below top of discharge pipe which is near 533 ft-MSL according to design drawings.) The inlet invert of the tower can be raised by inserting stop logs in the open sides.

Additional descriptions of the physical characteristics of the above structures are presented on pp. 2-4 of the 1981 report. Plan and section views and selected details of the primary and secondary basin dikes, the divider dike and the dry storage basin dike are shown on Figures 4 through 6 of the 1981 report; these figures are included for reference in Appendix A of this current report. (One plan view, Figure 4, has been updated to show the dredge pond dike, the new berm on the divider dike and the location of instrumentation installed since the last independent inspection.) Figure 7 in Appendix A is a new figure showing a section of the divider dike with new berm at the location of the primary basin outlet.

A relatively detailed account of historical information on the design, construction, operation, instrumentation monitoring and previous inspections of the ash storage facilities up to the time of the first independent consultant inspection is presented on pp. 4-8 of the 1981 report. Since that time the most significant changes or additions at the ash storage basins have included:

- the construction and filling of the dredge pond at the east end of the dry ash storage basin;
- the construction of the berm on the secondary basin side of the divider dike between the primary and secondary basins; and
- installation and monitoring of 10 piezometers in the primary basin dike, secondary basin dike and divider dike (see Figure 4 for locations).

## 2.2 Size Classification

The "wet" ash storage basin dikes at the Dan River Steam Station are classified as "intermediate" size dams under the U. S. Army Corps of Engineers guidelines and "large" by the criteria in the North Carolina Dam Safety Regulations. The maximum height, 60 ft, dictates the size classification. The dry ash storage basin dike is also classified "intermediate" size by the Corps' guidelines, but "medium" size by the State's criteria. The dredge pond dike is classified "small" size by both the Corps' guidelines and the State's criteria.

## 2.3 Hazard Classification

All the Dan River Ash Basin dikes are classified "low" hazard (Class 3) under the Corps' guidelines and "low" hazard (Class A) by the North Carolina criteria, due to the lack of downstream development.

## 2.4 Geology and Seismicity

The ash basins are located within the Dan River Basin which is an elongate, asymmetrical fault trough that trends northeastward through Stokes and Rockingham Counties, North Carolina and into southern Virginia, where it is known as the Danville Basin (Thayer, 1970). The Dan River Basin contains sedimentary rocks of Triassic/Jurassic age (180 to 245 m.y.). This "Triassic" basin, one of several within the Piedmont Physiographic Province, is about 3 miles in width at the latitude of the ash basins, but is about 6 miles wide in central Rockingham County to the south. The northwest side of the Dan River Basin is bounded by a southeast-dipping normal fault (about 3 miles from the site) which was active at the time that sediments were being deposited. Sedimentary rocks at the southeastern margin of the Basin, where the ash basins are located (Figure 3), lie unconformably on a complex of older, metamorphic rocks. Parts of the southeastern margin of the Dan River Basin are fault bounded (northwest-dipping normal faults of Triassic/Jurassic age).

Sedimentary rocks within the Basin consist of clastics from conglomerates to shale and mudstone with a few coal-bearing units in areas to the south of the ash basins. Rocks at the ash basins consist principally of mudstone and siltstone in various shades of orange and brown interfingered with gray siltstone, claystone, shale and sandstone (Carpenter, 1982).

As discussed above, the Dan River fault zone is the northwestern boundary of the Dan River Basin and faults locally form the southeastern margin. Diabase dikes (tabular, intrusive bodies of rock) that cut across those and similar faults have been determined by radiometric dating methods to be at least 170 million years in age (Law Engineering Testing Company, 1974).

Because earthquake epicenters cannot be correlated with tectonic structures, the present practice is that earthquakes in this part of the United States are identified with the tectonic province in which they are located. The Dan River ash retention dikes are located in the southern Piedmont province (or seismotectonic region) in which the highest historical seismicity is Intensity VII MM. The dikes are also located in Seismic Zone 2; the Corps of Engineers' guidelines indicate that, "in general, projects located in Seismic Zones 0, 1 and 2 may be assumed to present no hazard from earthquake provided static stability conditions are satisfactory and conventional safety margins exist".

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Carpenter, P.A., III — Geologic Map of Region G, North Carolina: North Carolina Department of Natural Resources and Community Development.

Law Engineering Testing Company 1974 — Preliminary Safety Analysis Report for Cherokee Nuclear Station, Appendix 2C, Geology: Duke Power Company.

Thayer, P.A. 1970 — Stratigraphy and Geology of Dan River Triassic Basin, North Carolina: Southeastern Geology, Vol. 12, No. 1, pp. 1-31.

## 3.0 ENGINEERING AND OPERATIONAL INFORMATION

### 3.1 Engineering Information

A description of the available information on design of the Dan River ash retention dikes up to the time of the last independent inspection is contained on pp. 5-6 of the 1981 inspection report. Design studies, drawings and specifications were made by Duke Power engineers in 1982 for the dredge pond dike. Subsurface exploration of the foundation and borrow areas, laboratory testing of the borrow soils and foundation soils, stability analyses and hydrologic/hydraulic analyses were all done by Duke Power for the dredge pond project.

In 1984 Duke Power engineers re-analyzed stability of the slopes of the primary, secondary and intermediate (divider) dikes based on results of updated shear strength testing of the in-place embankment soils and ash. The data for the re-analyses were obtained by Duke by drilling nine soil test borings, obtaining relatively undisturbed (Shelby) tube samples in the borings, performing laboratory triaxial shear tests, as well as classification tests on the samples, and performing field vane shear tests in the ash. Piezometers were installed in all the borings. As a result of these re-analyses, design drawings and specifications were developed for construction of the berm at the divider dike.

#### 3.1.1 Slope Stability:

The latest slope stability analyses (1984) of the primary, secondary and intermediate (divider) dikes used design parameters for soil and ash as shown in the following table.

| Material | Unit Wt. | SCU(1) Parameters | | SCUe(2) Parameters | |
|---|---|---|---|---|---|
| Foundation Soil | 124.6 pcf | -- | -- | $\phi'=28^{\circ}$, | $c'=800$ psf |
| Original Fill | 121.7 pcf | $\phi=13^{\circ}$, | c=800 psf | $\phi'=27^{\circ}$, | $c'=700$ psf |
| 1968 Fill | 121.5 pcf | $\phi=18^{\circ}$, | c=500 psf | $\phi'=32^{\circ}$, | $c'=300$ psf |
| 1976 Fill | 122.8 pcf | $\phi=23^{\circ}$, | c=750 psf | $\phi'=33^{\circ}$, | $c'=0$ |
| Consolidated Ash | 91.0 pcf | -- | -- | $\phi'=30^{\circ}$, | $c'=0$ |
| Unconsolidated Ash | 91.0 pcf | -- | -- | $\phi'=16^{\circ}$, | $c'=0$ |

(1) SCU = Saturated Consolidated Undrained Triaxial Test (R)
(2) SCUe = Saturated Consolidated Undrained Triaxial Test Corrected for Pore Pressure ($\overline{R}$)

A computer program (LANSLI) which uses a method of analysis similar to the Ordinary Method of Slices was used in the analyses of static slope stability. The results of the 1984 analyses were as follows:

| Structure | Condition | Slope | Calculated Factor of Safety (FS) |
|---|---|---|---|
| Primary Basin Dike | Steady State Seepage | Upstream | 1.36* |
| | | Downstream | 1.40** |
| | Rapid Drawdown (El. 535 to 530) | Upstream | 1.27 |
| Secondary Basin Dike | Steady State Seepage | Upstream | 3.43 |
| | | Downstream | 1.45*** |
| | Rapid Drawdown (El. 527 to 522) | Upstream | 2.58 |
| Intermediate Dike (Without Berm) | Steady State Seepage | Upstream | 1.42** |
| | | Downstream | 1.01 |
| | Rapid Drawdown (El. 535 to 530) | Upstream | 1.27 |
| | | Downstream | 1.01 |
| Intermediate Dike (With New Berm) | Steady State Seepage | Downstream | >1.50 |
| | Rapid Drawdown (El. 535 to 530) | Downstream | 1.27 |

* Calc. FS is for approx. 9-ft deep potential failure arc. Deeper potential failure arcs have FS > 1.5

** Calc. FS is for a potential failure arc of about 5 ft deep. Factors of safety in range of 1.30 to 1.40 were calculated for potential failure arcs less than 5 ft deep.

*** Calc. FS is for shallow potential failure arc. Deeper potential failure arcs have FS > 1.5.

In the 1982 analyses of the dredge pond dike the following design soil parameters were used:

| Material | Unit Wt | UU(1) Parameters | | SCU Parameters | | SCUe Parameters | |
|---|---|---|---|---|---|---|---|
| Foundation Soil | 130.1 pcf | -- | -- | $\phi$=31.5°, | c=0 | $\phi'$=30°, | c'=500 psf |
| Borrow Soil | 118.9 pcf | $\phi$=13°, | c=500 psf | -- | -- | -- | -- |
| Borrow Soil | 128.8 pcf | -- | -- | -- | -- | $\phi'$=32°, | c'=50 psf |

(1) Unconsolidated Undrained Triaxial Test (Quick Test)

The results of these analyses of the dredge pond dike downstream slope indicated minimum calculated factors of safety of 2.63 for the end of construction condition and 1.77 for steady state operating conditions.

As noted in the previous independent inspection report, the calculated factors of safety of the downstream slope of the original main dike of the dry storage basin were 1.42 for end of construction and 1.57 for steady state conditions. The design parameters for the borrow soils were as follows:

| Material | Moist Unit Wt. | Saturated Unit Wt. | UU Parameters | SCUe Parameters |
|---|---|---|---|---|
| Borrow Soil | 120 pcf | 124 pcf | $\phi$=12°, c=500 psf | $\phi'$=28°, c'=150 psf |

As noted in the 1981 independent inspection report, the slope safety factor criteria recommended by Law Engineering and adopted by Duke Power (during the 1976 modifications) were 1.25 for end of construction and 1.40 for steady state seepage condition. For rapid drawdown conditions Duke used a minimum safety factor criterion of 1.2 which is that recommended by the Corp of Engineers.

As shown by the results given previously, the calculated factors of safety from Duke's most current analyses of the various dikes generally meet or exceed the above minimum safety factor criteria. The factor of safety for the upstream slope at one section of the primary basin dike was computed to be slightly less than 1.4 for an approximately 9-ft deep potential failure arc under steady seepage conditions, and some very shallow potential failures were computed to have factors of safety in the range of 1.30 to 1.40 for other dike slopes under steady seepage conditions; however, deep seated potential failure arcs were computed to have factors of safety greater than 1.5 under steady seepage conditions for all the dikes, including the intermediate dike with the new berm. From field observation, the as-built configuration of the berm on the downstream side of the intermediate dike appeared to be more substantial and thus more "stabilizing" than the one used in Duke's stability analyses. The downstream side of the intermediate dike was not analyzed by Duke for rapid drawdown of the secondary basin level, presumably because the ash level is at approximately the full pond elevation (525 ft-MSL) next to the dike on the secondary basin side.

3.1.2 Hydrology and Hydraulics:

Approximate analyses of hydrology and hydraulics of the ash storage basins at the Dan River Steam Station are presented on pp. 9-10 of the 1981 independent inspection report. In those analyses a design storm with a 100-year recurrence interval was checked and all the basins were found to be hydrologically safe against this storm (7.4 inches in 24 hours).

The indicated spillway design flood (SDF) for the "wet" primary and secondary basins is the 100-year flood to 1/2 PMF (Probable Maximum Flood) by the Corps of Engineers criteria; according to the criteria in the North Carolina Dam Safety regulations, the SDF is that resulting from 1/3 PMP (Probable Maximum Precipitation). Thus, the primary and secondary basins were rechecked for 1/3 PMP using the approximate, conservative procedures outlined in the 1981 report. The 6-hour, PMP rainfall depth is 29.3 inches (Fig. 2-4, SCS TR-60); adjusting for 24-hour duration the PMP rainfall depth is 36.8 inches (Fig. 2-6B, SCS TR-60) and thus the 24-hour, 1/3 PMP rainfall depth is 12.3 inches. Pertinent hydrologic data and results of the re-analyses are summarized below:

West (Primary) Basin

| | |
|---|---|
| Drainage Area (Including 11 acres from yard drainage): | 41± acres |
| Top of Spillway Elevation (Max. Stop-log Height): | 535 ft-MSL |
| Top of Dike Elevation: | 540 ft-MSL |
| Time of Concentration: | 0 (instantaneous) |
| Curve Number (CN): | 100 (100% Runoff) |
| Base Flow (From Sump and Ash Sluice Lines): | 5 cfs |
| Pond Elevation at Base Flow: | 535.35 ft-MSL |
| Peak Inflow (1/3 PMP=12.3 inches): | 196 cfs |
| Peak Outflow: | 34 cfs |
| Peak Pond Elevation: | 536.3 ft-MSL |
| Freeboard: | 3.7 ft |

East (Secondary) Basin

| | |
|---|---|
| Drainage Area: | 24± acres |
| Top of Spillway Elevation (Max. Stop-log Height): | 525 ft-MSL |
| Top of Dike Elevation: | 530 ft-MSL |
| Time of Concentration: | 0 (instantaneous) |
| Curve Number (CN): | 100 (100% Runoff) |
| Peak Inflow (1/3 PMP=12.3 Inches): | 205 cfs |
| Peak Outflow: | 118 cfs |
| Peak Pond Elevation: | 527.4 ft-MSL |
| Freeboard: | 2.6 ft |

Our approximate analyses conservatively ignored the flood attenuating capacity of the dry storage basin and the filled-in dredge pond within the dry storage basin. It is concluded that the primary and secondary basins should be safe also for a SDF produced by 1/3 PMP. It is further concluded, on the basis of the 1981 analyses, that the dry storage basin (which is not an impoundment) should still be safe for the 100-year storm. These degrees of hydrologic safety are contingent upon keeping the outlet structures maintained in good working order.

The presence of the dredge pond serves to attenuate flood flow through the dry basin. Review of Duke's calculations indicates that the dredge pond was designed to have about 1 ft of freeboard under a SDF produced by the 100-year, 24-hour duration rainfall. The rainfall amount of 8.5 inches which was used in the analysis apparently was misread, on the conservative side, from the rainfall maps in the Rainfall Frequency Atlas of the United States (TP-40). (Correct amount is 7.4 inches.) On the basis of our review of Duke's calculations, it is concluded that the dredge pond (which also is not an impoundment) should be safe for the 100-year storm.

## 3.2 Operations Related to Project Safety

Operation of the Dan River ash basins is described on p. 7 of the 1981 independent inspection report. No major additions or modifications to the ash storage facilities are anticipated by Duke at this time. Safety related operations are outlined below.

Safety related operations at the subject facilities involve routine inspections and maintenance as required. Inspections are carried out by Duke personnel and by outside consultants.

Plant personnel perform routine inspections of the subject facilities. Duke Power design engineers make annual inspections and prepare written reports documenting their observations. At five-year intervals, independent inspections by outside consultants are performed per NCUC regulations; these inspections are also documented by written reports.

# 4.0 FIELD INSPECTION OBSERVATIONS

The field inspection was done on April 15, 1986 by Mr. Fred C. Tucker, P. E. of Law Engineering in company with Mr. Tony Mathis from Duke's Design Engineering Department, Mr. Larry Harper with Fossil Operations and Mr. K. Chandrasuwan from the plant. Weather conditions during inspection were partly sunny to cloudy and mild with a light shower in early afternoon. Water levels in the ash basins at the time of inspection appeared to be near the latest available recorded levels (March 3, 1986) as follows:

| | |
|---|---|
| West (Primary) Basin | 531.5 ft-MSL |
| East (Secondary) Basin | 522.9 ft-MSL |

The water levels had been lowered to facilitate the construction of the berm at the intermediate (divider) dike and were still well below normal operating levels at the time of inspection though it was understood that they were being slowly built back up. Conditions observed are presented below. Photographs referenced below are contained in Appendix B.

## 4.1 Primary and Secondary Basin Dikes and Outlet Works

### 4.1.1 Crest and Inside Slope:

Typical views of the crests of the primary basin dike, secondary basin dike and intermediate dike are shown in Photos 4-1, 4-2 and 4-3, respectively. No obvious signs of settlement or displacement (tension cracking) were observed on the crests. The crest of the secondary basin dike had been resurfaced with crusher run stone along most of its length; this resurfacing filled-in ruts that were noted in the last independent inspection. The intermediate dike also appeared to have new crusher run surfacing. Grass was observed to be growing through the crusher run surfacing along the centerline of the primary basin dike. Overall, the dike crests were observed to be in good condition.

The inside (upstream) slopes of the dikes were observed to have a good grass growth that had been recently mowed down to the normal water levels in the basins. Typical views of the inside slopes of the primary basin dike and the secondary basin dike are shown in Photos 4-4 and 4-5, respectively; a typical view of the upstream (primary basin) side of the intermediate dike is shown in Photo 4-6. No slumps, slides or significant surface erosion were observed on the inside slopes of the dikes. There still is some minor wave erosion on the inside slope of the secondary basin dike. Wave erosion that was noted on the upstream side of the intermediate dike in the 1981 inspection was not noticeable in this current inspection due to ash build-up on the slope and growth of vegetation. In the primary basin the ash surface was observed to be above water level in much of the basin, and it generally was overgrown with vegetation.

4.1.2 Outside Slope and Toe:

The lower portions of the outside slopes of the primary and secondary basin dikes next to the river were observed to still be heavily overgrown with trees, bushes, briars and other vegetation as was observed in the 1981 inspection. The outside slope of the raised portion of the primary basin dike and the upper part of the outside slope of the secondary basin dike were observed to have a grass cover as shown in Photos 4-7 and 4-8, respectively. No slumps, slides or major erosion were observed on the more visible, grassed portions of the slopes. Close inspection for slumps, slides or other evidence of shear failure on the lower portions of the slopes next to the river was not possible due to the dense vegetation, but none of these conditions was obvious except for a very old slump area, shown in Photo 4-9, of limited extent on the outer slope above the rockfill berm at the primary basin dike; the old slump was barely noticeable and showed no signs of recent movement. At many locations on the outside slope of the primary basin dike there were bare soil areas that had recently been tracked with a dozer and hydroseeded; one of these areas is shown in Photo 4-10.

