File Pettit Dam
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REPORT OF ENGINEERING EVALUATION

PETIT COVE DAM

DAWSON COUNTY, GEORGIA

LETCO JOB NUMBER 7502

Law Engineering Testing Company

**LAW ENGINEERING TESTING COMPANY**
*Geotechnical and Materials Engineers*
412 PLASTERS AVENUE, N.E. ATLANTA, GEORGIA 30324 (404) 873-4761

March 18, 1974

Baldwin & Cranston Associates, Inc.
1103 Green Street
Augusta, Georgia 30902

Attention: Mr. T. H. Robertson
Project Engineer

Subject: Report of Engineering Evaluation
Petit Cove Dam
Dawson County, Georgia
LETCO Job Number 7502

Gentlemen:

Law Engineering Testing Company has completed the authorized subsurface investigation and engineering evaluation of seepage from the Petit Cove Dam in Dawson County, Georgia. The dam, which is approximately 125 feet high and approximately 900 feet long, is part of the Big Canoe project, and is located approximately 65 miles north of Atlanta. This report presents the findings of the investigation along with our evaluations and recommendations.

Our initial observations of the dam indicated a spring in the right abutment approximately 100 feet from the embankment face. The embankment face was wet on the right side up to approximately the second berm. Major seepage of water was observed at the intersection of the embankment face and the left abutment. We were informed that the measured flow was approximately 50 gallons per minute.

The purpose of this investigation was to determine the most probable seepage paths and to recommend appropriate solutions for controlling flow of this water. The investigation consisted of soil test borings on the dam abutments, visual observation of the dam, the installation of piezometers on the downstream face of the dam, data review of pertinent material regarding the construction of the dam, and discussions with Mr. William E. Edens of Baldwin & Cranston Associates in regard to construction of the dam.

## INVESTIGATIVE PROCEDURES

Field Investigation: The nature and consistency of the abutment soils were investigated by two soil test borings. The borings were drilled at locations shown on the accompanying site plan. The borings were located in the field by our drill crew by taping distances and estimating right angles from prominent site features. Standard penetration tests were performed at regular intervals within the borings. The drilling and sampling procedures used conformed to ASTM Designation D 1586-70.

Rock was cored in both of the soil test borings to determine the nature and continuity of the rock underlying the site. The procedures used during rock coring conformed to ASTM Designation D 2113-62.

Test boring records are attached graphically showing soil descriptions, ground surface elevations, and penetration resistances. Rock descriptions, recovery percentages, and the bit size used are shown on the accompanying core boring records. The method of soil test boring and rock coring are discussed in more detail in the Appendix.

Piezometers: The phreatic surface on the downstream face of the dam was measured by piezometers. Five piezometers were installed on the downstream berms by augering 20 feet into the dam. A casing (2 inch diameter minimum) was inserted into the auger boring, and a small diameter pipe was located within the casing. The small diameter pipe extends below the outside casing pipe and is seated into a well graded sand. The sand is used to fill the void created by the pipes. A bentonite clay is used for sealing the void from rising water.

## AREA GEOLOGY

Dawson County is located in the southern Piedmont Physiographic Region, which contains the oldest geologic formations in the southeastern United States. Since their origin, over 600 million years ago, these Precambrian rocks have undergone repeated cycles of metamorphism, folding, faulting, and intrusion.

The parent rocks in this region are composed primarily of quartz, feldspar, mica, and a wide variety of the dark minerals such as hornblende and the various pyroxenes. Because of their crystalline structure, chemical decomposition occurs first as a breakdown along boundaries of individual mineral crystals. As a result, partially weathered rock has the appearance of a dense sand. The crystals occupy the same position as they occupied in the original rock. With further weathering, the individual crystals are attacked and the mass becomes a micaceous silty sand or a micaceous sandy silt in which the original banding of the parent rock is apparent but in which the original crystalline structure is obscured. Finally, in the more advanced stages of chemical weathering, the materials change into a red or reddish brown silty clay or clayey silt. The original structure and banded appearance are lost. Soils formed by this chemical weathering process of the underlying rock are called "residual".

## SUBSURFACE CONDITIONS

Four major subsurface soils were encountered by our exploration. These are: 1.) fill soils, 2.) residual soils, 3.) very hard soils and partially weathered rock, and 4.) rock. At the ground surface, a thin layer of the gravel base course of the roadway was encountered in both the borings as shown on the boring logs.

Fill Soils: The dam embankment is compacted soil fill. Fill soils were encountered in both soil test borings and in all of the piezometer borings. Both soil test borings encountered approximately 1 foot of base course material for use in the roadway construction. The maximum depth of fill soil in the soil test borings was 8.5 feet in Boring B-2, while fill soil was encountered in all the piezometer borings to their termination depth. The consistency of the fill soils varied from hard to very stiff. The fill soil consisted of brown, fine to coarse, sandy micaceous silt in Borings B-1 and B-2, while in piezometers P-1 through P-5 the soil was a brown gravelly micaceous silt. Small rock boulders were encountered within some of the piezometer borings.

Residual Soils: Both soil test borings encountered residual soils. Residual soils are those soils which have been formed by the in-place weathering of the crystalline rock. The residuum at the site is generally very stiff to hard brown to reddish brown fine sandy micaceous silt. The upper residual soils vary in thickness from approximately 3.5 feet in Boring B-2 to 5.0 feet in Boring B-1.

Very Hard Soils and Partially Weathered Rock: There is a transition zone of very hard soils or partially weathered rock below the upper residual soils, and above the relatively sound unweathered rock. The partially weathered rock zone is very dense or very hard in consistency. This material is also residual soil in that it has been formed by the in-place weathering of the underlying rock. Within this zone, weathering has not proceeded to an advanced stage, and represents a lesser degree of weathering. While the partially weathered rock or very hard soil material can be classified as very dense or very hard soil, it retains a rock-like character and banded appearance similar to the sounder unweathered rock which it overlies. The partially weathered rock zone varied in thickness from approximately 17 feet in Boring B-2 to 29 feet in Boring B-1.

Rock: Refusal to the drilling process was encountered in the two soil test borings at depths of 35 feet in Boring B-1 and 30 feet in Boring B-2. Core boring procedures were used in these borings to penetrate the rock and to determine its character and continuity. The rock is a gray granitic gneiss with weathered seams. Generally, the upper surface of the rock is soft to moderately hard. With an increase in depth, the rock hardness generally increased to hard or very hard. The upper rock zone is fractured and jointed, and contains numerous weathered seams. The upper rock zone has vertical joints as well as horizontal joints.

EMBANKMENT DESIGN AND CONSTRUCTION

We have received from Baldwin and Cranston Associates, a copy of the general plan and profile of the dam and spillway, the drainage system and low level outlet plan and profile, and the earthwork details. The dam has a maximum width at the base of approximately 825 feet, a crest length of approximately 900 feet, and is approximately 125 feet high. The lake level is at approximately Elevation 1635, while the top of the dam is at approximate Elevation 1645. The lowest portion of the embankment extends down to approximate Elevation 1517 at the low level outlet. The upstream face of the dam has a rip rap blanket layer between Elevations 1638 to 1633 for the protection of this slope, while the downstream face of the dam has been grassed. The surrounding abutments are heavily wooded and contain little other vegetation.

Material for the core of the dam is supposed to be a clayey silt or silty clay (ML or CL). The slope of the core material is approximately 1(H):2(V). Material for the shell of the dam is supposed to be a silty sand (SM). The slope of the upstream side of the dam is 3.5(H):1(V), while the downstream slope is 2.5(H):1(V) between berms. On the downstream face there are 6 berms approximately 10 feet in width. The berms slope slightly toward the dam. The materials for both the core and the shell were designed to be placed in thin lifts (9 inches maximum loose thickness) and compacted to 100% of its maximum dry density as determined by the Standard Proctor compaction test (ASTM Designation D 698, Method-A).

The internal drainage system for the dam was designed to be a 8 inch asbestos cement perforated pipe embedded in a 4 foot square sand filled trench. This 8 inch pipe traversed the valley bottom on the downstream face of the core, approximately 230 feet from the east-west centerline of the dam. Two 4 foot square sand drains (without a pipe) extend up the abutments at the abutment-core intersection. The low level outlet for the dam is a 30 inch reinforced concrete pipe. The low level outlet pipe travels along the north-south centerline of the dam for approximately 400 feet to the intersection with the east-west centerline of the dam. At that point, the outlet pipe curves and intersects the internal drainage system on the left side where two 8 inch asbestos cement pipes join the 30 inch outlet pipe and extend to the exit on the downstream face of the dam. The collector box for the outlet pipe and the internal drainage pipe consist of a 18 foot by 10 foot reinforced concrete box structure.

The following information was provided us by Mr. William E. Edens, of Baldwin and Cranston Associates. We were informed by Mr. Edens that the measured seepage loss was approximately 50 gallons per minute from the left abutment. We were also informed that the core materials used were actually CL and ML materials, while the shell material was an ML or SC. The materials were compacted to 100% of the Standard Proctor density tests. The abutment slopes on the upstream face are nearly vertical due to the stripping of the slopes for borrow material. We were informed that the core trench is located 5 to 10 feet into the abutment faces, and was extended through the alluvial material in the stream bed but does not extend to sound rock. The material upon which the core trench is resting is weathered rock that is very fractured and blocky. The low level outlet drain was relocated slightly west of the design plan location.

The following observations were made by the writers during site visits. Observation and examination of the dam and surrounding abutment slopes revealed a "spring" in the right abutment approximately 100 feet from the embankment face. Also, very small boils were observed along the right abutment near the toe of the dam. The embankment face was wet on the right side up to approximately the second berm. Either rain water or seepage water was observed ponding on the lower berms of the right side of the dam. Minor sloughing has occurred along the right abutment and dam interface above the third berm.

A major seepage of water was observed between the toe and the first berm at the intersection of the left abutment and the dam. A significant flow of water was also observed along the left abutment beyond the toe of the dam. The water in this area appeared to be flowing through the fractured rock of the abutment and exited from the holes created by tree roots. Observation and examination of the area indicated that no soil material was being transported by the seepage water.

Observations made during a more recent site visit, revealed a large wet area located in the middle of the dam on the third berm. A pine straw mat was placed on the wet area. Observation of the internal drainage system of the dam revealed a heavier flow of water on the recent site visit than on earlier site visits. Small boils were also found in the creek channel directly south of the internal drainage system and low level outlet.

## RECOMMENDATIONS AND CONCLUSIONS

The primary purpose of our investigation was to evaluate probable seepage paths and make recommendations for control of the observed seepage. The following recommendations are based on the above visual observations, structural information, the data obtained by our borings; and our past experience with evaluation of existing earth fill dams.

The present condition of the dam is such that the following alternatives for remedial action may be considered to control the seepage water. These alternatives are:

1. To attempt to seal the flow of water through the dam and through the abutment slopes by grouting or an upstream blanket on the dam and abutment faces.

2. To control the seepage by collecting it and channelling it into conduits so that loss of soil fines from the embankment and abutments by seepage erosion (piping) is prevented. This will require installation of filtered drains along the dam and abutment interfaces as well as external drains on the berms.

Grouting of the seamy and fragmented rock with a cement or chemical grout curtain is a very expensive operation, and would require extensive grouting in the abutment areas. Grouting is not effective in stopping seepage unless practically all of the voids through which the water is moving are completely filled with grout.

An alternative to grouting would be to place an upstream blanket of impervious soil on the embankment and abutment faces. This would require draining the lake to provide access to the upstream faces of the embankment and abutments. This does not appear to us to be a practical solution, in that a substantial quantity of relatively impervious clayey soil (CL) would be required, and is probably not available.

We also believe that grouting is not a practical alternative, in that it is relatively expensive, would require extensive drilling in order to penetrate the numerous joints, fractures, and seams of the rock, and may not be particularly effective. Therefore, we recommend that you give primary consideration to controlling the effects of seepage by collecting it into filtered conduits.

The primary concern in controlling the seepage is protecting the integrity of the dam by preventing piping of the embankment and abutment soils. The prevention of piping can be accomplished by the adequate installation of filter blankets and drains. The areas of greatest concern are the abutments below the dam, the intersection of the abutments with the dam, and the berms of the downstream face of the dam.

The problems associated with the seepage can be categorized into each area and the type of seepage problem (see Figure 5). The left abutment on the downstream side of the dam has major seepage occuring at the abutment-embankment interface, and through the residual soils of the abutment. The right abutment also has seepage through the abutment residual soils and the embankment fill as well as sloughing near the interface of embankment and abutment on the third berm, Elevation 1580. On the downstream face of the dam, minor sloughing has occurred approximately at the middle of the second berm, Elevation 1560. The three upper berms on the downstream face (Elevation 1580, 1600 and 1620 respectively), has ponding of surface runoff, while the two lower berms (Elevations 1540 and 1560) has seepage water ponding due to the high phreatic line intersecting at these berms.

Remedial action in the areas of concentrated seepage (see Figure 5) should consist of undercutting of the loose soils and the installation of filtered drains. The undercutting and removal must be done under the inspection of a qualified soils engineer to avoid excessive removal of the embankment soils. The depth of undercutting as determined by the soils engineer can be accomplished by manual labor or by a tractor-mounted backhoe. Installation of the filtered drain can be as follows (see Figure 6):

1. Shape the bottom of the trench and place a 12 inch filter layer of graded sand in the bottom of the trench to separate the shell material from the coarse aggregate drainage layer. The graded sand should meet the gradation requirements of ASTM Designation C-33 for Concrete Sand.

2. Next, place a 12 inch layer of coarse aggregate such as No. 89 stone on the filter sand. The coarse aggregate should conform to ASTM Designation D-448, Coarse Aggregate for Highway Construction.

3. A perforated pipe, minimum diameter 6 inches, should be installed in the thickened section of the coarse aggregate drainage layer.

4. The sand layer should surround the coarse aggregate layer and should be used to fill the rest of the trench.

5. An alternative to the two layer filtered drain would be to install a porous wall concrete pipe. This has the advantage of requiring only a single filter layer.

6. The pipe should be designed to discharge by gravity.

7. Installation of the drainage system should begin at the downstream end and proceed upstream.

The three upper berms, Elevations 1580, 1600 and 1620, are ponding surface runoff. Elimination of this ponding can be done by the installation of paved concrete gutters with sufficient slope to discharge the surface runoff to the abutments or to another paved ditch down the embankment-abutment interface. The berms should first be graded to remove any surface vegetation. A paved concrete gutter shaped to the berm slope can then be installed (see Figure 7). The concrete gutter should be sloped to discharge by gravity toward the abutments.

The two lower berms, Elevations 1540 and 1560, have water ponding on their surface due to surface runoff and a high phreatic surface due to seepage through the embankment. Collection of this water can be accomplished by using both an internal filtered drain and a paved concrete gutter. The internal filtered drain can be constructed similar to the filtered drains along the embankment-abutment interface (see Figure 6). The internal filtered drain trench should be approximately 4 feet in width and no more than 5 feet in depth. There should be a 12 inch layer of graded sand meeting ASTM Designation C-33 to separate the drainage material from the shell material. Next, a 12 inch layer of coarse aggregate such as No. 89 stone should be placed on top of the sand filter layer. A perforated pipe, minimum diameter of 6 inches, should be embedded within the coarse aggregate layer. Coarse aggregate is then placed around the pipe and the filter sand used to fill the rest of the trench (see Figure 8). The pipe should discharge by gravity toward the abutments. An alternative to this would be to use porous wall concrete pipe installed in the trench surrounded by a graded filter sand. A minimum layer of 12 inches of graded sand would be required in the bottom of the trench for embedment of the porous pipe. The paved concrete gutter should also be installed on the lower berms to remove surface runoff water.