The rockfill berm at the base of the primary basin dike next to the river was observed to be in good condition though vegetation is beginning to overgrow the berm in places as shown in Photo 4-11. A shallow animal hole was noted at one location in the primary basin dike slope just above the rockfill berm.

A light pole located on the outside slope of the raised portion of the primary basin dike next to the plant entrance road, near where the road veers away from the dike and toward the plant, was observed to be leaning out at the top away from the dike as evidenced by the slack guy wire shown in Photo 4-12. Between this pole and the next pole to the north a slight "bulge" or "hump" in the outside slope was noticed as shown in Photo 4-13. The next pole to the north was also observed to be leaning slightly.

The wet area located at the base of the original (1956) dike at the west end of the primary basin, where the ash sluice and waste water sump lines cross over to the basin, still exists in much the same way it appeared in the 1981 inspection; a view of part of this area is shown in Photo 4-14. A hole caused by seepage erosion was observed at the base of the dike next to the ash line shown in Photo 4-14. A view of this hole and orange colored seepage emerging from it is shown in Photo 4-15; the hole was probed and found to extend approximately 18 inches horizontally into the toe of the dike. The erosion (piping) did not appear to be active at the time of inspection, based on the lack of soil fines in the seepage.

The toe ditch, located between the plant access road and the toe of the primary basin dike, that was full of water and wet ground vegetation in the 1981 inspection was observed to be dry in this current inspection, possibly due to the lower basin water level during this inspection.

A view of the newly constructed berm on the secondary basin side of the intermediate dike is shown in Photo 4-16. As shown, only one level of the berm is evident; it appeared to be at or near the upper level of the two-level berm called for by design. Grassing had not yet become well established on the berm surface, except along the drainage swales where Excelsior blanket had been installed. No tension cracks, depressions, slumps or other signs of instability were observed on the berm or along its riprapped toe. Evidence of small boils in the ash at the toe of the berm was noted at one location, as shown in Photo 4-17, near the southeast quarter point of the dike. The boils were not active at the time of inspection. A small flow of clear seepage was observed emerging from the riprapped toe (visible in Photo 4-17).

A wet area, apparently due to poor surface drainage, was noted on the north side of the secondary basin. A drainage ditch had been excavated to drain the area into the basin. In our opinion, this wet area has no implication with respect to safety of the dikes.

The entrance end of the easternmost culvert (52/36-inch diameter lines) that passes beneath the ash basin is shown in Photo 4-18. Seepage from the banks of the drainage swale upgradient of the entrance still flows into the culvert. A build-up of sediment was observed at the entrance as shown in Photo 4-18. The outlet end of this culvert is shown in Photo 4-19. A close-up inspection of the water being discharged revealed no sediment or ash being carried by the water. The concrete apron below the outlet was observed to be cracked and undermined. The outlet end of the other culvert (48-inch diameter line) which passes beneath the middle of the primary basin is shown in Photo 4-20. The water flowing from this culvert was observed to be clear in a close-up inspection.

Trees and dense undergrowth still cover the area between the steep river bank and the toe of the dikes, except where the rockfill berm exists. The alluvial riverbank was inspected from a boat to check for any major seeps, holes or evidence of potential piping. None of these conditions was observed. Some bank undercutting caused by river scour was observed.

### 4.1.3 Outlet Structures:

The visible part of the primary drainage tower is shown in Photo 4-21, and the downstream end of the 36-inch diameter reinforced concrete pipe (RCP) outlet for the tower is shown in Photo 4-22. The drainage tower appeared to be in good condition except for the rusty steel frame on top of the tower. Also, ash was observed to be built-up around much of the tower perimeter, and the skimmer structure was observed to be grounded on the ash. No dropouts were observed in the embankment material over the outlet pipe; no seepage was observed around the outlet end of the pipe.

The visible part of the secondary drainage tower is shown in Photo 4-23; the outlet end of the 36-inch diameter reinforced concrete discharge pipe is shown in Photo 4-24. These structures also appeared to be in good condition though the steel frame on top of the drainage tower is also rusted. No obvious signs of seepage and piping of soils around the outlet pipe were observed. The discharge from the pipe was clear flowing.

## 4.2 Dry Ash Storage Basin Dike and Outlet

The crest, upstream slope and downstream slope of the maximum height section of the dry ash storage basin dike are shown in Photos 4-25, 4-26 and 4-27, respectively. As shown, grass has not been very well established on the slopes of this dike. However, the slopes recently had been tracked with a dozer and hydroseeded. No tension cracks or significant depressions were observed on the crest, and no slumps, slides or other signs of shear failure were observed on either the upstream or downstream slopes. Some seepage or wet areas were observed just below the toe drains at the higher sections of the northeast and southwest portions of the dike. A view of the wet area at the toe of the southwest portion of the dike is shown in Photo 4-28; this is typical of the wet conditions observed. Some of the seepage from this area comes from a spring in natural ground down-gradient of the toe, rather than from the toe drain.

The dry ash basin drainage tower is shown in Photo 4-29, and the outlet end of the 36-inch diameter RC bottom discharge pipe is shown in Photo 4-30. These structures were in good visual condition. Some trash was accumulated on the bar screens of the drainage tower. No dropouts or leakage were observed in the embankment soils where the outlet pipe is buried through the dike and railroad embankment.

### 4.3 Dredge Pond Dike

The crest and downstream slope of the dredge pond dike are shown in Photos 4-31 and 4-32, respectively. No major depression or tension cracks were observed on the crest. The downstream slope was only sparsely grassed; it had recently been tracked with a dozer and hydroseeded. No signs of instability were observed on the downstream slope. The upstream slope is almost completely buried with ash; only the upper several feet are visible. A small, practically imperceptible flow of clear seepage was observed at the downstream toe of the dike. The two 24-inch diameter outlet pipes located through the right end of the dike were observed to be unobstructed and in good condition.

## 5.0 PREVIOUS INSPECTIONS AND PERTINENT REPORTS

As previously mentioned, Duke Power design engineers make annual inspections which are documented. The annual inspection reports for the past 4 years (1982, 1983, 1984 and 1985) were reviewed. None of these reports indicated any serious conditions which would immediately jeopardize the safety of the Dan River ash retention dikes. Some of the same conditions reported in this current inspection were noted. The slight bulge and leaning light poles on the outside slope of the raised portion of the primary basin dike next to the plant entrance road were first noted in Duke's 1984 inspection report.

## 6.0 MONITORING INFORMATION

In 1984 nine piezometers (P1 through P7, P9 and P10) were installed in the dikes which impound the primary and secondary basins: two (P1 and P2) were installed in the secondary basin dike, five (P3 through P7) were installed in the primary basin dike, and two (P9 and P10) were installed in the intermediate (divider) dike. The piezometers were installed to depths ranging from 10 to 20 ft; each was sealed 7 ft above the bottom of the 1/2 inch diameter PVC piezometer tube which was slotted in the bottom 5 feet. Water level readings in the piezometers have been taken on a monthly basis since October, 1984. The primary and secondary basin water levels are also recorded on a monthly basis along with the piezometer readings.

Nine settlement monuments (M1 through M9) were installed on the crest of the dry ash storage basin dike in 1980, but monitoring of elevations on these monuments was not begun till September, 1981. The elevations were surveyed monthly through October, 1982, then yearly beginning in February, 1983.

Approximate locations of the monitoring instruments are shown on Figure 4 in Appendix A. Furnished time versus reading plots of the instrumentation data are included in Appendix C; the monitoring record shown by these plots extends to March, 1986. The individual readings of the piezometers and of the water levels in the basins are also included in Appendix C for reference. Comparisons of the highest recorded piezometer levels with the design phreatic line are shown on four cross sections in Figure 8 in Appendix A.

The monitoring record indicates that the water levels in the primary basin were below the maximum stop-log elevation 535 ft-MSL during the period of available record (October, 1984 through March, 1986) and fluctuated between a low at 528.4 ft-MSL on August 21, 1985 and a high at 533.4 ft-MSL on November 13, 1985. The recorded water levels in the secondary basin generally were below the maximum stop-log elevation 525 ft-MSL, except for a high of 525.4 ft-MSL on February 22, 1985; the recorded low was 519.9 ft-MSL on October 2, 1985. The differential water level between the primary and secondary basins generally was less than 10 ft except in October-December, 1985 when the differential level was greater than 11 feet (11.2 ft maximum in December, 1985).

The two piezometers (P1 and P2) in the secondary basin dike have shown water level fluctuations in the range of 3 to 4 feet with no apparent upward trend in water level. The fluctuations appear generally to have been directly influenced by fluctuations in the secondary basin water level. The recorded water levels in those piezometers were below the elevations of the seals; thus, these piezometers were functioning like observation wells. The highest recorded level in P1 on the crest was just slightly above (by less than 0.5 ft) the phreatic line used in stability analysis; that in P2 on the outside slope was well below the phreatic line used in the analysis.

Three of the piezometers (P3, P4 and P5) are located on a section of the primary basin dike next to the river and two (P6 and P7) are located on a section next to the plant access road. Water levels in the piezometers (P3 and P6) located on the crest have shown very little fluctuation, less that 1 foot. Water levels in the piezometers (P4, P5 and P7) located on the outside slope have shown wide fluctuations, on the order of 6 ft at the section next to the river (P5) and over 9 ft at the section next to the plant access road (P7).

None of the fluctuations in piezometer levels in the primary basin dike have shown any correlation with the fluctuation in level of water in the basin. In fact, the highest levels recorded in P4, P5 and P7 occurred when the primary basin water level was at its lowest; these three piezometers had an unusual, gradual rise in water levels beginning in April, 1985, rising to a peak in August, 1985, then falling back to near previous levels by January, 1986. No permanent increasing trend in the water levels is evident. The maximum recorded water levels at the piezometers in the primary basin dike were below the phreatic line assumed in stability calculations, except that at P7 which was less than 1 ft above the design phreatic line. Typical water levels in all these piezometers were below the design phreatic line. Only P5 functions as a true piezometer (water levels located above seal) all the time. Except for the highest recorded levels in P7, the other piezometers in the primary basin dike function as observation wells.

The two piezometers (P9 and P10) in the intermediate dike also function as observation wells. That (P9) located on the crest has shown a fluctuation of less than 2 ft in water levels, and P10 on the downstream side has shown a maximum water level fluctuation on the order of 4.5 feet. (No readings were made in P10 after September, 1985 due to construction of the berm.) The bottom of piezometer P9 is just below the design phreatic line; thus the highest recorded water level in this piezometer was somewhat above the design phreatic line by approximately 1.5 ft; however, this piezometer has been typically dry during the period of available record. The maximum recorded water level in P10 was below the design phreatic line. No increasing trend in water levels in these piezometers is apparent.

As indicated above, most of the piezometers actually function as observation wells since the recorded water levels have generally been below the elevation of the seals in the piezometers. It is noted that true piezometers do not give a direct measure of the phreatic surface (free surface). In most embankment dams, where there is a downward component of seepage flow, the pressure head or piezometric surface is lower than a hydrostatic distribution below the phreatic surface. This should be kept in mind when comparing recorded piezometer levels with the design phreatic line.

Monitoring of the settlement monuments on the dry ash storage basin dike has shown no significant settlement. In fact, most of the monuments have shown a slight net heave. Two monuments (M2 and M5) have shown a slight net settlement of 0.12 inch. These indicated movements probably fall within the margin of error for the survey.

Monitoring also has been done of the inflow and outflow at the 52/36-inch diameter culvert under the ash basin. According to Duke's yearly inspection reports this monitoring has noted no significant difference between inflow and outflow, thus indicating no significant seepage into the culvert at the joints. This monitoring apparently has been based on visual observation and qualitative assessment of the flow rates.

# 7.0 CONCLUSIONS AND RECOMMENDATIONS

## 7.1 Conclusions

The dikes and outlet structures at the Dan River ash basins are in relatively good visual condition. There are no obvious visual signs of imminent instability or serious inadequacy of any of the project works that would require emergency remedial action. Visual inspection is still hampered by the dense vegetation growing on the lower part of the outside slopes of the primary/secondary basin dikes next to the river. Overall, the conditions of the dikes are not appreciably different than observed in the 1981 inspection.

In our opinion, the engineering analyses, as reviewed and supplemented by this report, give an adequate indication of the hydrologic capabilities of the Dan River ash retention dikes. The project's degree of hydrologic safety, based on the results of the latest hydrologic evaluation, meets the criteria established by the Corps of Engineers and N.C. Dam Safety regulations. Other than the vegetative growth around the inlet of the primary drainage tower and the ash build-up which is causing problems with the skimmer structure at the tower (see Photo 4-21), no conditions were observed that would have a potentially serious impact on the assumptions used in the flood routing analyses. The vegetation should not be allowed to become so prolific as to restrict flows through the tower. No further study of hydrologic safety with respect to downstream flood hazard appears warranted at this time.

Duke Power's latest slope stability analyses of the Dan River dikes indicate computed factors of safety that generally meet or exceed the minimum safety factor criteria recommended by Law Engineering and adopted by Duke. The analyses indicate conventional margins of safety against deep seated shear failure. The design soil parameters used in the latest stability analyses appear generally to be reasonable except that the effective cohesion used for the foundation soil ($c'$=800 psf) and original fill ($c'$=700 psf) in the 1984 re-analyses of the primary and secondary basin dikes and the effective cohesion used for the foundation soil ($c'$=500 psf) in the 1982 analysis of the dredge pond dike are greater than values that would normally be expected for either undisturbed or remolded residual soils. Review of the triaxial shear test data indicates that the design strength parameters for these soils were based on the lower bound and the lower 1/3 to 1/2 bound of the tested strengths. However, the saturated, consolidated, undrained triaxial shear tests, corrected for pore pressure, yielded effective cohesion values which appear inordinately high. It is possible that such erroneously high cohesion values can result, particularly on clayey soil samples, if the rate of shearing during the test is too fast.

The design strength parameters used for the foundation soils in the 1982 analysis of the dredge pond dike are also unusual in that the effective cohesion ($c'$=500 psf) is greater than the total cohesion (c=0) and the effective friction angle ($\phi'=30^o$) is less the total friction angle ($\phi=31.5^o$); this is the reverse of what would normally be expected.

Review of the computer output for the 1984 re-analyses of slope stability of the primary and secondary basin dikes under steady state seepage conditions indicates that the computed factors of safety for potential failure arcs passing through the original fill materials or through the foundation soils are greater than 2.0. Further, it is noted that the LANSLI computer program yields slightly conservative (i.e., lower) factors of safety than other programs which are based on more accurate methods of analysis, such as the Modified Bishop Method. Thus, it is our opinion that the overall conclusions of the stability analyses (i.e., FS>1.5 for deep seated potential failure arcs and FS=1.3 to 1.4 for very shallow potential failure arcs) would not change significantly if the design strength parameters for the original fill and foundation soils were adjusted to have lower effective cohesion values and slightly higher friction angles.

The computer output for the 1982 analysis of slope stability of the dredge pond dike was not available for review to evaluate what effect a lower effective cohesion assumption for the foundation soil might have on the conclusions of the stability analysis under steady state seepage conditions. However, precedent would suggest that this relatively low dike with 2.5H:1V downstream slope should have adequate safety against deep seated shear failure. Further, this dike, situated as it is in the dry ash storage basin, has no implication with respect to public safety.

No further analyses of slope stability appear warranted for public safety at this time, except as may be required in the investigation of the slight bulge subsequently discussed in this report. For record purposes, Duke may wish to re-run some stability analyses under steady state conditions based on re-evaluated effective strength parameters for the soils discussed above.

The very old slump area noted on the outer slope of the primary basin dike and shown in Photo 4-9 appeared to have been shallow seated and of limited areal extent. No evidence of recent movement was noted; the area appeared stabilized by the vegetation.

The shallow animal hole noted in the outer slope of the primary basin dike next to the rockfill berm does not threaten the safety of the dike.

Vegetation should not be allowed to overgrow the rockfill berm. As noted previously, the dense vegetation growing on the lower part of the outer slopes of the dikes next to the river severely hampers visual inspection.

The undercutting noted at some locations along the river bank does not currently threaten to undermine the toe of the dikes next to the river.

The leaning light poles on the outside slope of the primary basin dike next to the plant access road possibly are the result of insufficient lateral support. The poles probably are embedded in the saturated part of the embankment and possibly even extend into the saturated ash which underlies the embankment; the lateral support capacity of the saturated materials probably is not very substantial. The tension in the guy wires may have influenced the direction of leaning of the poles. (The poles were observed to lean in the direction of the guy-wire pull.)

The slight bulge noted on the slope between the two leaning poles possibly is a feature constructed into the embankment and not noticed before. The feature is not prominent and could easily escape attention. No tension cracks, shear displacements or other signs of movement were seen at the bulge or in the slope and crest above the bulge. The occurrence of the bulge in the vicinity of the power poles may only be coincidence.

A conceivable cause for the bulge, however, could be uplift on the embankment created by a build-up of pore water pressures in the foundation ash during the time that there was a gradual large rise in recorded water level in piezometer P7 (and in piezometers P4 and P5). The bulge may be a manifestation of the uplift at a local weak zone or thinner zone of embankment soil over the ash. The cause of the leaning of the nearby light poles may somehow be related to the unusual "surge" of pore water pressures, possibly by reducing the effective weight of the materials in which the poles are embedded and thereby reducing the lateral load resistance of the frictional materials. There appears to be no reasonable explanation for the gradual rise, then fall of the water levels in piezometers P4, P5 and P7. The bulge should be investigated by making a soil test boring and installing a piezometer and observation well as recommended in the following section. The area of the bulge and leaning light poles should be closely monitored, particularly when high water level readings are being recorded in the piezometers; during such times particular attention should be paid to any water bubbling up around the poles or at the bulge.