In areas of localized sloughing, the areas can be resolved in a similar manner to that of the concentrated seepage. The areas corrected should encompass an additional 5 feet outside the perimeter of the sloughing area. The soft, loose soils should be undercut and removed under the inspection of a qualified soils engineer. The depth of undercutting should be determined by the soils engineer. Remedial construction for the areas along the embankment-abutment interface can be similar to the areas of concentrated seepage (see Figure 6). It is not necessary to install the perforated pipe, but provisions should be made so that it will be free draining to the next lower berm. In areas of sloughing on the face of the dam, a 12 inch sand filter meeting ASTM Designation C-33 should be placed between the shell material and the coarse aggregate drainage layer. After the sand layer has been placed, crushed stone, such as No. 89, should be used to fill the excavation. The surface of the crushed stone drainage layer should exit and connect to the top of the paved concrete gutter so that seepage water can be effectively removed (see Figure 9).

As stated previously, small boils were observed near the toe of the dam. The boils observed were along the right abutment at the toe of the dam and along the creek channel south of the low level outlet drain. The effect of the boils is to loosen and then transport the soil by flowing water. As the soil within the zone of boiling is carried away by the flowing water, a small open pit or channel can develop. This phenomenon is known as "piping".

Since additional grading for a roadway across the toe of the dam will be required, we recommend that a filtered drain similar to Figure 6 be constructed across the toe to intercept the seepage. After the roadway fill is placed, the toe of the roadway fill and the surrounding areas should be inspected periodically for evidence of boils or areas of concentrated seepage. If found, they may be repaired by covering them with a two-layer filter blanket of sand and stone, similar to the filters previously described.

It is possible that cracks in the low level outlet pipe can occur due to settlement of the dam. If cracks develop in the low level outlet, piping can occur from these cracks. Therefore, we recommend monitoring of the low level outlet due to the settlement of the dam, both for vertical movement of the pipe and visual observation to determine that there is no flow of water through the low level outlet.

Periodic inspections of the low level outlet as well as the surrounding areas should be made. We suggest that inspection of the dam, the filtered drains, and low level outlet be made by a qualified soils engineer on a semi-annual basis for a minimum of two years, and annually thereafter.

We appreciate the opportunity of having assisted you on this project and would be happy to answer any questions which you may have concerning this report.

Very truly yours,

LAW ENGINEERING TESTING COMPANY

John Kent Lominac

John Kent Lominac
Soils Engineer

David E. Pauls

David E. Pauls
Civil Engineer
Registered, Georgia 6450

JKL/DEP:mo

APPENDIX

# FIELD INVESTIGATIVE PROCEDURES

## SOIL TEST BORING

The borings were made by mechanically rotating a hollow-stem auger into the ground. After the groundwater surface had been penetrated, water was used to fill the auger stem to a height equivalent to, or higher than, the groundwater surface. The water was used to stabilize the bottom of the hole. At regular intervals, soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-tube sampler. The sampler was first seated 6 inches to penetrate any loose cuttings and then driven an additional foot with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the final foot was recorded and is designated the "standard penetration resistance".

Representative portions of the soil samples, thus obtained, were placed in glass jars and transported to our laboratory. In the laboratory, the samples were classified by a soils engineer. Test Boring Records are attached, graphically showing the soil descriptions and penetration resistances.

The soil samples obtained are presently in storage. We plan to dispose of these samples in 60 days unless instructed to do otherwise.

## ROCK CORING

Core drilling procedures were performed in accordance with ASTM Specification D 2113-62T. This drilling procedure consists of coring the material with a diamond-studded bit fastened to the end of a hollow double-tube core barrel. This device is rotated at high speeds and is capable of cutting the hardest rock. Core samples of the material are protected and retained in the swivel-mounted inner tube. Upon completion of the drill run, the core barrel is brought to the surface and the samples removed and placed in wooden boxes. The samples were returned to our laboratory where the rock was identified and recovery determined. The recovery is the ratio of the sample length obtained to the depth drilled, expressed as a percent. The value of the recovery is a qualitative index to the nature and soundness of the rock. Descriptions and recoveries and the bit size used are shown on the appropriate Test Boring Records.

## LIST OF FIGURES

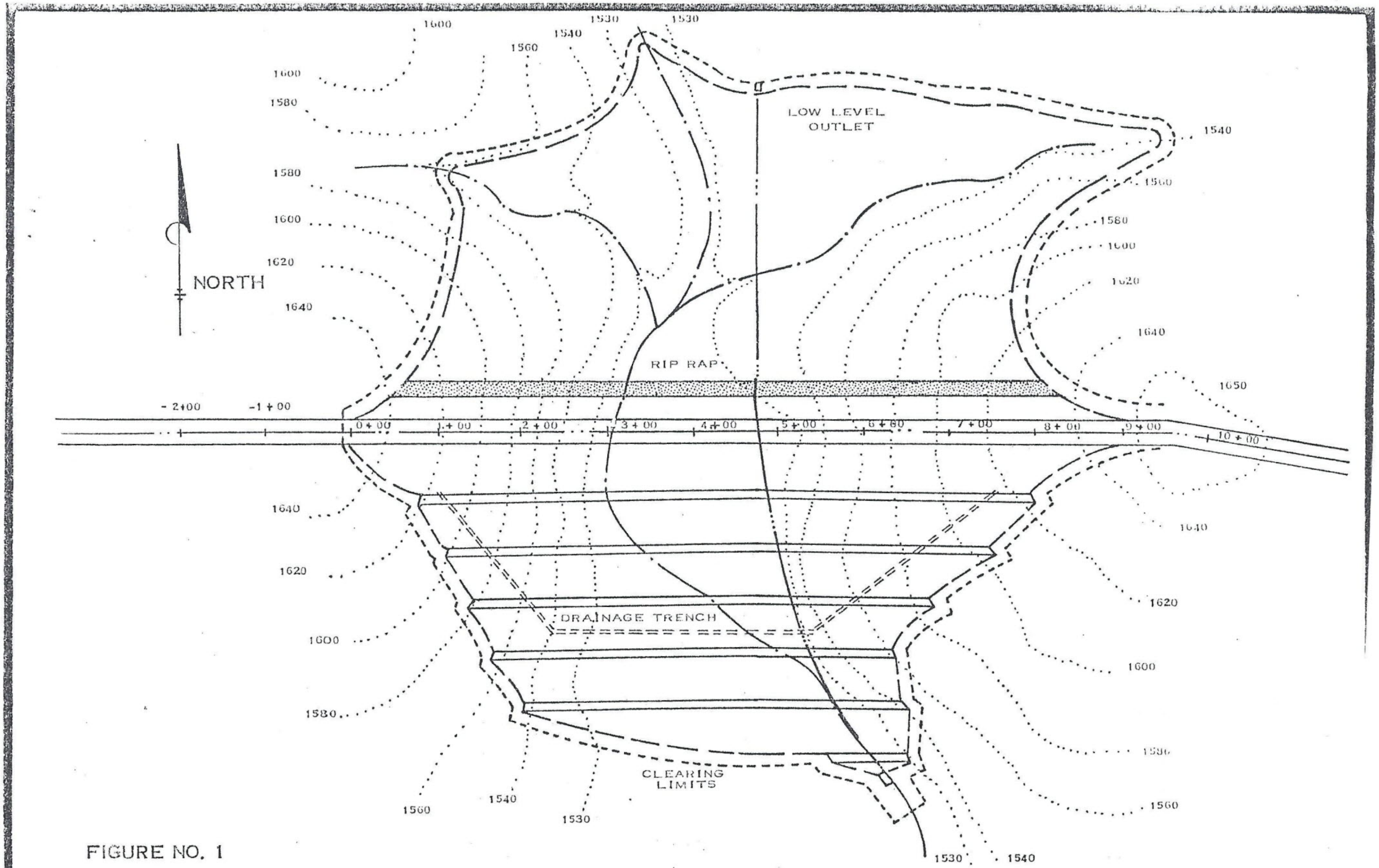



FIGURE NO. 1

| LEGEND | | PROJECT |
|---|---|---|
| CONTOUR INTERVAL 20 FEET | LAW ENGINEERING TESTING COMPANY<br>412 PLASTERS AVENUE, N.E.<br>ATLANTA, GEORGIA | PETIT COVE DAM<br>DAWSON COUNTY, GEORGIA<br>LETCO JOB NO. 7502 |
| SCALE NO SCALE | | |

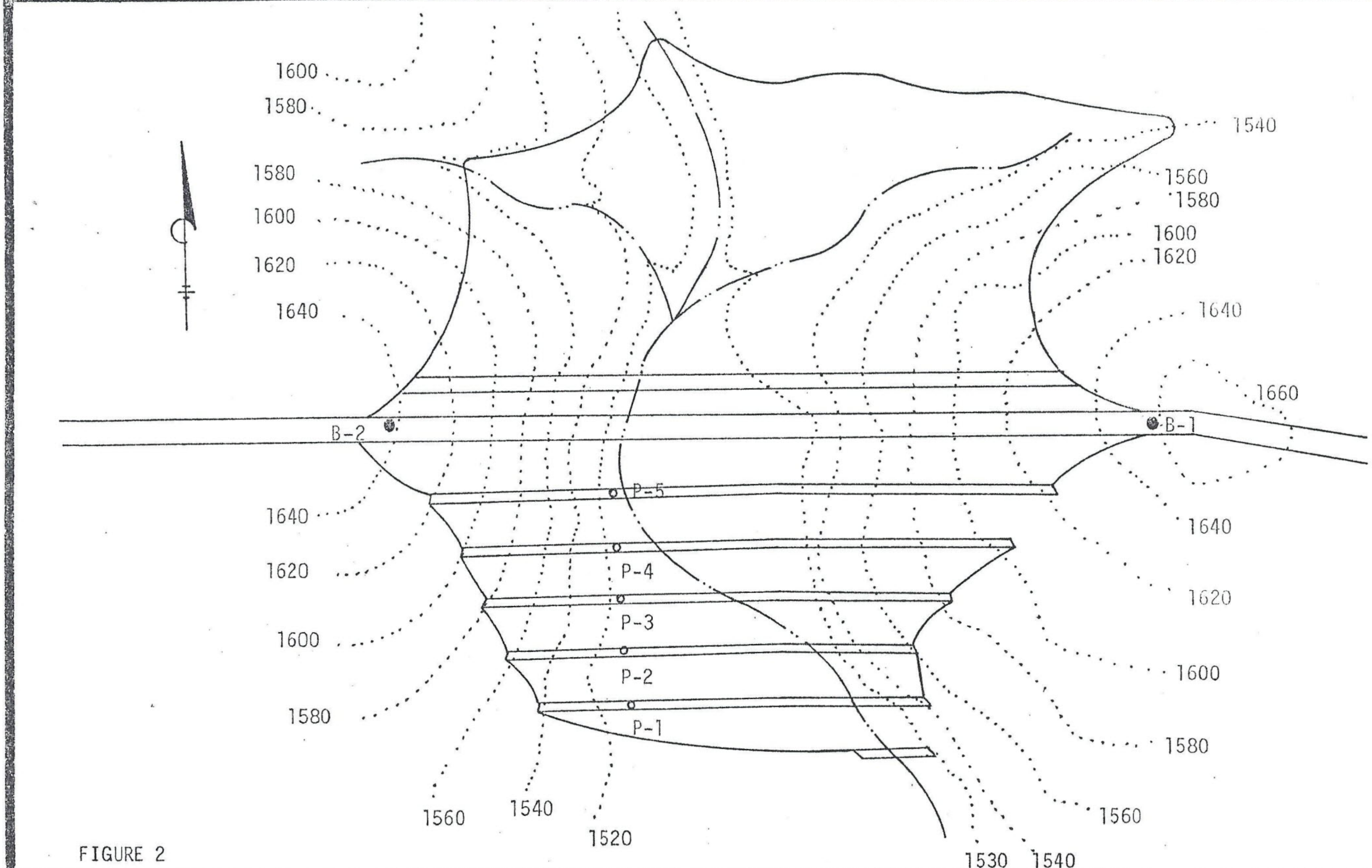
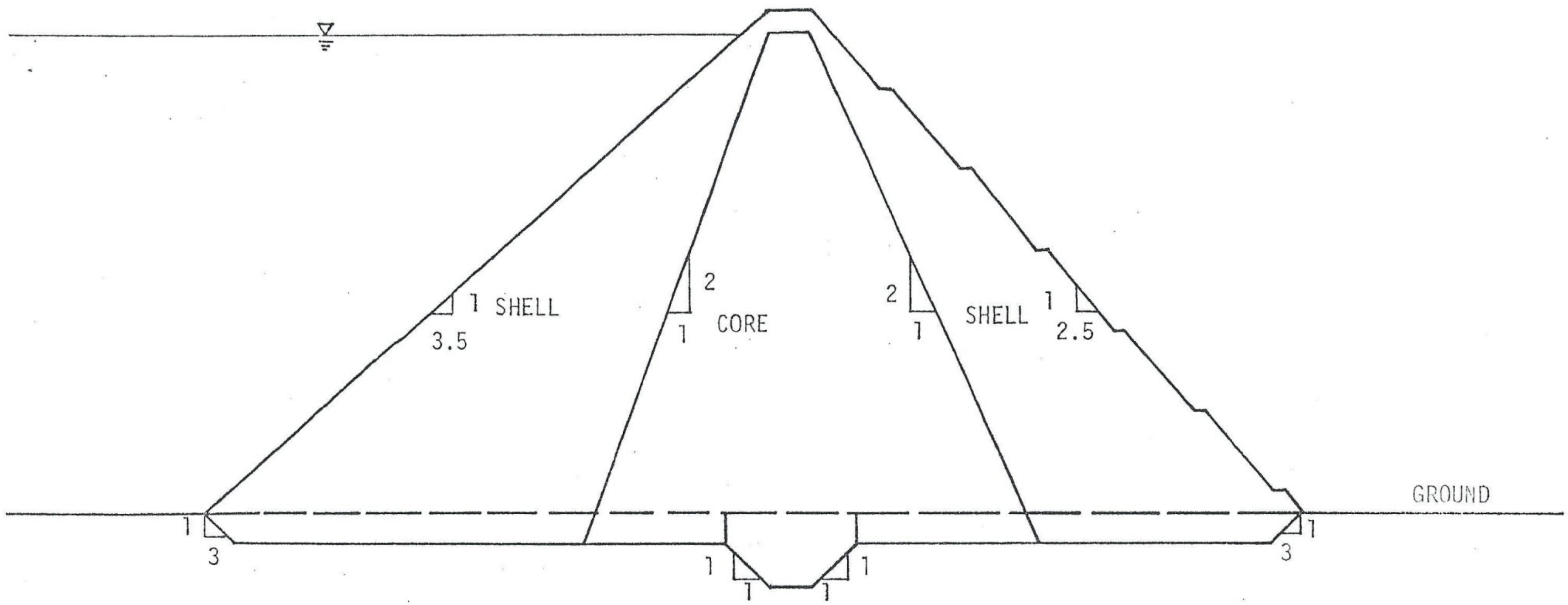