The gradual seepage erosion that created the hole in the toe of the dike in the wet area at the west end of the primary basin does not currently threaten safety of the dike. (The erosion was perhaps more active during the time of the "surge" in the recorded water levels in the piezometers.) The area should be protected against further erosion by placing an inverted filter over it.

The boils in the foundation ash at the toe of the new berm at the intermediate dike may have occurred during construction of the berm as a result of consolidation of ash, or they may have occurred when the differential water level between the primary and secondary basins was greater than 11 ft in October through December, 1985. The cause of the boils should be investigated to determine if a maximum differential water level should be established to prevent their occurrence.

The wet toe areas at the dry ash storage basin dike do not threaten the stability of the dike. The gradual flows of clear seepage from the toe drains indicate that the drains are functioning. The wet area at the toe of the dredge pond dike also does not threaten the stability of that dike.

Methods of maintenance and surveillance, as they relate to overall project safety, appear generally to be adequate. Maintenance should be provided as necessary to keep a good stand of erosion resistant grass on the slopes of the dikes. Vegetation growth around the primary drainage tower and trash around the dry ash basin drainage tower should not be allowed to restrict flow into these towers. The steel frames on top of the primary and secondary drainage towers are in need of painting. The sediment build-up at the entrance of the 52/36-inch diameter culvert should be removed. The need for establishing a maximum operating differential water level between the primary and secondary basins should be investigated as previously mentioned.

The monitoring program appears adequate, except that it would be desirable to quantitatively (rather than qualitatively) monitor the inflow and outflow at the 52/36-inch diameter culvert, as recommended in the 1981 inspection report, to check for joint leakage. It would also be desirable to do quantitative monitoring of inflow and outflow of the 48-inch diameter culvert that also passes beneath the ash basin; part of this culvert is constructed of corrugated metal pipe which would be expected to have less longevity of satisfactory service than the reinforced concrete pipes.

The settlement monitoring data indicate no large settlements of the dry ash storage basin dike. The piezometer monitoring data indicate typical water levels that were below the design phreatic line and maximum water levels that also generally were below the design phreatic line, except at P1, P7 and P9 where maximum water levels were less than .5 ft, 1 ft and 1.5 ft, respectively, above the design phreatic line. These temporary slightly elevated readings do not warrant a reanalysis of slope stability, in our opinion.

As previously noted, there was an unusually high rise, then fall of the water levels in piezometers P4, P5 and P7. This unusual fluctuation was not influenced by the water levels in the primary basin, and it occurred over a time period of 10 months (April to January with peak in August), thus showing no seasonal pattern and no correlation with rainfall/infiltration. The cause for the unusual fluctuation is not apparent.

## 7.2 Recommendations

1) No further study of hydrologic safety is recommended at this time. An investigation of the slight bulge on the outside slope of the primary basin dike is recommended as outlined in item 5 below.

2) It is recommended that consideration be given to establishing grass in place of the trees and undergrowth on the lower part of the outside slope of the dikes next to the river, to enhance the value of visual inspections. The clearing should be done with care to minimize disturbance of the slope soils and the riprap on the toe of the slope. The riprapped part of the slope of course need not be grassed but should be treated with a suitable herbicide to discourage future growth. The rockfill berm should be similarly treated.

If it is determined that the 2H:1V slopes are too steep to allow practical maintenance of a grass cover, it is recommended that, at a minimum, a cleared path be maintained along the toe of the dikes next to the river and all thick underbrush on the slopes be cut and removed prior to the annual inspections. Trees larger than about 6 inches in diameter should not be allowed to grow on the dike slopes; trees approaching this size should be cut with a chain saw and removed.

3) Quantitative monitoring of the basin water levels and the piezometer water levels should continue on a monthly basis. Yearly monitoring of the settlement monuments on the dry ash storage basin dike should continue more as a means of monitoring the crest for shear displacements from potential slope failures than as a means of monitoring consolidation settlements.

4) It is recommended that quantitative monitoring of inflow and outflow be done at the culverts which pass under the ash basin to check for potential leakage. It is recommended that this monitoring be done at 6-month intervals. If there is a significant difference between inflow and outflow, or whenever there is some cause to suspect leakage, the inside of the culverts should be inspected for leakage.

5) It is recommended that a soil test boring be made at the slight bulge on the raised portion of the outer slope of the primary basin dike next to the plant access road to explore subsurface conditions at the bulge. The boring should extend through the embankment and underlying ash down to the residual soil beneath the ash. Standard penetration testing

should be performed continuously. It is recommended that a piezometer be installed in the boring and sealed in the foundation ash (i.e., the seal should be located at the bottom of the embankment) to allow monitoring of piezometric levels in the ash. It is further recommended that an observation well for monitoring the phreatic level in the embankment be installed in the same hole above the piezometer seal or in a nearby hole that does not extend below the bottom of the embankment. A re-analysis of slope stability of the section at the bulge may be required if soft soils are encountered and high piezometric and phreatic levels are indicated. It is recommended that future inspections closely observe conditions at the leaning light poles and the slight bulge. Plant personnel should make a special effort to view conditions at these locations whenever high piezometer water levels are recorded. Observations of strong seepage flows emerging around the base of the light poles and at the bulge should be immediately reported to Duke Design Engineering for evaluation.

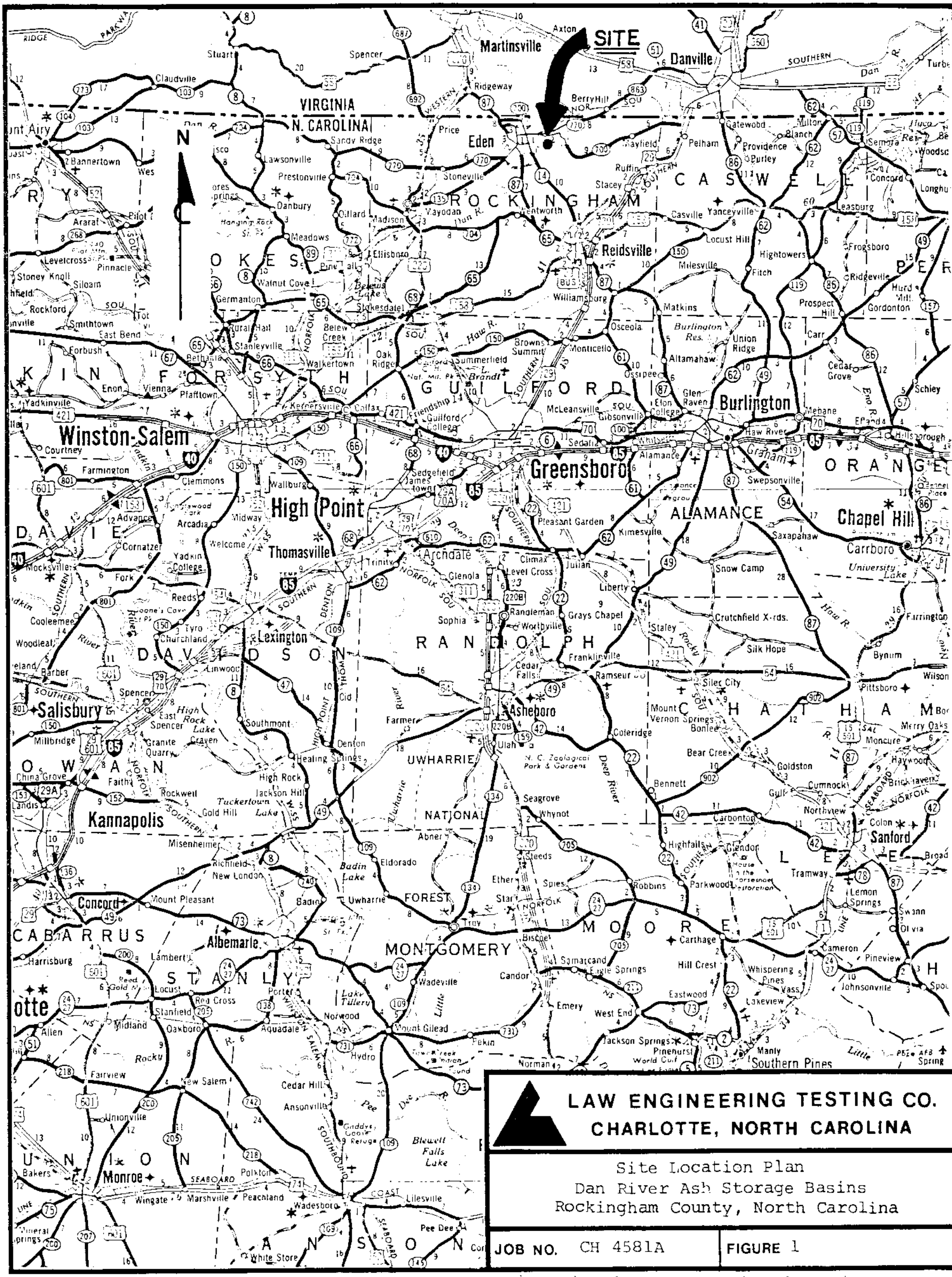
6) It is recommended that an inverted filter be placed over the hole at the toe of the dike at the west end of the primary basin.

7) It is recommended that an investigation be made to determine if a maximum differential water level between the primary and secondary basins should be established to prevent the occurrence of boils and associated piping along the toe of the berm at the intermediate (divider) dike.

8) It is recommended that vegetation be removed from around the weir openings at the primary drainage tower. Other maintenance items are noted in the previous Section 7.1.

APPENDIX A


| | |
|---|---|
| Figure 1 | Site Location Plan (From 1981 Report) |
| Figure 2 | Site Vicinity (From 1981 Report) |
| Figure 3 | Area Geology (From 1981 Report) |
| Figure 4 | Plan of Ash Retention Dikes (From 1981 Report) |
| Figure 5 | Sections Through Primary and Secondary Dikes (From 1981 Report) |
| Figure 6 | Typical Sections and Details (From 1981 Report) |
| Figure 7 | Section at Primary Basin Outlet (New Figure) |
| Figure 8 | Piezometer Readings at Selected Sections (New Figure) |




**LAW ENGINEERING TESTING CO.**
**CHARLOTTE, NORTH CAROLINA**

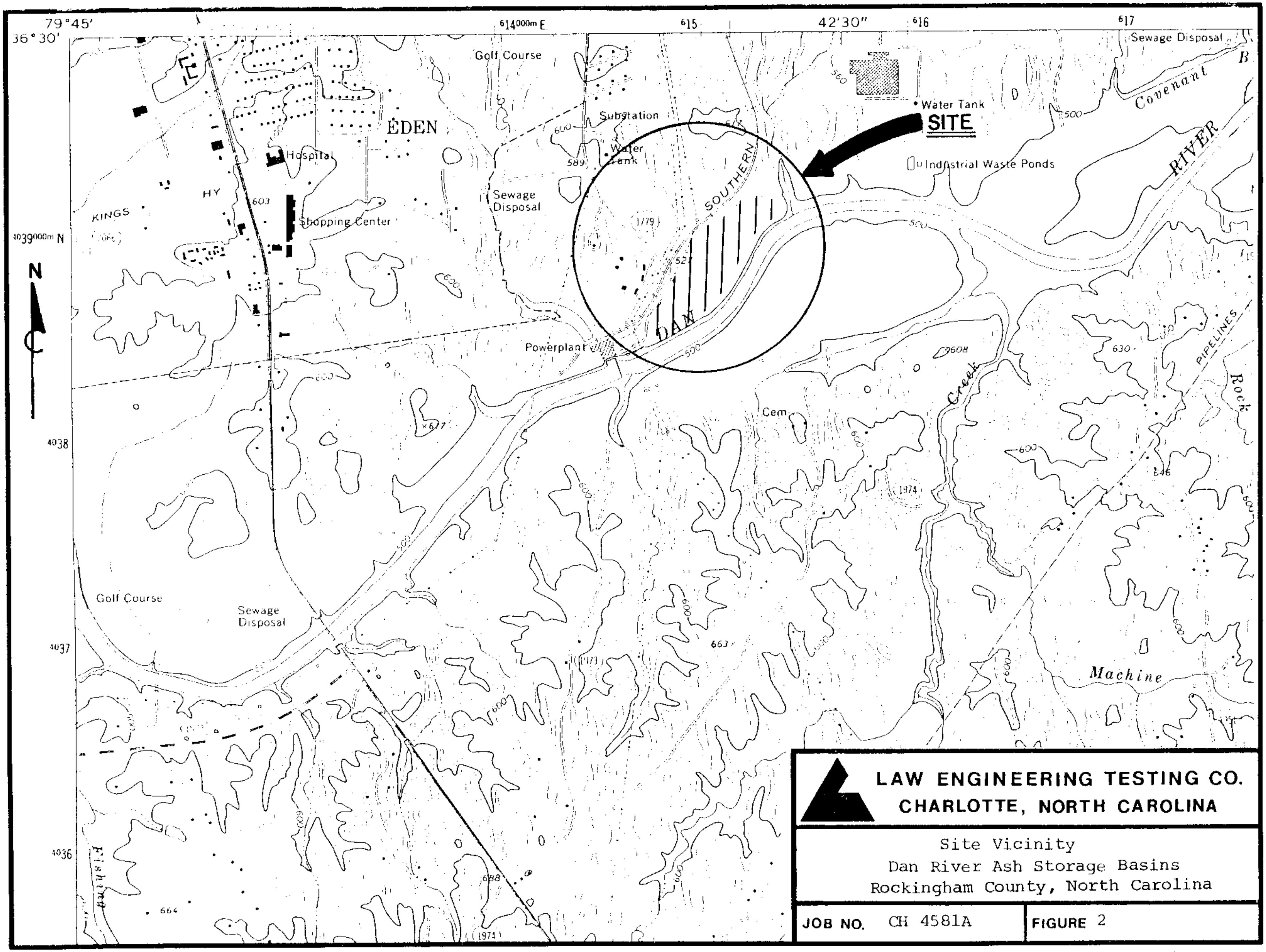
Site Location Plan
Dan River Ash Storage Basins
Rockingham County, North Carolina

| JOB NO. CH 4581A | FIGURE 1 |
|---|---|

Reissued under Job No. CHW 5475 (June 1986)

Ref: Official North Carolina Road Map for 1976-1977

Scale: 1"= 13 Mi. (Approx.)




**LAW ENGINEERING TESTING CO.**
**CHARLOTTE, NORTH CAROLINA**

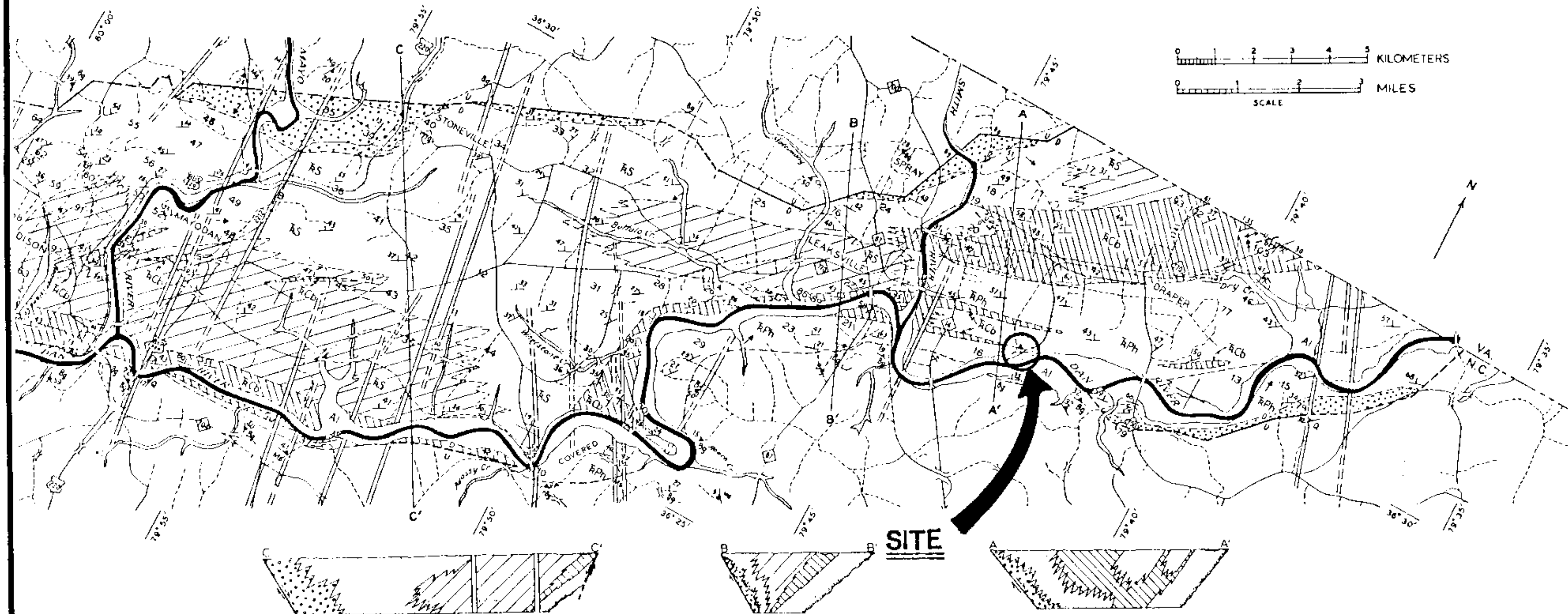
Site Vicinity
Dan River Ash Storage Basins
Rockingham County, North Carolina

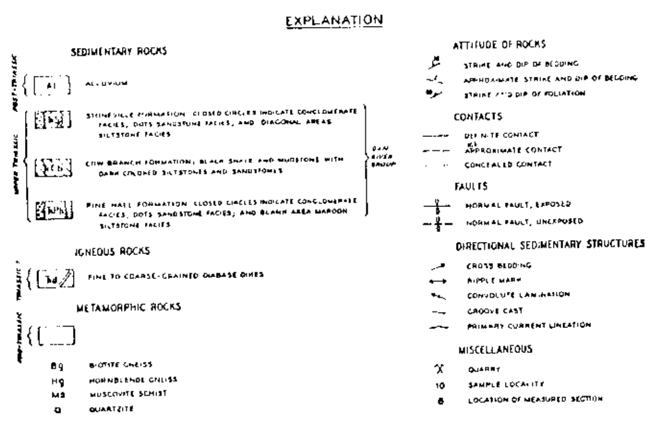
| JOB NO. CH 4581A | FIGURE 2 |
|---|---|

Reissued under Job No. CHW 5475 (June 1986)

Ref: USGS Southeast Eden, N. C. Quadrangle (1971)

Scale: 1" = 2000' (Approx.)