FIGURE 2

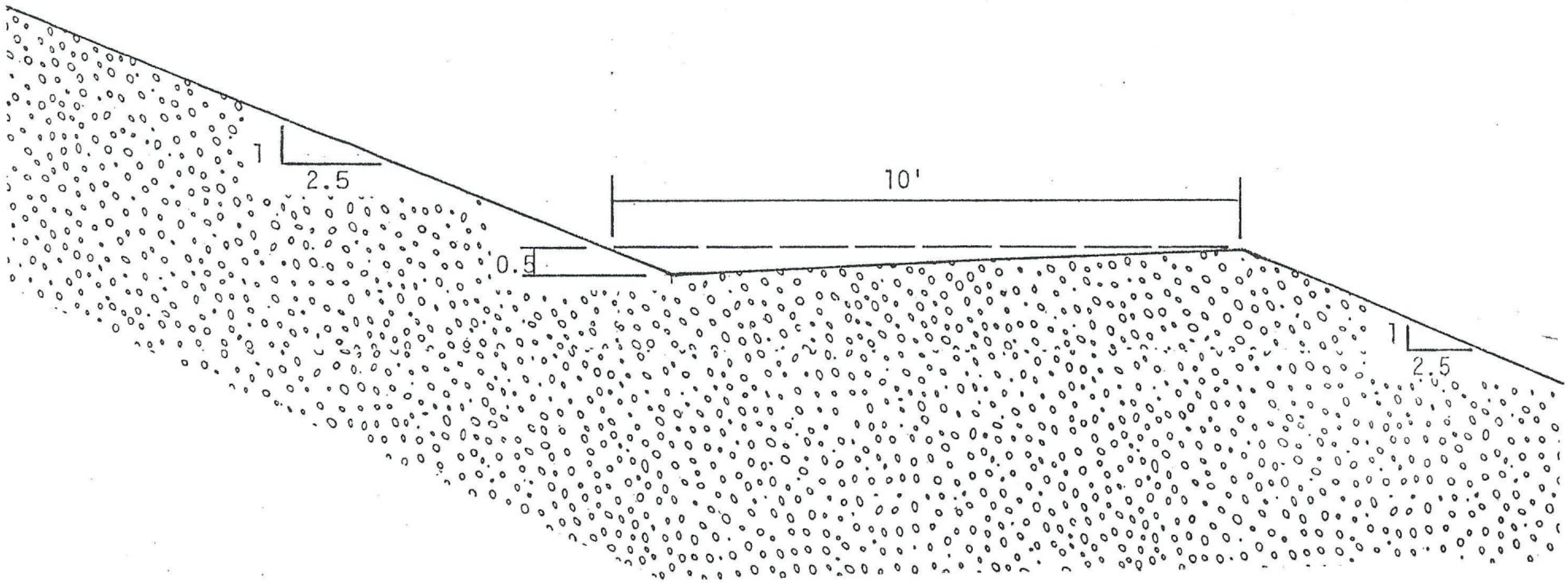
| LEGEND | LAW ENGINEERING TESTING COMPANY | PROJECT |
|---|---|---|
| ● SOIL TEST BORING<br>○ PIEZOMETER BORING<br>SCALE No Scale | 412 Plasters Avenue, N.E.<br>ATLANTA, GEORGIA | PETIT COVE DAM<br>DAWSON COUNTY, GEORGIA<br>LETCO JOB NO. 7502 |




TYPICAL CROSS-SECTION
AT STATION 4 + 50

FIGURE 3

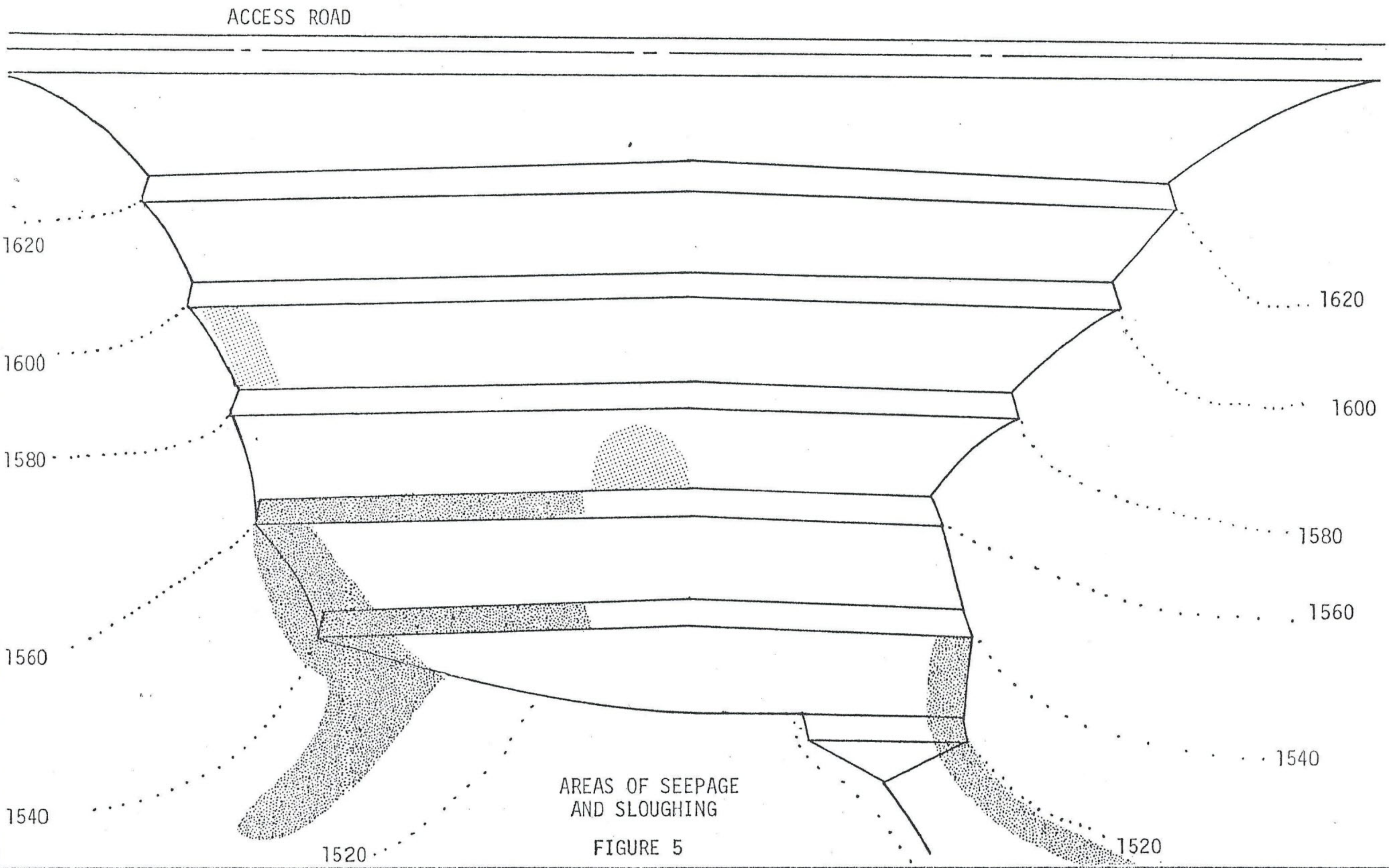
| LEGEND | LAW ENGINEERING TESTING COMPANY | PROJECT |
|---|---|---|
| SCALE No Scale | 412 Plasters Avenue, N.E.<br>ATLANTA, GEORGIA | PETIT COVE DAM<br>DAWSON COUNTY, GEORGIA<br>LETCO JOB NO. 7502 |




TYPICAL SECTION THROUGH
BERM

FIGURE 4

| LEGEND | LAW ENGINEERING TESTING COMPANY | PROJECT |
|---|---|---|
| SCALE No Scale | 412 PLASTERS AVENUE, N.E.<br>ATLANTA, GEORGIA | PETIT COVE DAM<br>DAWSON COUNTY, GEORGIA<br>LETCO JOB NO. 7502 |




AREAS OF SEEPAGE
AND SLOUGHING

FIGURE 5

LEGEND

AREAS OF SEEPAGE

AREAS OF SLOUGHING

SCALE

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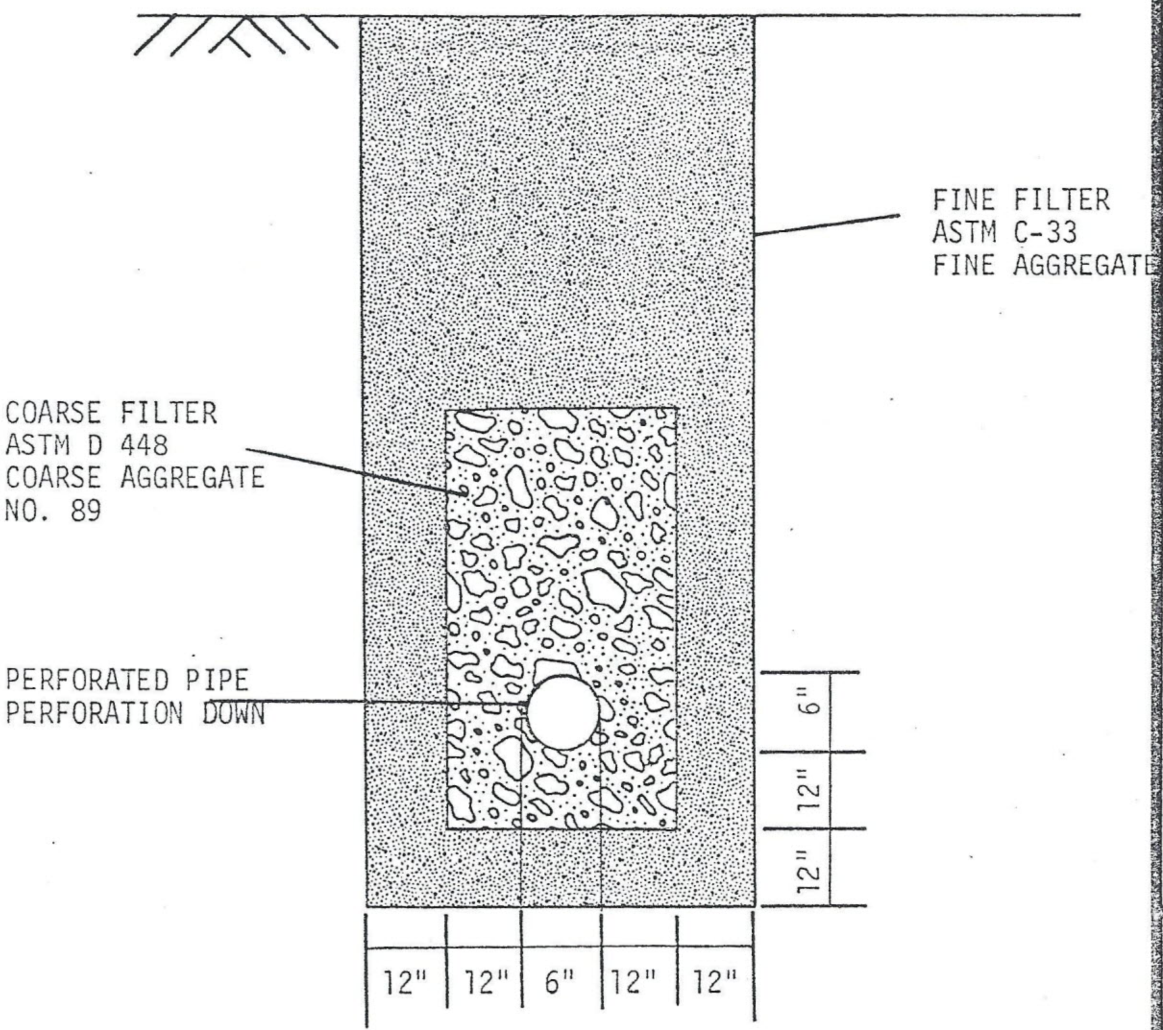
412 Plasters Avenue, N.E.