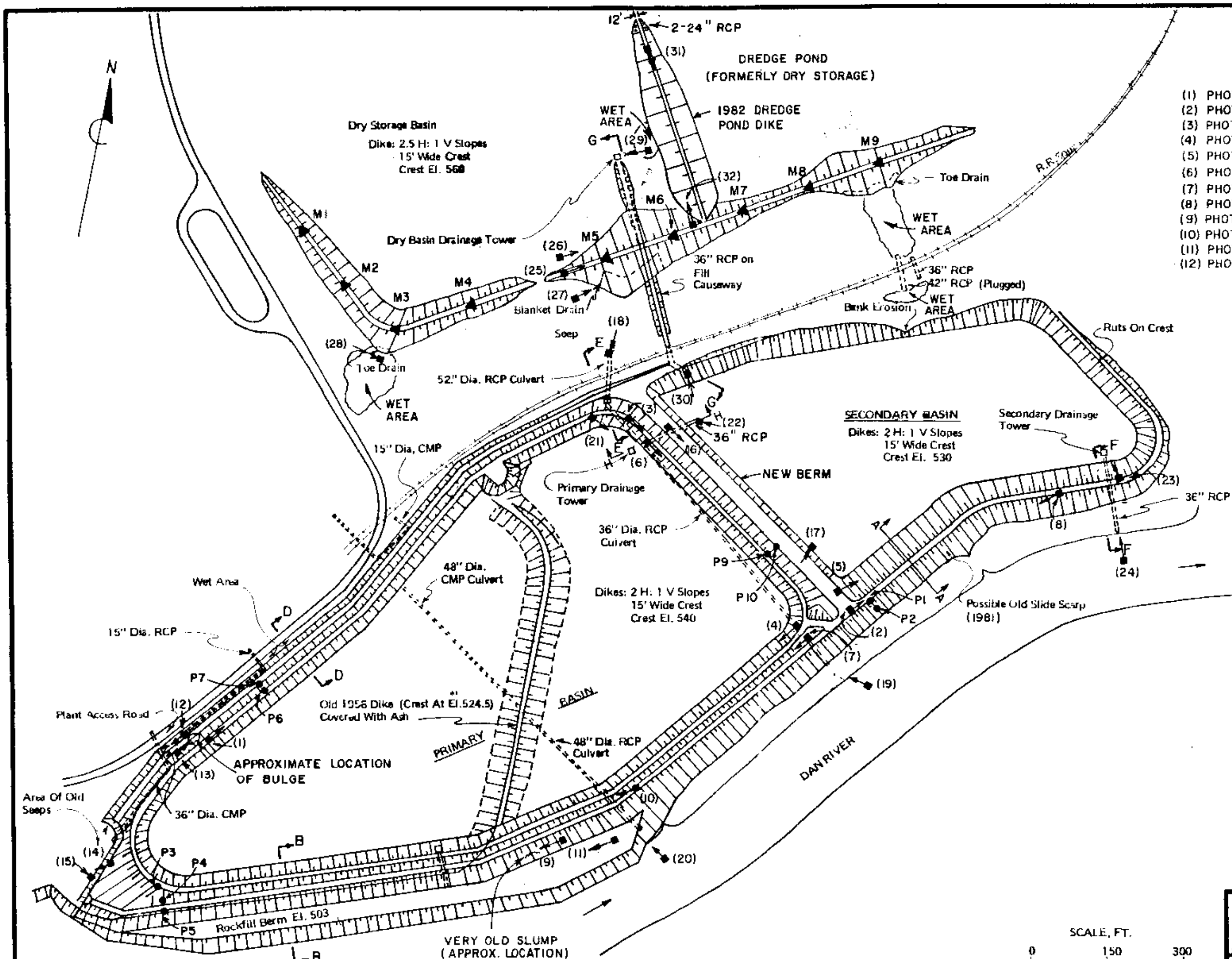

Ref: Thayer, Paul A., "Stratigraphy and Geology of Dan River Triassic Basin, North Carolina", Southeastern Geology, Vol. 12, No. 1, 1970.

**LAW ENGINEERING TESTING CO.**
**CHARLOTTE, NORTH CAROLINA**

Area Geology
Dan River Ash Storage Basins
Rockingham County, North Carolina

| | |
|---|---|
| **JOB NO.** CH 4581A | **FIGURE** 3 |

Reissued Under Job No. CHW 5475 (June 1986)




INDEX TO 1986 PHOTOGRAPHS

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| (2) PHOTO 4-2 | (14) PHOTO 4-14 | (26) PHOTO 4-26 |
| (3) PHOTO 4-3 | (15) PHOTO 4-15 | (27) PHOTO 4-27 |
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| (12) PHOTO 4-12 | (24) PHOTO 4-24 | |

NOTE: Arrows (■→) Show Approximate Direction of View of Photographs

INSTRUMENTATION SYMBOLS

▲ SETTLEMENT MONUMENT

• PIEZOMETER

Note: See Figures 5 and 6 For Sections

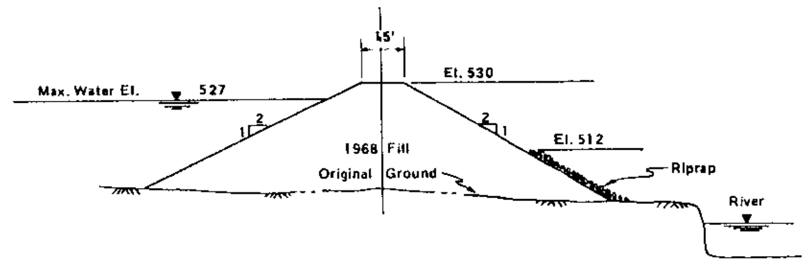
Ref: Duke Power Company Drawing No. D-1039M Latest Revision Dated 10-31-80

REVISED TO SHOW 1986 PHOTO LOCATIONS, INSTRUMENTATION LOCATIONS, NEW BERM, AND PERTINENT FIELD OBSERVATIONS.

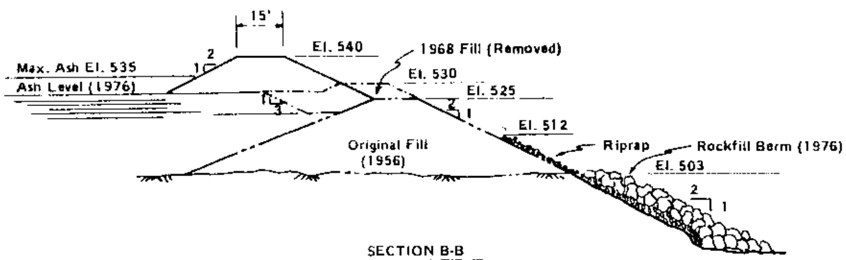
LAW ENGINEERING TESTING CO.
CHARLOTTE, NORTH CAROLINA

Plan Of Ash Retention Dikes
Dan River Ash Storage Basins
Rockingham County, North Carolina

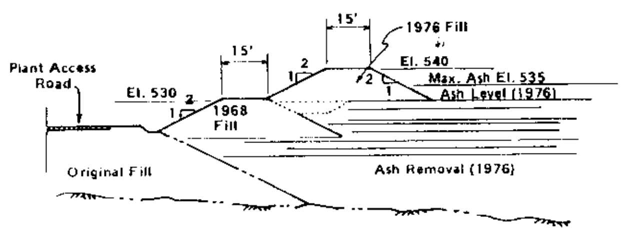
JOB NO. 4581A | FIGURE 4



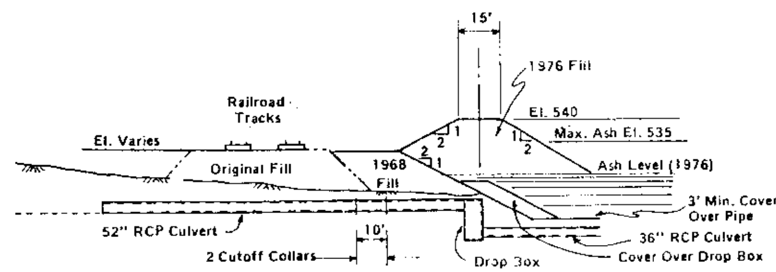

SECTION A-A



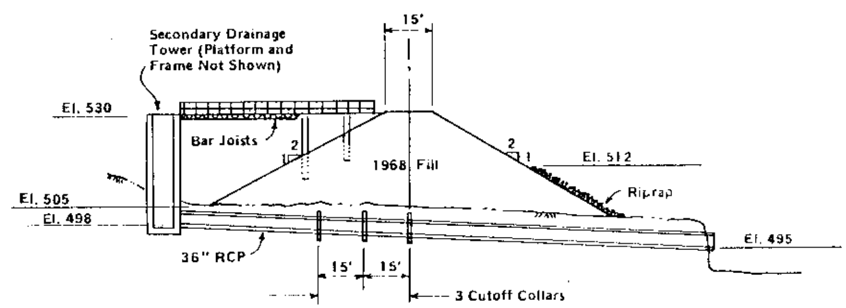

SECTION B-B




SECTION D-D




SECTION E-E




SECTION F-F

Note: See Figure 4 for Location of Sections

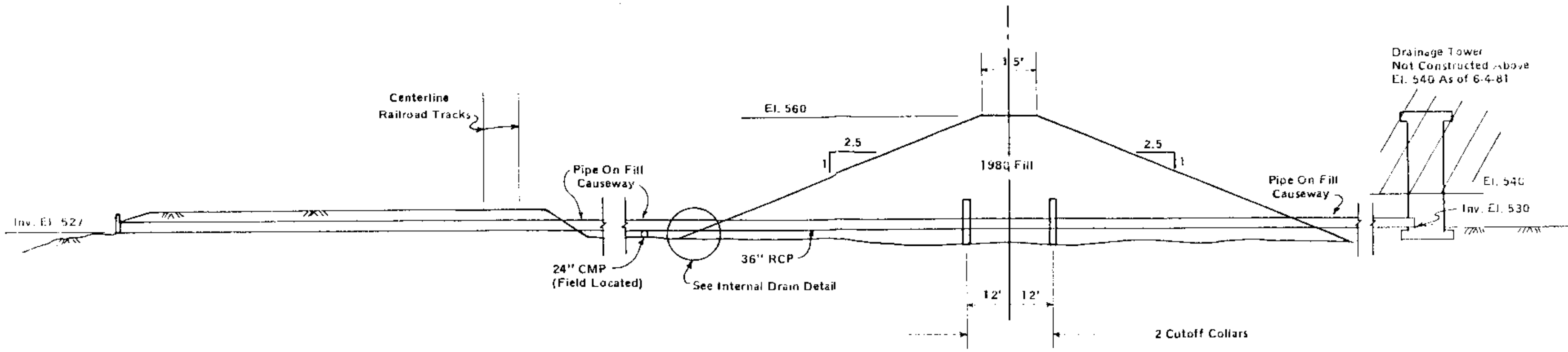
Ref: Duke Power Company Drawing No. D-1039-J, Latest Revision Dated 7-13-76

LAW ENGINEERING TESTING CO.
CHARLOTTE, NORTH CAROLINA

Sections Through Primary And Secondary Dikes
Dan River Ash Storage Basins
Rockingham County, North Carolina

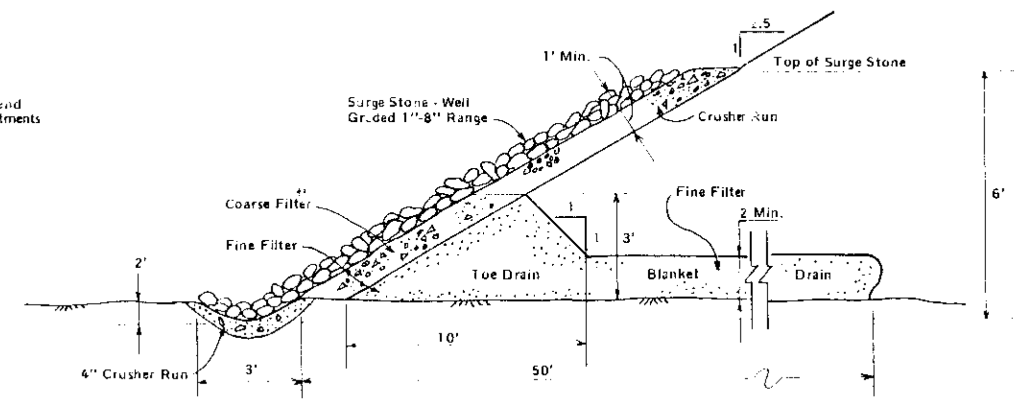
JOB NO. CH 4581A

FIGURE 5




SECTION G-G

Note: Blanket Drain to Extend Up to El. 530 on Abutments




TYPICAL INTERNAL DRAIN DETAILS

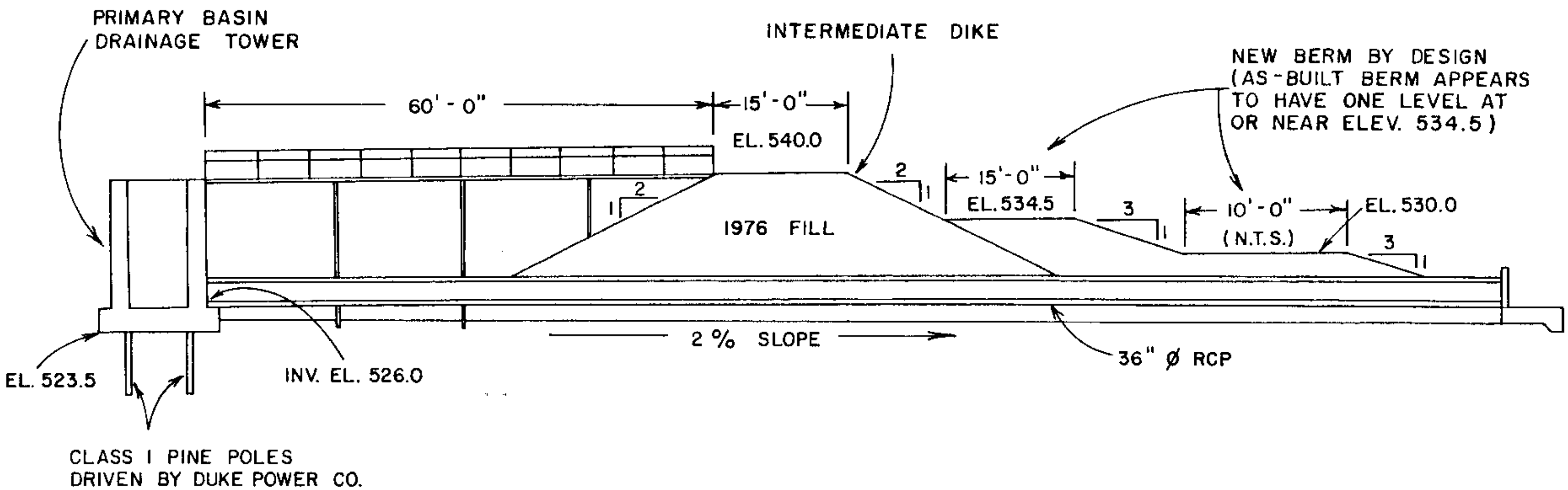
Note: See Figure 4 for Location of Section G-G

Ref: Duke Power Company Drawing No. D-1c39-M-1, Latest Revision Dated 11-7-80

**LAW ENGINEERING TESTING CO.**
**CHARLOTTE, NORTH CAROLINA**

Typical Section and Details-Dry Storage Basin Dike and Outlet
Dan River Ash Storage Basins
Rockingham County, North Carolina

| | |
|---|---|
| JOB NO. CH 4581A | FIGURE 6 |



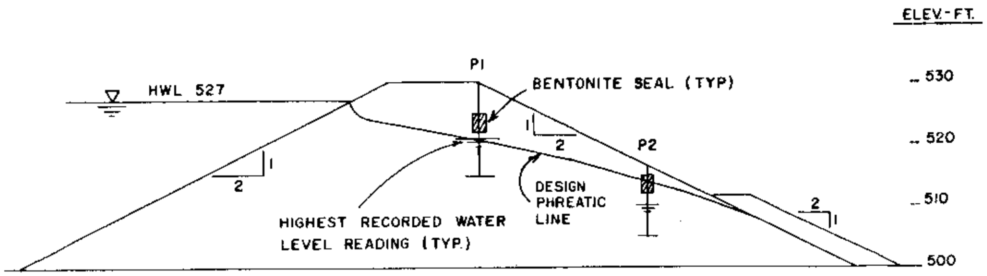

## SECTION H-H

(SEE FIG. 4 FOR LOCATION)

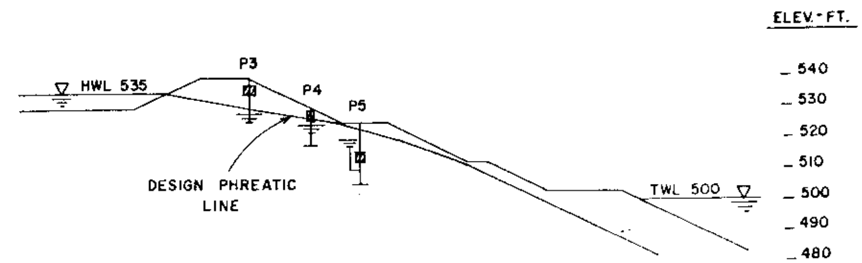
LAW ENGINEERING TESTING CO.
CHARLOTTE, NORTH CAROLINA

SECTION AT PRIMARY BASIN OUTLET
DAN RIVER ASH STORAGE BASINS
ROCKINGHAM COUNTY, N. C.

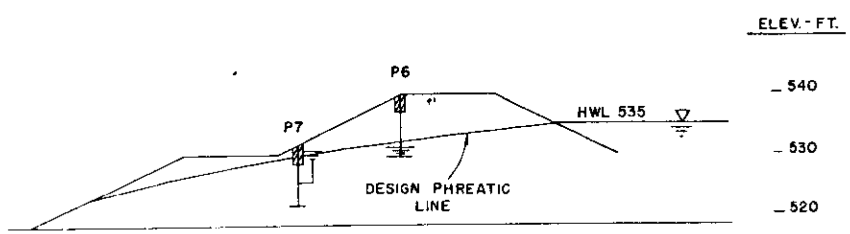
| | |
|---|---|
| JOB NO. CHW 5475 | FIGURE 7 |



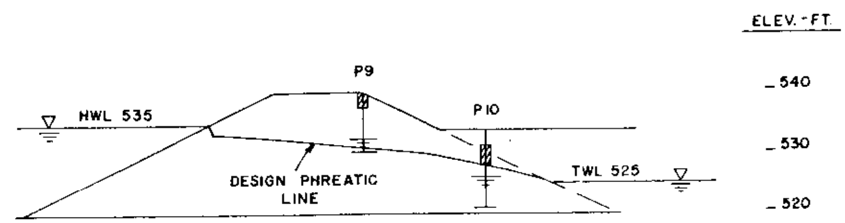

SECONDARY BASIN DIKE




PRIMARY BASIN DIKE
(NEXT TO RIVER)




PRIMARY BASIN DIKE
(NEXT TO PLANT ACCESS ROAD)




INTERMEDIATE DIKE
(NEW BERM NOT SHOWN)

LAW ENGINEERING TESTING CO.
CHARLOTTE, NORTH CAROLINA

PIEZOMETER READINGS AT SELECTED SECTIONS OF DAN RIVER ASH BASIN DIKES ROCKINGHAM COUNTY, N.C.

| JOB NO. CHW5475 | FIGURE 8 |
|---|---|

BRUNING 40-21 55592

# APPENDIX B

# PHOTOGRAPHS

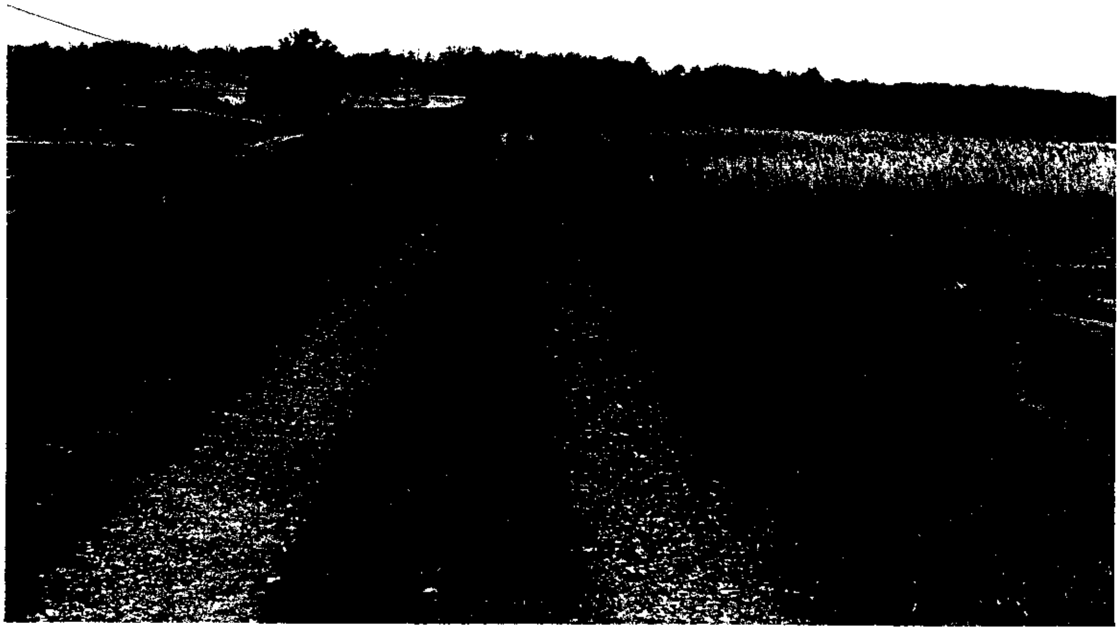


PHOTO 4-1 CREST OF PRIMARY BASIN DIKE
(TYPICAL VIEW)

PHOTO 4-2 CREST OF SECONDARY BASIN DIKE
(TYPICAL VIEW)

PHOTO 4-3 CREST OF INTERMEDIATE DIKE
(TYPICAL VIEW)

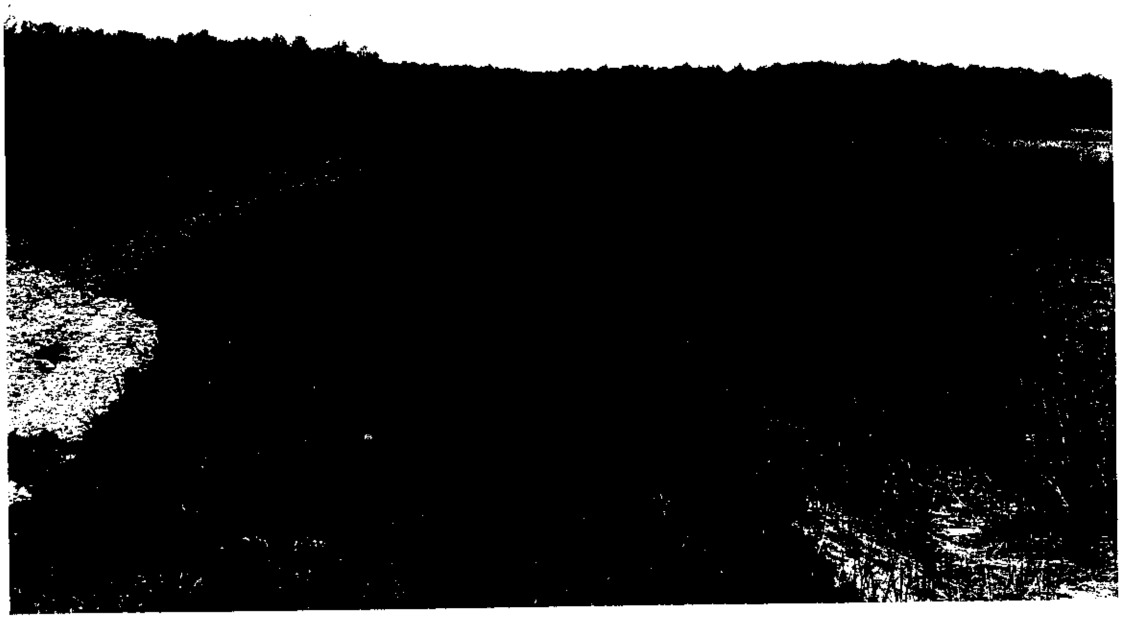


PHOTO 4-4 INSIDE SLOPE OF PRIMARY BASIN DIKE
(TYPICAL VIEW)

PHOTO 4-5 INSIDE SLOPE OF SECONDARY BASIN DIKE
(TYPICAL VIEW)

PHOTO 4-6 UPSTREAM SIDE OF INTERMEDIATE DIKE
(TYPICAL VIEW)

PHOTO 4-7 OUTSIDE SLOPE OF RAISED PORTION OF PRIMARY BASIN DIKE NEXT TO RIVER

PHOTO 4-8 OUTSIDE SLOPE OF SECONDARY BASIN DIKE NEXT TO RIVER

PHOTO 4-9 AREA OF OLD SLUMP ON OUTSIDE SLOPE OF PRIMARY BASIN DIKE ABOVE ROCKFILL BERM

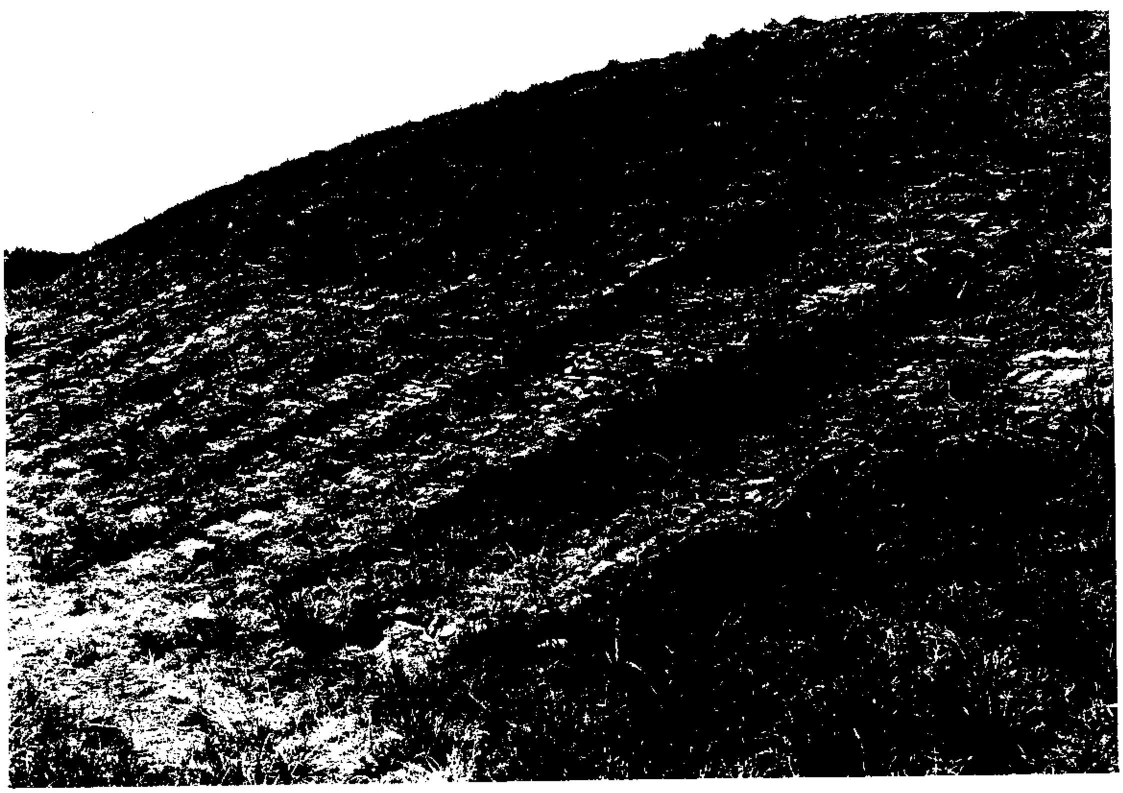


PHOTO 4-10 TYPICAL VIEW OF REPAIRS TO BARE SOIL AREAS ON PRIMARY BASIN OUTSIDE SLOPE (TRACKED WITH DOZER AND HYDROSEEDED)