ATLANTA, GEORGIA

PROJECT

PETIT COVE DAM
DAWSON COUNTY, GEORGIA
LETCO JOB NO. 7502




INTERNAL ABUTMENT DRAIN

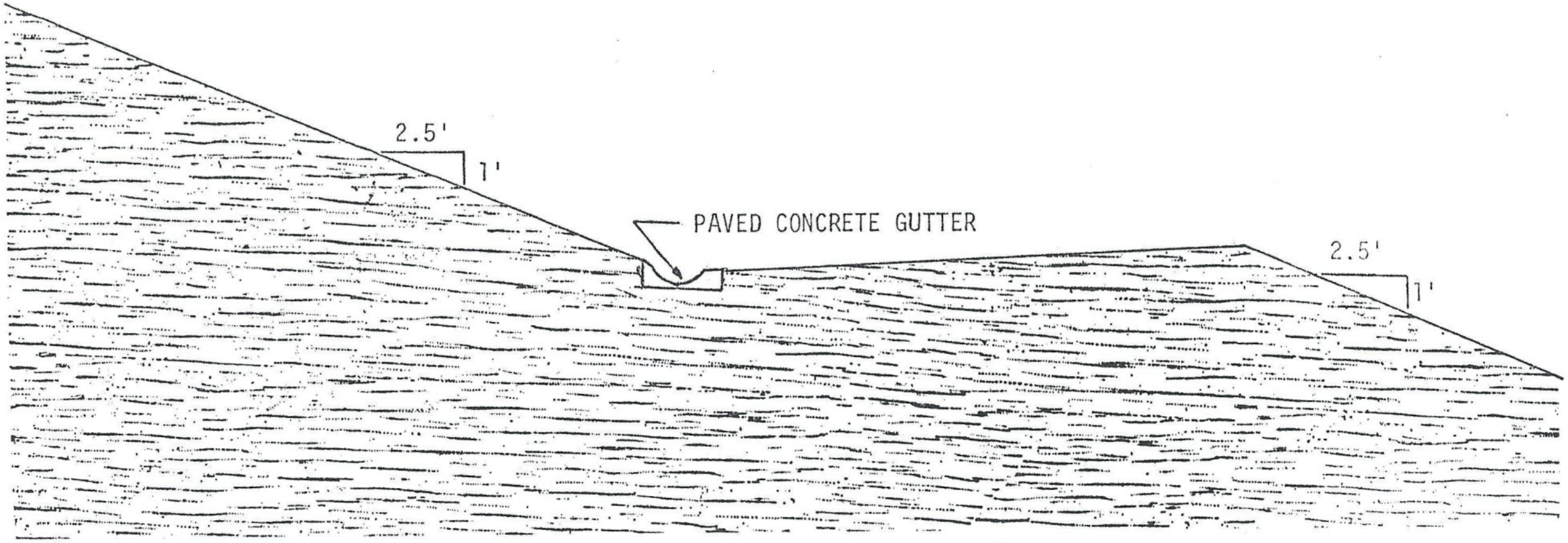
FIGURE 6

PROJECT
PETIT COVE DAM
DAWSON COUNTY, GEORGIA
LETCO JOB NO. 7502

LAW ENGINEERING TESTING COMPANY
412 Plasters Avenue, N.E.
ATLANTA, GEORGIA

LEGEND

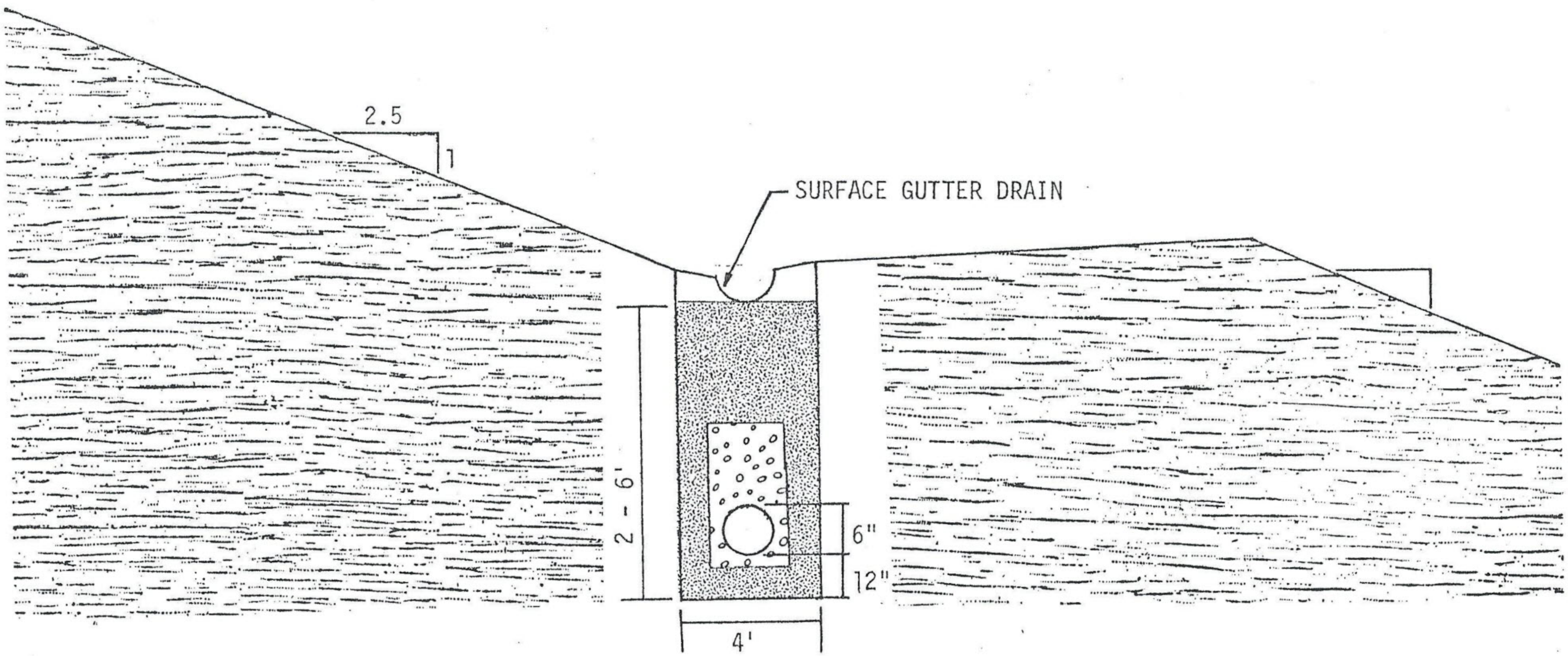
SCALE
No Scale




UPPER BERM SURFACE DRAIN

FIGURE 7

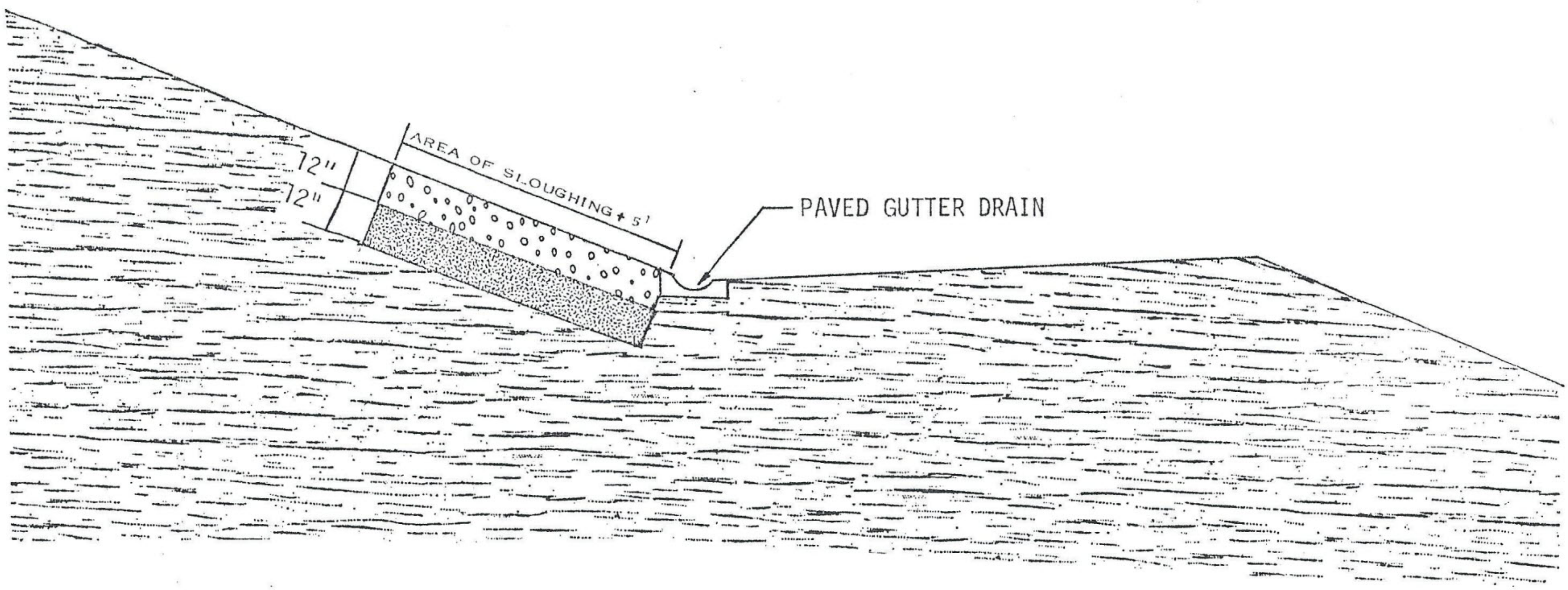
| LEGEND | | PROJECT |
|---|---|---|
| SCALE No Scale | **LAW ENGINEERING TESTING COMPANY**<br>412 Plasters Avenue, N.E.<br>ATLANTA, GEORGIA | PETIT COVE DAM<br>DAWSON COUNTY, GEORGIA<br>LETCO JOB NO. 7502 |




LOWER BERM SURFACE AND INTERNAL DRAINS

FIGURE 8

| LEGEND | | PROJECT |
|---|---|---|
| SCALE No Scale | **LAW ENGINEERING TESTING COMPANY**<br>412 Plasters Avenue, N.E.<br>ATLANTA, GEORGIA | PETIT COVE DAM<br>DAWSON COUNTY, GEORGIA<br>LETCO JOB NUMBER 7502 |




INTERNAL DRAIN FOR AREA OF SLOUGHING ON EMBANKMENT FACE

FIGURE 9

| LEGEND | LAW ENGINEERING TESTING COMPANY | PROJECT |
|---|---|---|
| SCALE No Scale | 412 Plasters Avenue, N.E.<br>ATLANTA, GEORGIA | PETIT COVE DAM<br>DAWSON COUNTY, GEORGIA<br>LETCO JOB NO. 7502 |