PHOTO 4-11  ROCKFILL BERM AT BASE OF PRIMARY BASIN DIKE NEXT TO RIVER

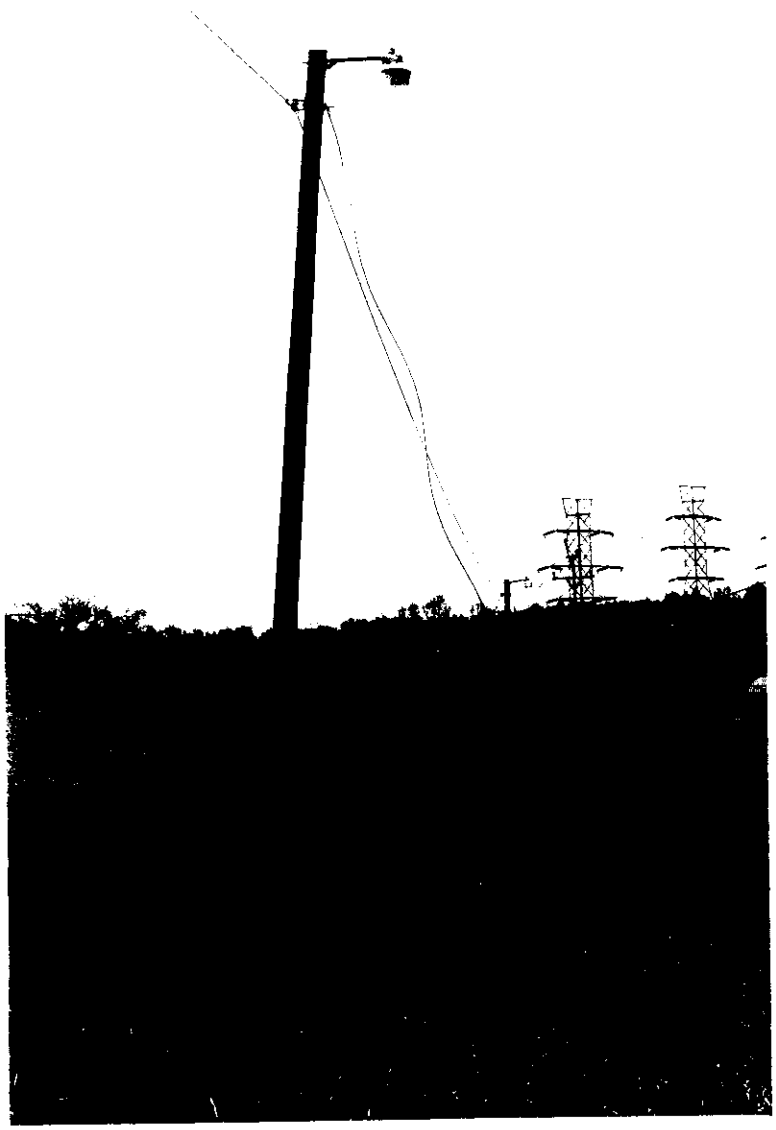


PHOTO 4-12 LEANING LIGHT POLE ON PRIMARY BASIN DIKE NEXT TO PLANT ENTRANCE ROAD

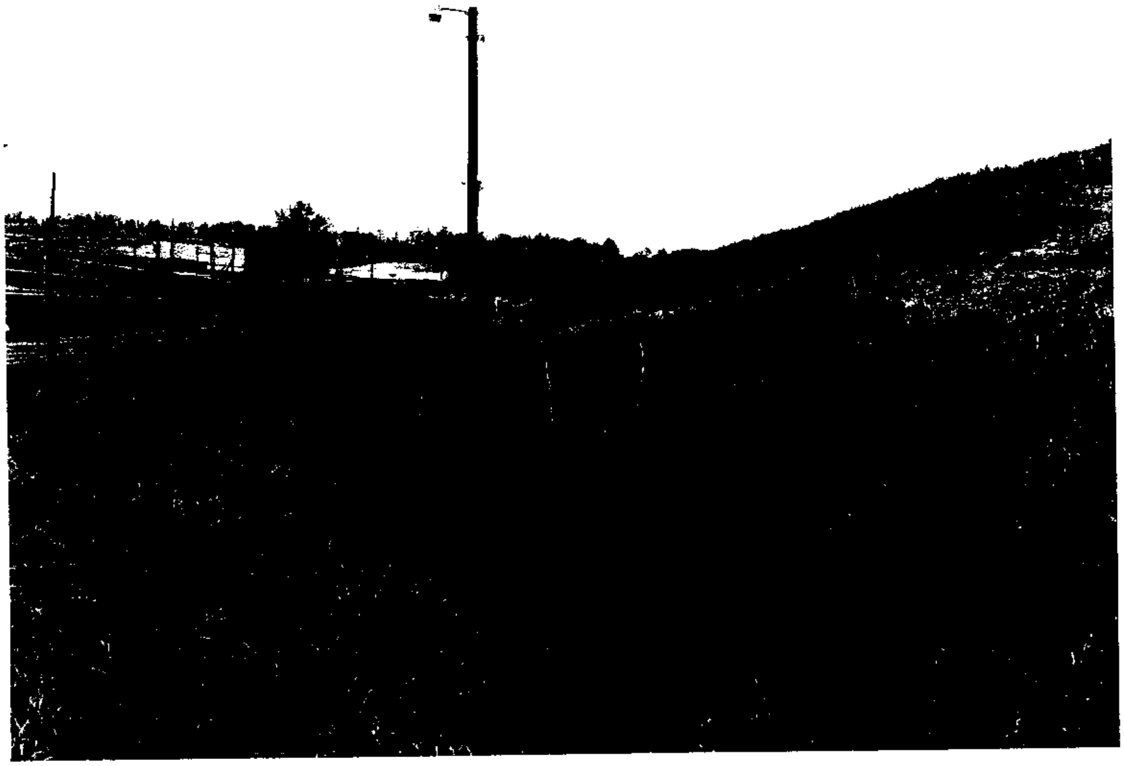


PHOTO 4-13 SLIGHT "BULGE" ON OUTSIDE SLOPE OF RAISED PORTION OF PRIMARY BASIN DIKE NEXT TO PLANT ACCESS ROAD

PHOTO 4-14 WET TOE AREA AT WEST END OF PRIMARY BASIN

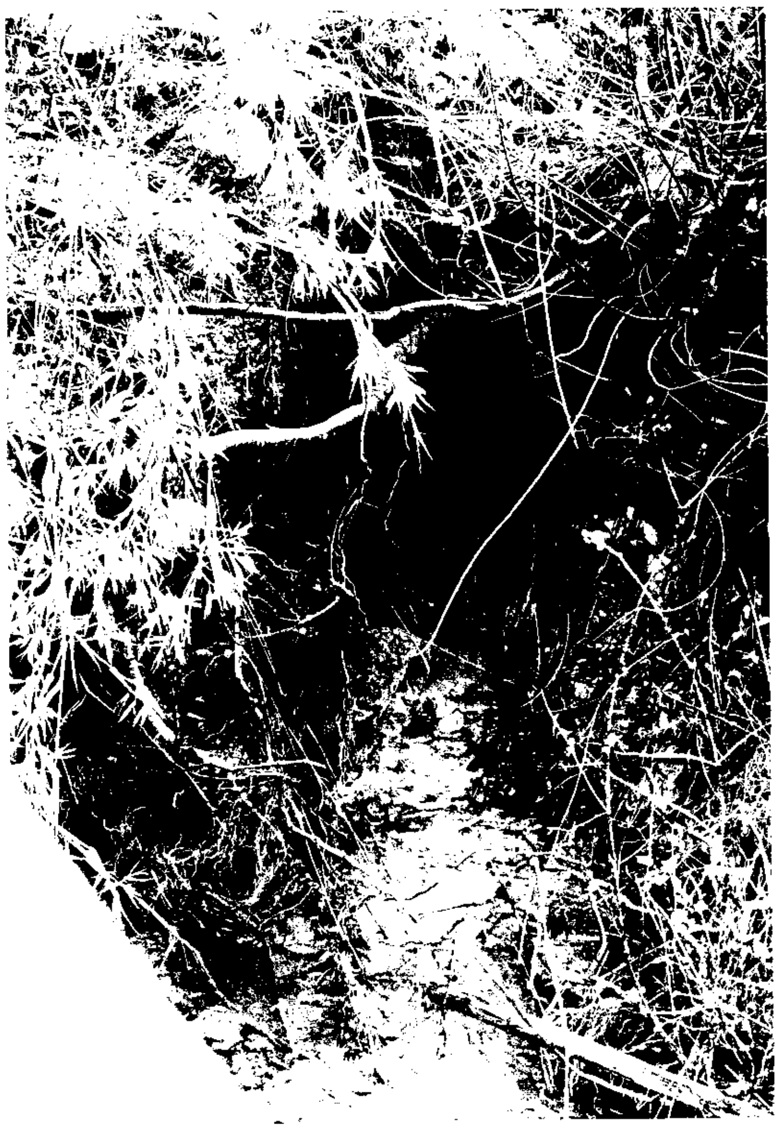


PHOTO 4-15 HOLE CAUSED BY SEEPAGE EROSION AT TOE OF DIKE IN WET AREA SHOWN IN PHOTO 4-14

PHOTO 4-16 NEW BERM ON SECONDARY BASIN SIDE OF INTERMEDIATE DIKE

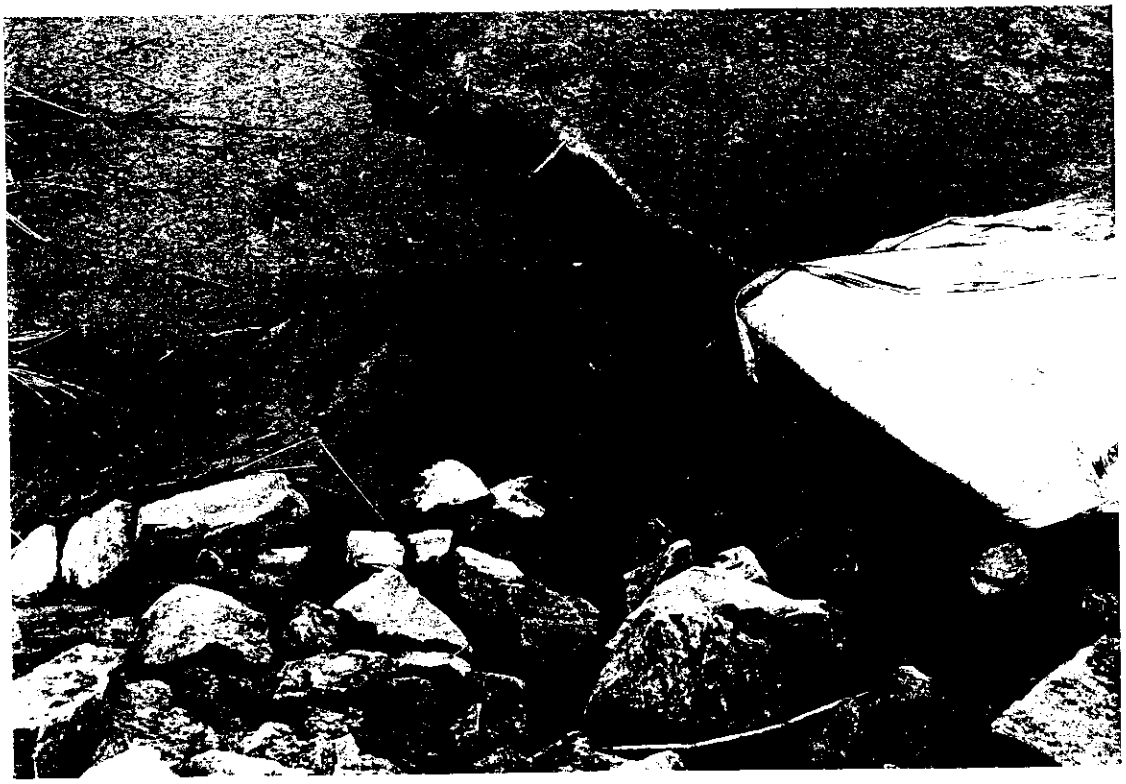


PHOTO 4-17 EVIDENCE OF SMALL BOILS IN ASH AT TOE OF NEW BERM (BOILS NOT ACTIVE) AND CLEAR SEEPAGE

PHOTO 4-18 ENTRANCE END OF 52/36-INCH RCP CULVERT

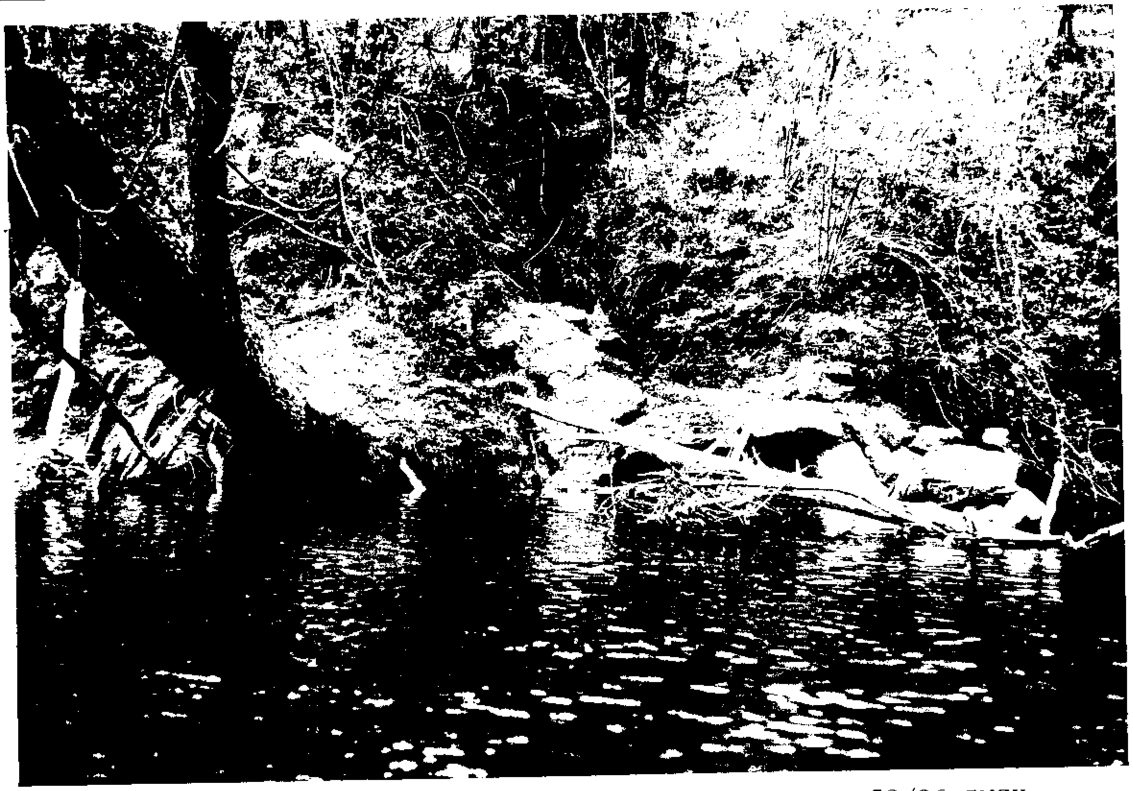


PHOTO 4-19 OUTLET END OF 36-INCH RCP SECTION OF 52/36-INCH CULVERT BENEATH ASH BASIN

PHOTO 4-20 OUTLET END OF 48-INCH CULVERT THAT PASSES BENEATH MIDDLE OF PRIMARY BASIN

PHOTO 4-21 VIEW OF PRIMARY BASIN DRAINAGE TOWER

PHOTO 4-22 OUTLET END OF 36-INCH RCP PRIMARY BASIN OUTLET

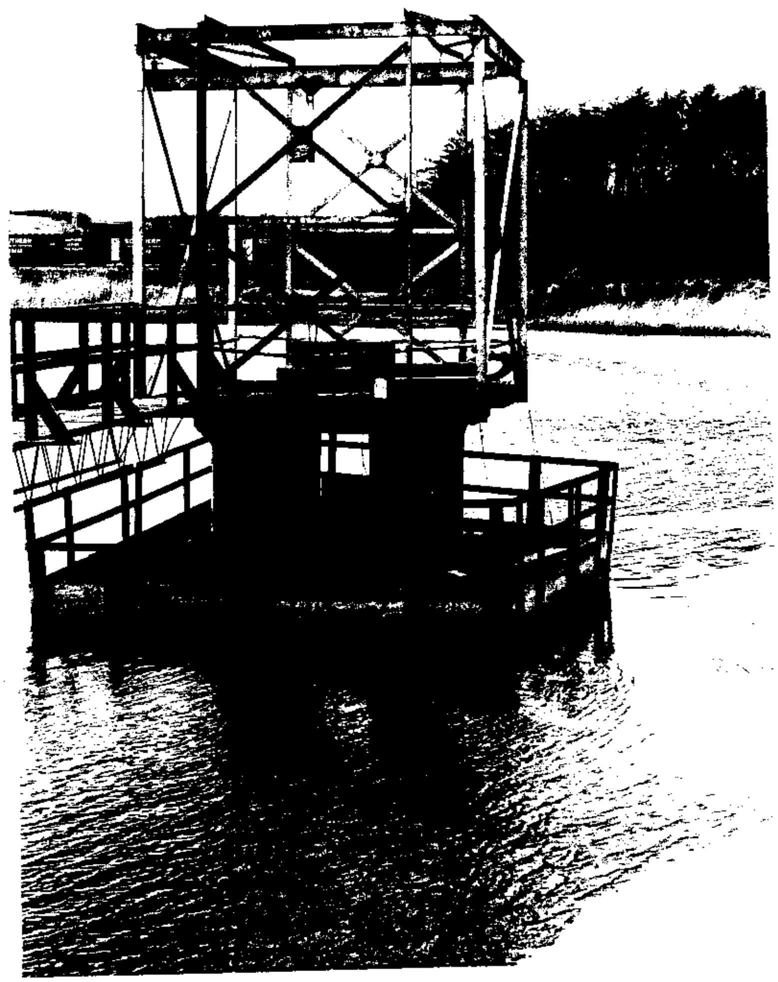


PHOTO 4-23 VIEW OF SECONDARY BASIN DRAINAGE TOWER

PHOTO 4-24  OUTLET END OF 36-INCH RCP SECONDARY BASIN OUTLET

PHOTO 4-25 CREST OF MAXIMUM HEIGHT SECTION OF DRY ASH STORAGE BASIN DIKE (SW TO NE VIEW)

PHOTO 4-26 UPSTREAM SLOPE OF MAXIMUM HEIGHT SECTION OF DRY ASH STORAGE BASIN DIKE (SW TO NE VIEW)

PHOTO 4-27 DOWNSTREAM SLOPE OF MAXIMUM HEIGHT SECTION OF DRY ASH STORAGE BASIN DIKE (SW TO NE VIEW)

PHOTO 4-28 WET DOWNSTREAM TOE AREA OF SOUTHWEST PORTION OF DRY ASH STORAGE BASIN DIKE

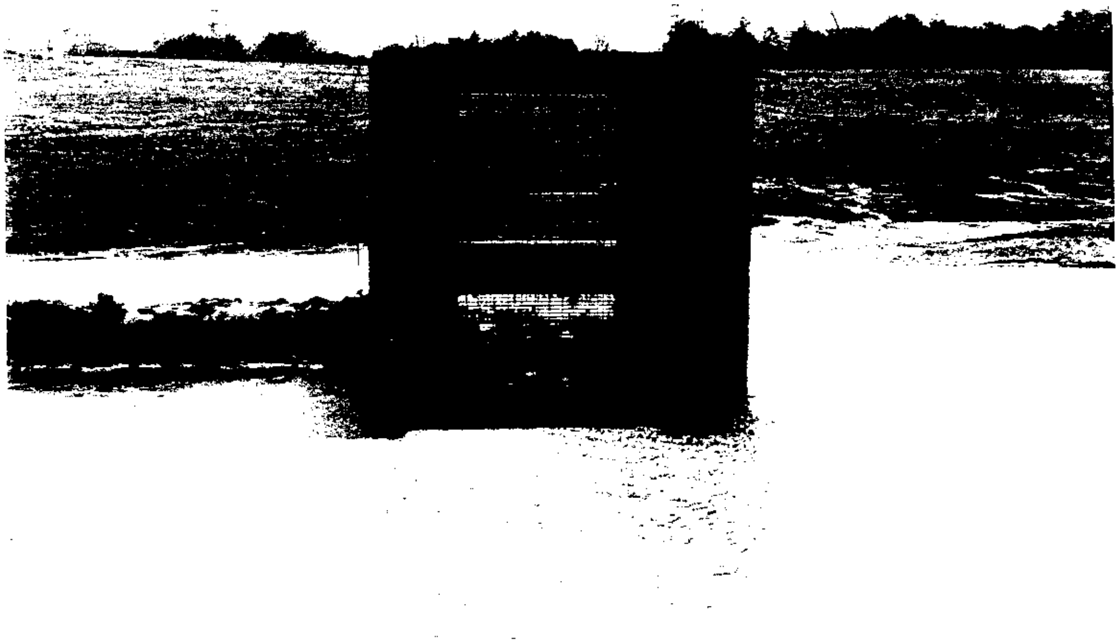


PHOTO 4-29 DRY ASH STORAGE BASIN DRAINAGE TOWER

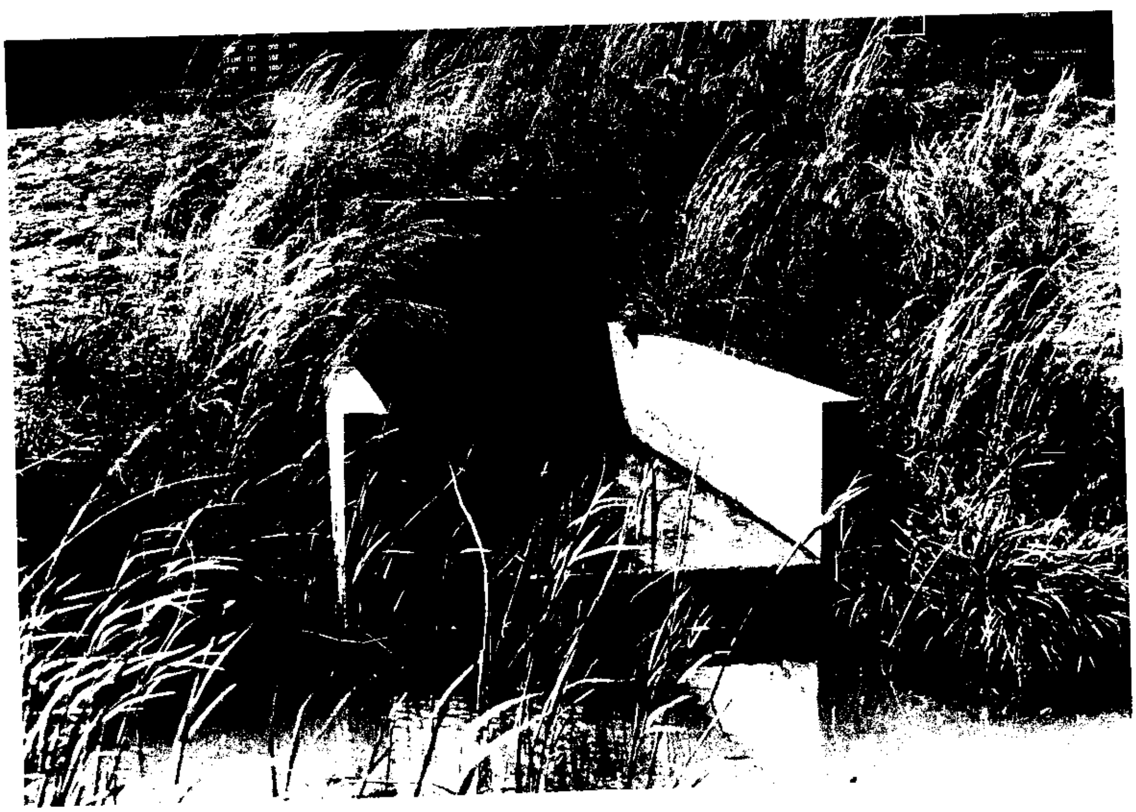


PHOTO 4-30 OUTLET END OF 36-INCH RCP OUTLET FOR DRY ASH STORAGE BASIN

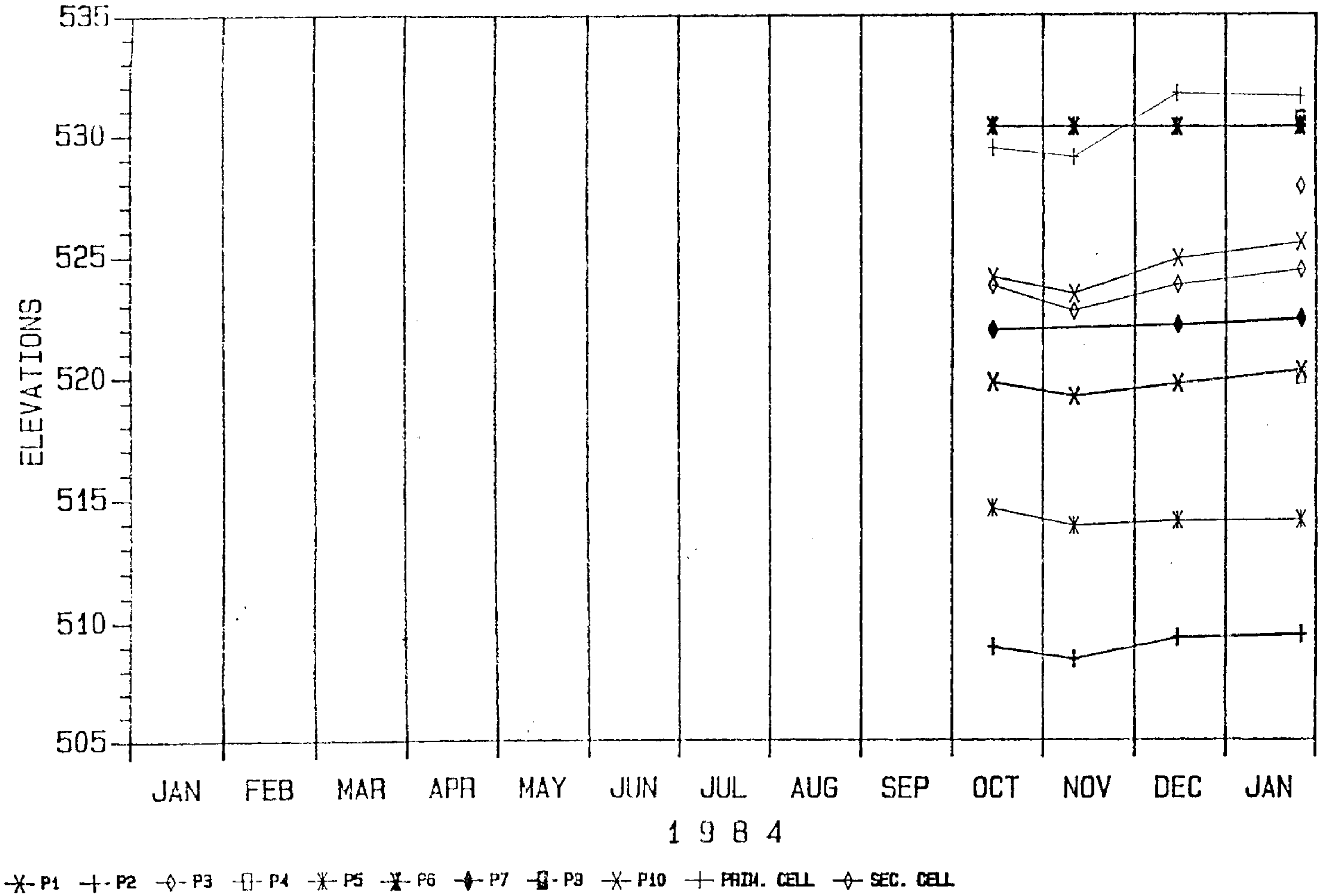
PHOTO 4-31 CREST OF DREDGE POND DIKE (NW TO SE VIEW)

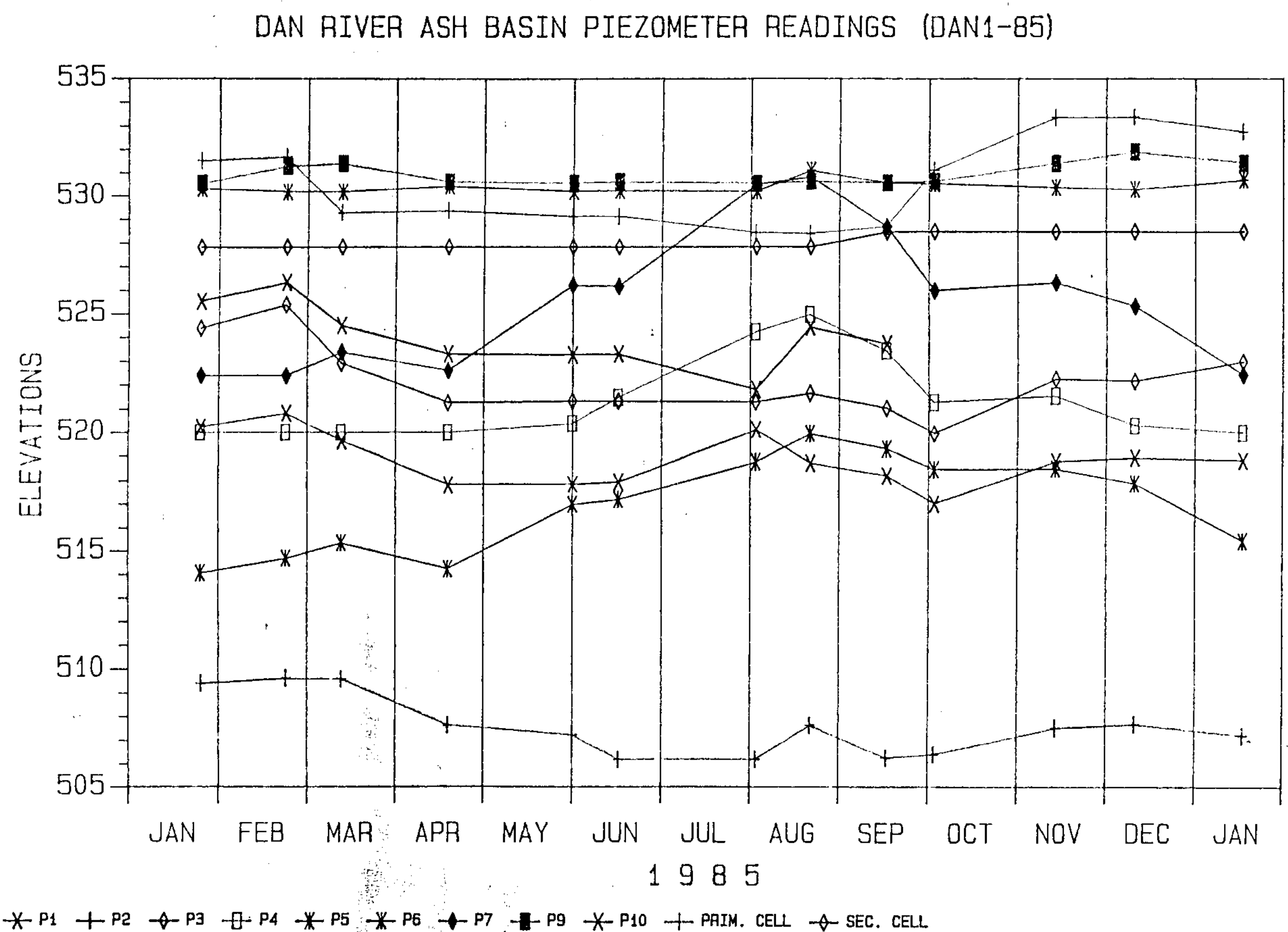
PHOTO 4-32 DOWNSTREAM SLOPE OF DREDGE POND DIKE (SE TO NW VIEW)

APPENDIX C

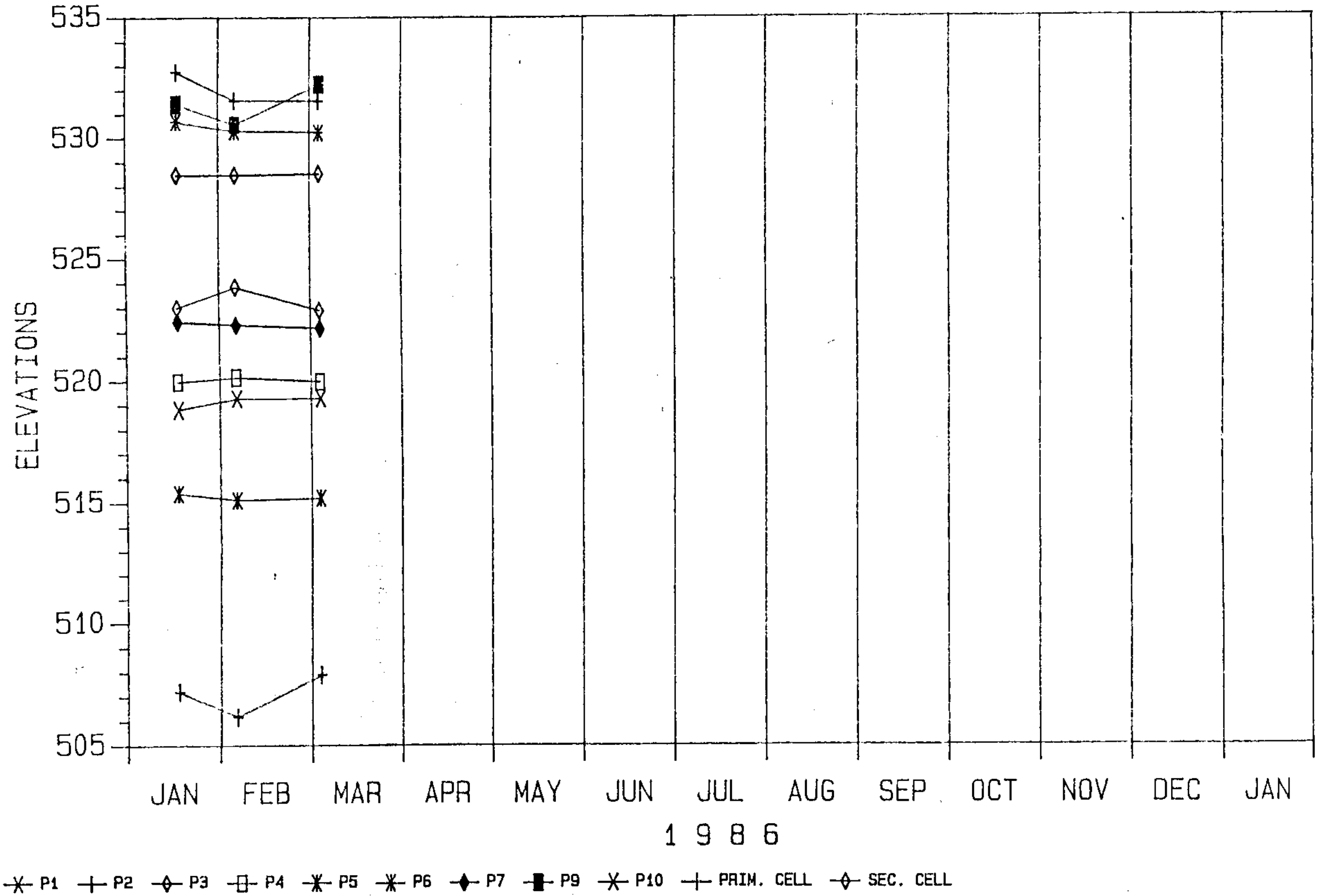
MONITORING DATA

DAN RIVER ASH BASIN PIEZOMETER READINGS (DAN1-84)




DAN RIVER ASH BASIN PIEZOMETER READINGS (DAN1-85)
535
530
525
520
515
510
505
ELEVATIONS
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
JAN
1 9 8 5
P1
P2
P3
P4
P5
P6
P7
P9
P10
PRIM. CELL
SEC. CELL

DAN RIVER ASH BASIN PIEZOMETER READINGS (DAN1-86)



TRM

DAN RIVER ASH BASIN
PIEZOMETER READINGS

Name of Observer: R. PRICE - K. PRICE    Date of Observation: 10-12-84

| Piezometer Number | Drawing | Section | Line | Station | Location | Elevation Top of Piezometer Tube | Basin Water Elevation | Observed Distance Top of Casing Pipe To Water Surface | Elevation of Water In Piezometer |
|---|---|---|---|---|---|---|---|---|---|
| P1 | D-1039-M | A-A | R | 27+00 | Top of Dike Downstream Edge | 532.97 | 523.76 | 13.23 | 519.74 |
| P2 | D-1039-M | A-A | R | 27+00 | 30' Downstream from P1 | 518.19 | 523.76 | 9.29 | 508.90 |
| P3 | D-1039-M | B-B | A | 9+00 | Top of Dike Downstream Edge | 540.73 | 529.39 | TO BOTTOM *12.94 | * NO WATER |
| P4 | D-1039-M | B-B | A | 9+00 | 20' Downstream from P3 | 532.42 | 529.39 | TO BOTTOM *12.69 | *NO WATER |
| P5 | D-1039-M | B-B | A | 9+00 | 35' Downstream from P3 | 528.22 | 529.39 | 13.67 | 514.55 |
| P6 | D-1039-M | D-D | L | 5+00 | Top of Dike Downstream Edge | 543.10 | 529.39 | 12.78 | 530.32 |
| P7 | D-1039-M | D-D | L | 5+00 | 20' Downstream from P6 | 535.05 | 529.39 | 13.11 | 521.94 |
| P9 | D-1039-M | G-G | M | 2+50 | Top of Dike Downstream Edge | 543.09 | 529.39 | TO BOTTOM *12.61 | *No WATER |
| P10 | D-1039-M | G-G | M | 2+50 | 20' Downstream from P9 | 532.50 | 529.39 | 8.40 | 524.10 |

Notes: 1) All water elevations to be read correct to .01 feet.
2) Frequency of observations: monthly intervals
3) Send one copy of completed readings to: R. S. Bhatnagar
Design Engineering

SBC

CIVIL SUPPORT SECTION
RECEIVED
OCT 16 1984
COPY
FILE NO. DS-6

DS-26A

DAN RIVER ASH BASIN
PIEZOMETER READINGS



TRM

Name of Observer: Price, R. & Gilbert    Date of Observation: Nov 9, 1984

| Piezometer Number | Drawing | Section | Line | Station | Location | Elevation Top of Piezometer Tube | Basin Water Elevation | Observed Distance Top of Casing Pipe To Water Surface | Elevation of Water In Piezometer |
|---|---|---|---|---|---|---|---|---|---|
| P1 | D-1039-M | A-A | R | 27+00 | Top of Dike Downstream Edge | 532.97 | 522.70 water | 13.82 | 519.15 |
| P2 | D-1039-M | A-A | R | 27+00 | 30' Downstream from P1 | 518.19 | 522.70 water | 9.82 | 508.37 |
| P3 | D-1039-M | B-B | A | 9+00 | Top of Dike Downstream Edge | 540.73 | 529.00 No water | 13.00 | No water |
| P4 | D-1039-M | B-B | A | 9+00 | 20' Downstream from P3 | 532.42 | 529.00 No water | 12.52 | No water |
| P5 | D-1039-M | B-B | A | 9+00 | 35' Downstream from P3 | 528.22 | 529.00 water | 14.41 | 513.81 |
| P6 | D-1039-M | D-D | L | 5+00 | Top of Dike Downstream Edge | 543.10 | 529.00 water | 12.79 | 530.31 |
| P7 | D-1039-M | D-D | L | 5+00 | 20' Downstream from P6 | 535.05 | 529.00 No water | 13.03 | No water |
| P9 | D-1039-M | G-G | M | 2+50 | Top of Dike Downstream Edge | 543.09 | 529.00 No water | 12.61 | No water |
| P10 | D-1039-M | G-G | M | 2+50 | 20' Downstream from P9 | 532.50 | 529.00 water | 9.10 | 523.40 |

Primary cell dike - 11.00 Ft
secondary cell dike - 7.30 Ft

Notes:
1) All water elevations to be read correct to .01 feet.
2) Frequency of observations: monthly intervals
3) Send one copy of completed readings to: R. S. Bhatnagar
Design Engineering

TRM

DAN RIVE SH BASIN
PIEZOMETE READINGS

Name of Observer: Price R & Price K    Date of Observation: Dec 14, 1984

| Piezometer Number | Drawing | Section | Line | Station | Location | Elevation Top of Piezometer Tube | Basin Water Elevation | Observed Distance Top of Casing Pipe To Water Surface | Elevation of Water In Piezometer |
|---|---|---|---|---|---|---|---|---|---|
| P1 | D-1039-M | A-A | R | 27+00 | Top of Dike Downstream Edge | 532.97 | 523.79 | 13.28 | 519.69 |
| P2 | D-1039-M | A-A | R | 27+00 | 30' Downstream from P1 | 518.19 | 523.79 | 8.90 | 509.29 |
| P3 | D-1039-M | B-B | A | 9+00 | Top of Dike Downstream Edge | 540.73 | 531.65 | 12.89 | No Water |
| P4 | D-1039-M | B-B | A | 9+00 | 20' Downstream from P3 | 532.42 | 531.65 | 12.60 | Dry 519.82 |
| P5 | D-1039-M | B-B | A | 9+00 | 35' Downstream from P3 | 528.22 | 531.65 | 14.19 | 514.03 |
| P6 | D-1039-M | D-D | L | 5+00 | Top of Dike Downstream Edge | 543.10 | 531.65 | 12.81 | 530.29 |
| P7 | D-1039-M | D-D | L | 5+00 | 20' Downstream from P6 | 535.05 | 531.65 | 12.91 | 522.14 |
| P9 | D-1039-M | G-G | M | 2+50 | Top of Dike Downstream Edge | 543.09 | 531.65 | 12.64 | No Water |
| P10 | D-1039-M | G-G | M | 2+50 | 20' Downstream from P9 | 532.50 | 531.65 | 7.63 | 524.87 |

~~BSB~~/SGC

Notes: 1) All water elevations to be read correct to .01 feet.
2) Frequency of observations: monthly intervals
3) Send one copy of completed readings to: R. S. Bhatnagar
Design Engineering
EC-09G2

(Secondary) Lower Basin to water = 6.21 Ft
(Primary) Upper Basin to water = 8.35 Ft

BSB SGC
File No: DS-176

DAN RIVER STEAM STATION
DUKE POWER COMPANY
ASH BASIN PIEZOMETER READINGS

TRM

| ELEVATION TOP OF CASING PIPE (Ft.) | NUMBER | TYPE | DATE: 1-25-85 POND ELEV.: 531.52 OBSERVER: AW/RP /524.40 | DATE: 2-22-85 POND ELEV.: 531.69 OBSERVER: AW/KP /525.38 | DATE: 3-12-85 POND ELEV.: 529.27 OBSERVER: RP/KP /522.92 | DATE: 4-18-85 POND ELEV.: 529.36 OBSERVER: KP/AW /521.25 | DATE: 5-31-85 POND ELEV.: 529.11 OBSERVER: TG/RP /521.33 | DATE: 6-14-85 POND ELEV.: 529.11 OBSERVER: TG/AW /521.33 | DATE: 8-2-85 POND ELEV.: 528.42 OBSERVER: KP/AW /521.28 | DATE: 8/21/85 POND ELEV.: 528.38 OBSERVER: KP/TR /521.65 | DATE: 9/16/85 POND ELEV.: 529.18 OBSERVER: KP/TG /521.02 | DATE: 10/2/85 POND ELEV.: 531.12 OBSERVER: RP/KP /519.94 | DATE: 11/13/85 POND ELEV.: 533.38 OBSERVER: RP/TG /522.25 | DATE: 12/11/85 POND ELEV.: 533.39 OBSERVER: KP/AW /522.16 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|

DISTANCE FROM TOP OF CASING PIPE TO WATER SURFACE (Ft.)

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 532.97 | 1 | P | 12.74 | 12.16 | 13.34 | 15.17 | 15.15 | 15.04 | 12.86 | 14.28 | 14.81 | 16.00 | 14.19 | 14.03 |
| 519.19 | 2 | P | 8.77 | 8.56 | 8.59 | 10.56 | 10.96 | 12.00□ | 12.00□ | 10.55 | 11.94 | 11.78 | 10.67 | 10.51 |
| 540.73 | 3 | P | 12.91□ | 12.91□ | 12.91□ | 12.91□ | 12.91□ | 12.91□ | 12.91□ | 12.91□ | 12.26△ | 12.24△ | 12.25△ | 12.24△ |
| 532.42 | 4 | P | 12.42□ | 12.42□ | 12.42□ | 12.42□ | 12.04 | 10.94 | 8.20 | 7.47 | 9.00 | 11.16 | 10.88 | 12.14 |
| 528.22 | 5 | P | 14.15 | 13.53 | 12.88 | 14.00 | 11.25 | 11.04 | 9.46 | 8.28 | 8.92 | 9.79 | 9.75 | 10.39 |
| 543.10 | 6 | P | 12.78 | 12.92□ | 12.92□ | 12.67 | 12.92□ | 12.88△ | 12.91□ | 12.00 | 12.55 | 12.58 | 12.74 | 12.82△ |
| 535.05 | 7 | P | 12.65 | 12.67 | 11.67 | 12.46 | 8.85 | 8.90 | 4.55 | 4.23 | 6.38 | 9.09 | 8.75 | 9.75 |
| | | | | | | | | | | | | | | |
| 543.09 | 9 | P | 12.52□ | 11.79 | 11.67 | 12.47△ | 12.52□ | 12.46△ | 12.55□ | 12.46△ | 12.52□ | 12.45△ | 11.66 | 11.17 |
| 532.50 | 10 | P | 6.98 | 6.18 | 8.02 | 9.22 | 9.25 | 9.21 | 10.72△ | 8.08 | 8.79 | * | * | * |

WATER ELEVATIONS (Ft.)

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | P | 520.23 | 520.81 | 519.63 | 517.80 | 517.82 | 517.93 | 520.11 | 518.69 | 518.16 | 516.97 | 518.78 | 518.94 |
| 2 | P | 509.42 | 509.63 | 509.60 | 507.63 | 507.23 | 506.19 | 506.19 | 507.64 | 506.25 | 506.41 | 507.52 | 507.68 |
| 3 | P | 527.82 | 527.82 | 527.82 | 527.82 | 527.82 | 527.82 | 527.82 | 527.82 | 528.47 | 528.49 | 528.48 | 528.49 |
| 4 | P | 520.00 | 520.00 | 520.00 | 520.00 | 520.38 | 521.48 | 524.22 | 524.95 | 523.42 | 521.26 | 521.54 | 520.28 |
| 5 | P | 514.07 | 514.69 | 515.34 | 514.22 | 516.97 | 517.18 | 518.76 | 519.94 | 519.30 | 518.43 | 518.47 | 517.83 |
| 6 | P | 530.32 | 530.18 | 530.18 | 530.43 | 530.18 | 530.22 | 530.19 | 531.10 | 530.55 | 530.52 | 530.36 | 530.28 |
| 7 | P | 522.40 | 522.38 | 523.38 | 522.59 | 526.20 | 526.15 | 530.50 | 530.82 | 528.67 | 525.96 | 526.30 | 525.30 |
| | | | | | | | | | | | | |
| 9 | P | 530.57 | 531.30 | 531.42 | 530.62 | 530.57 | 530.63 | 530.54 | 530.63 | 530.57 | 530.64 | 531.43 | 531.92 |
| 10 | P | 525.52 | 526.32 | 524.48 | 523.28 | 523.25 | 523.29 | 521.78 | 524.42 | 523.71 | * | * | * |

| PARSHALL FLUME | (NOT APPLICABLE) |
|---|---|

CIVIL SUPPORT SECTION
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FEB 0 7 1986
FILE NO.

NOTES:
1. All water surface elevations are to be correct to 0.01 of a foot.
2. Parshall Flume reading shall be gallons per minute and correct to the nearest 0.01 gpm.
3. Type: P - Piezometer, OW - Observation Well.
4. All Piezometer/Observation Wells are to be completed with an elevation. If no water exist, footnote the depth as follows: △ Wet silty bottm, □ Dry silt, * Hard bottom
5. Send a copy of completed forms, page 1 and 2, to: Design Engineering, Mr. S. B. Hager, Chief Engineer, Attention: M. S. Sills.
6. Station to retain original and complete the next column at the next monitoring interval.

* Level can not be read due to basin dike construction.

DAN RIVER STEAM STATION
DUKE POWER COMPANY
ASH BASIN PIEZOMETER READINGS

File No: DS-176 TRM

CIVIL SUPPORT SECTION
RECEIVED
MAR 05 1986

| ELEVATION TOP OF CASING PIPE (Ft.) | NUMBER | TYPE | DATE: 1/17/86 POND ELEV.: 532.76 /523.00 OBSERVER: RP/KP | DATE: 2/5/86 POND ELEV.: 531.56 /523.84 OBSERVER: PG/KP | DATE: 3/3/86 POND ELEV.: 531.54 /522.88 OBSERVER: RP/PG | DATE: POND ELEV.: OBSERVER: |
|---|---|---|---|---|---|---|

**DISTANCE FROM TOP OF CASING PIPE TO WATER SURFACE (Ft.)**

| | | | | | | |
|---|---|---|---|---|---|---|
| 532.97 | 1 | P | 14.15 | 13.69 | 13.68 | |
| 518.19 | 2 | P | 10.98 | 12.00□ | 10.27 | |
| 540.73 | 3 | P | 12.25△ | 12.26△ | 12.21△ | |
| 532.42 | 4 | P | 12.44□ | 12.25△ | 12.44□ | |
| 528.22 | 5 | P | 12.83 | 13.12 | 13.03 | |
| 543.10 | 6 | P | 12.40 | 12.82△ | 12.89△ | |
| 535.05 | 7 | P | 12.63 | 12.77△ | 12.91△ | |
| | | | | | | |
| 543.09 | 9 | P | 11.65 | 12.52□ | 10.85 | |
| 532.50 | 10 | P | * | * | * | |

**WATER ELEVATIONS (Ft.)**

| | | | | | |
|---|---|---|---|---|---|
| 1 | P | 518.82 | 519.28 | 519.29 | |
| 2 | P | 507.21 | 506.19 | 507.92 | |
| 3 | P | 528.48 | 528.47 | 528.52 | |
| 4 | P | 519.98 | 520.17 | 519.98 | |
| 5 | P | 515.39 | 515.10 | 515.19 | |
| 6 | P | 530.70 | 530.28 | 530.21 | |
| 7 | P | 522.42 | 522.28 | 522.14 | |
| | | | | | |
| 9 | P | 531.44 | 530.57 | 532.24 | |
| 10 | P | * | * | * | |

| | |
|---|---|
| PARSHALL FLUME | (NOT APPLICABLE) |

NOTES:
1. All water surface elevations are to be correct to 0.01 of a foot.
2. Parshall Flume reading shall be gallons per minute and correct to the nearest 0.01 gpm.
3. Type: P - Piezometer, OW - Observation Well.
4. All Piezometer/Observation Wells are to be completed with an elevation. If no water exist, footnote the depth as follows: △ Wet silty bottm, □ Dry silt, * Hard bottom
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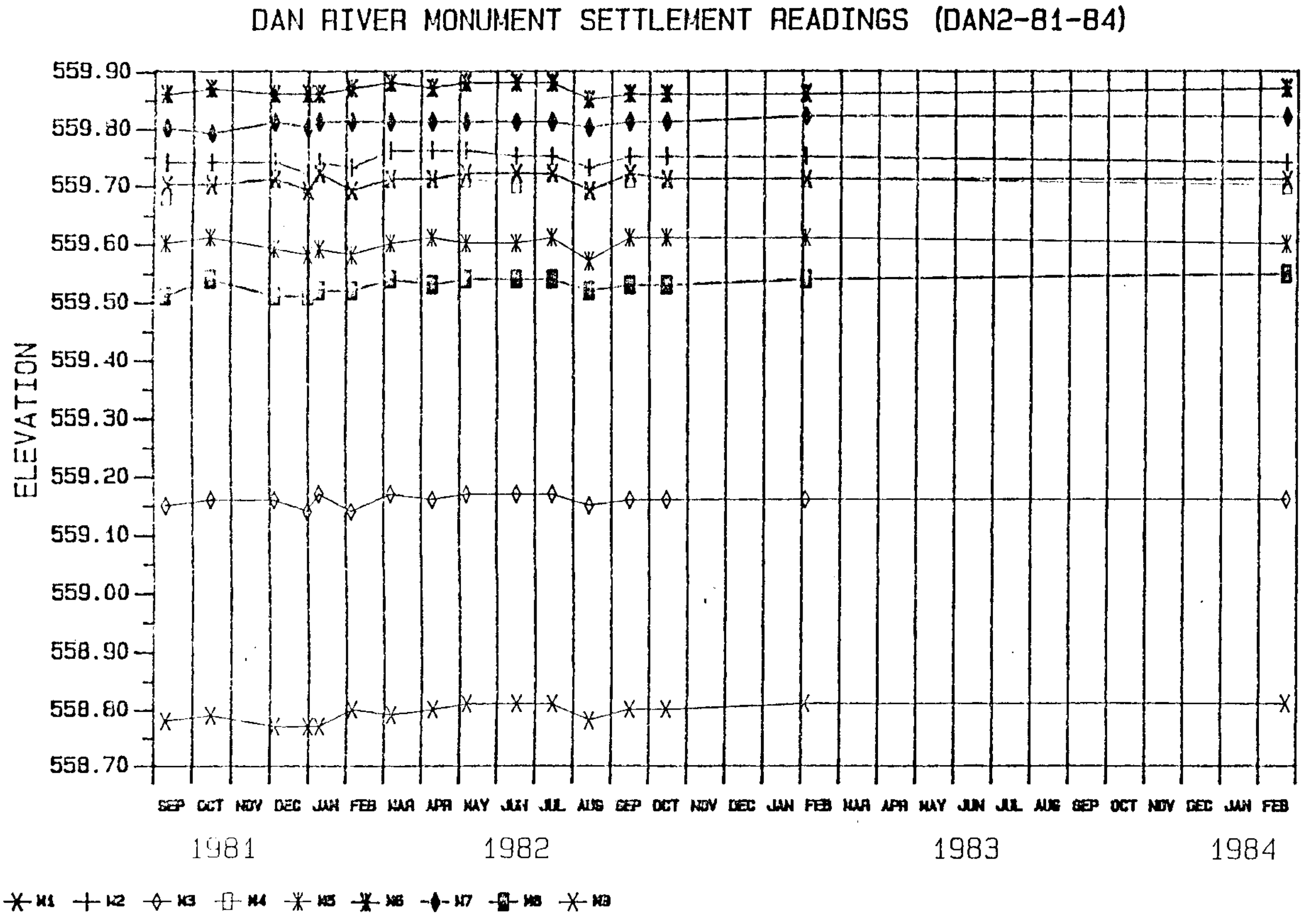
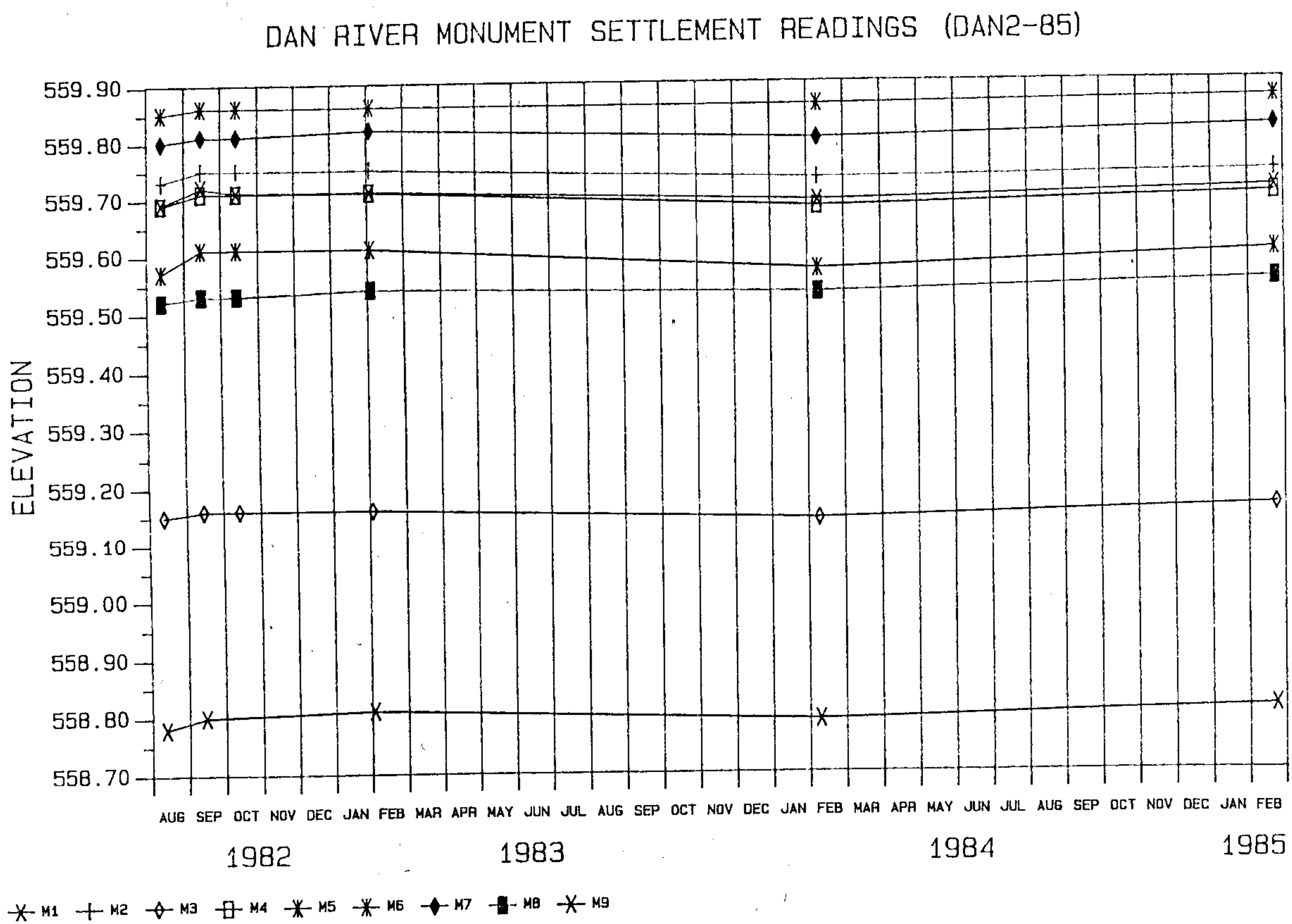

DAN RIVER MONUMENT SETTLEMENT READINGS (DAN2-81-84)
ELEVATION
559.90
559.80
559.70
559.60
559.50
559.40
559.30
559.20
559.10
559.00
558.90
558.80
558.70
SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB
1981
1982
1983
1984
M1 M2 M3 M4 M5 M6 M7 M8 M9


DAN RIVER MONUMENT SETTLEMENT READINGS (DAN2-85)
ELEVATION
559.90
559.80
559.70
559.60
559.50
559.40
559.30
559.20
559.10
559.00
558.90
558.80
558.70
AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB
1982
1983
1984
1985
M1
M2
M3
M4
M5
M6
M7
M8
M9

# DAN RIVER MONUMENT SETTLEMENT READINGS (DAN2-86)