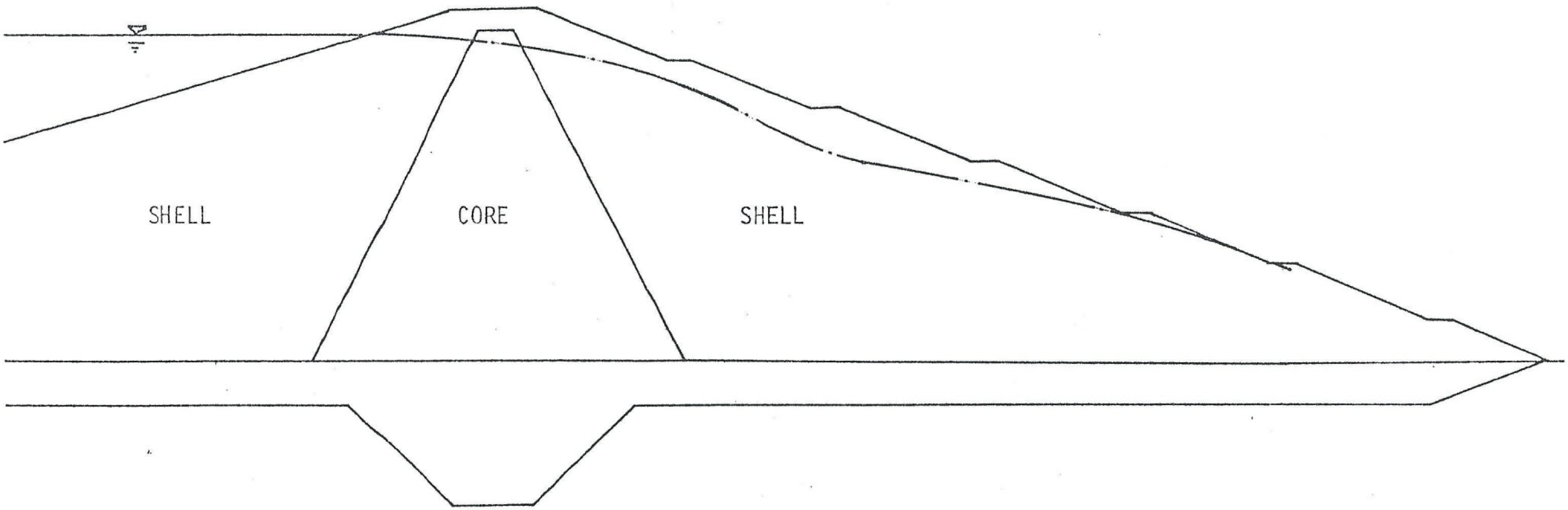



FIGURE 10

PHREATIC LINE

LEGEND

—·— Phreatic Line

SCALE No Scale

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ATLANTA, GEORGIA

PROJECT

PETIT COVE DAM
DAWSON COUNTY, GEORGIA
LETCO JOB NO. 7502

# KEY TO CLASSIFICATIONS AND SYMBOLS

## CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY

| | NO. OF BLOWS, N | RELATIVE DENSITY |
|---|---|---|
| SANDS | 0–4 | VERY LOOSE |
| | 4–10 | LOOSE |
| | 10–30 | FIRM |
| | 30–50 | DENSE |
| | OVER 50 | VERY DENSE |
| | | CONSISTENCY |
| SILTS AND CLAYS | 0–2 | VERY SOFT |
| | 2–4 | SOFT |
| | 4–8 | FIRM |
| | 8–15 | STIFF |
| | 15–30 | VERY STIFF |
| | 30–50 | HARD |
| | OVER 50 | VERY HARD |

## SYMBOLS

| | |
|---|---|
| ▮ | – Undisturbed sample (UD) recovered |
| ▯ | – Undisturbed sample (UD) not recovered |
| 100 / 2'' | – Number of blows (100) to drive the spoon a number of inches (2) |
| AX BX, NX | – Core barrel sizes which obtain cores 1–1/8, 1–5/8 and 2–1/8 inches in diameter respectively |
| 65 % | – Percentage (65) of rock core recovered |
| RQD | – Rock quality designation–% of core segments 4 or more inches long |
| | – Water table at least 24 hours after drilling |
| | – Water table one hour or less after drilling |
| ◀ | – Loss of drilling water |
| A | – Atterberg limits test performed |
| C | – Consolidation test performed |
| GS | – Grain size test performed |
| T | – Triaxial shear test performed |
| P | – Proctor compaction test performed |
| V | – Field vane shear test performed |
| 18 | – Percent of natural moisture content (18) |

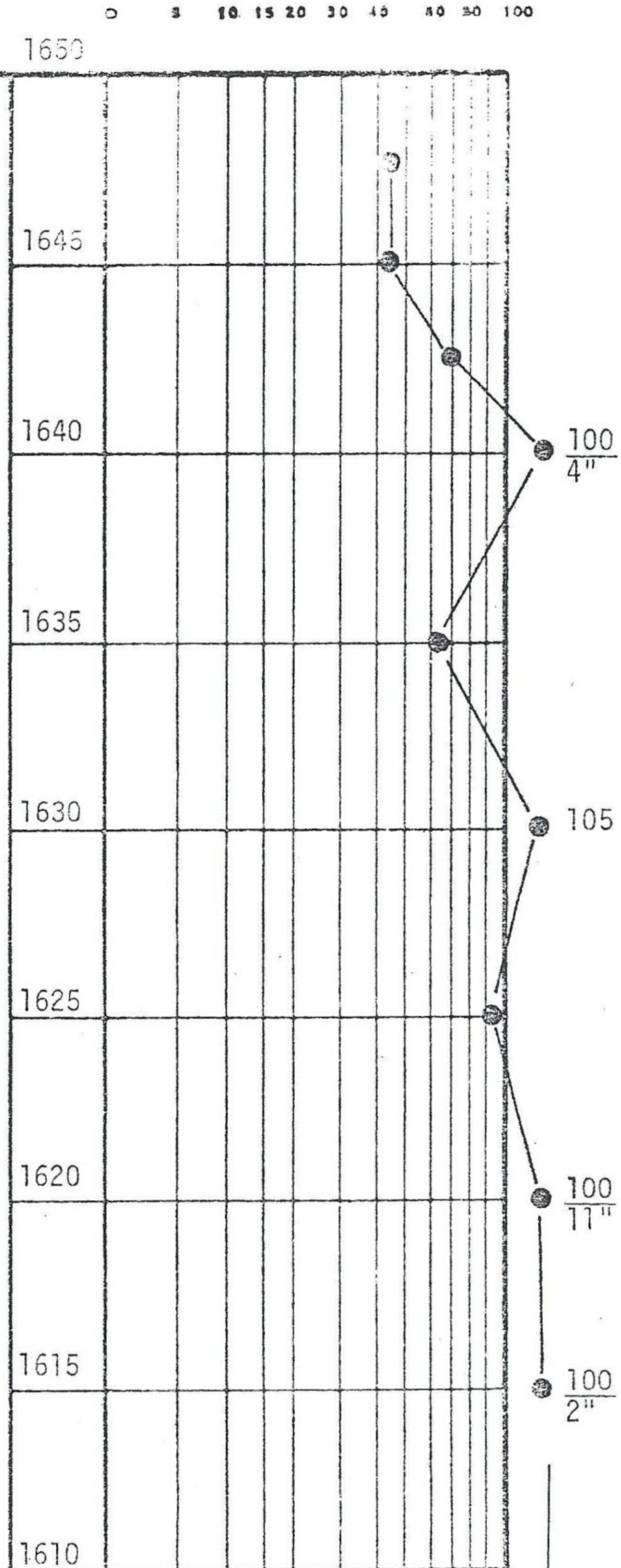
## DRILLING PROCEDURES

Soil sampling and penetration testing performed in accordance with ASTM D 1586–67.
The standard penetration resistance is the number of blows of a 140 pound hammer falling 30 inches to drive a 2 inch O.D., 1.4 inch I.D. split spoon sampler one foot.
Core drilling in accordance with ASTM designation D 2113–62T.
The undisturbed sampling procedure is described by ASTM specification D 1587–67.

# TEST BORING RECORD

| DEPTH FEET | DESCRIPTION |
|---|---|
| 0 | |
| 1.0 | DENSE GRAY AND BROWN SANDY ** |
| 3.0 | DENSE RED BROWN SLIGHTLY MICACEOUS FINE SANDY SILT |
| | DENSE GRAY BROWN SILTY FINE TO MEDIUM SAND |
| 6.0 | |
| | VERY DENSE GRAY AND WHITE FINE SILTY SAND - (PARTIALLY WEATHERED ROCK) * |
| 17.0 | |
| | VERY DENSE LIGHT BROWN WITH RED TO YELLOW BROWN MICACEOUS FINE SILTY SAND - (PARTIALLY WEATHERED ROCK) |
| 28.0 | |
| | VERY HARD DARK BROWN TO LIGHT TAN FINE SANDY SLIGHTLY MICACEOUS SILT - (PARTIALLY WEATHERED ROCK) |
| 35.5 | |
| | REFUSAL TO SOIL BORING OPERATION (SEE CORE BORING RECORD) |

ELEV. PENETRATION-BLOWS PER FOOT




REMARKS: * NO RECOVERY OF SAMPLE
** GRAVEL - (BASE COURSE FILL)

BORING NUMBER B-1

DATE DRILLED 12-19-73

JOB NUMBER 7502

LAW ENGINEERING TESTING COMPANY

| DEPTH FT. | DESCRIPTION | CORE % | BIT SIZE | ELEV. 1615 | REMARKS |
|---|---|---|---|---|---|
| 35.5 | HARD DARK GRAY GNEISS | | | | |
| 37.5 | SOFT TO MODERATELY HARD LIGHT BROWN AND GRAY WEATHERED GNEISS | 40 RQD =0 | NX | 1605 | CONTAINS NUMEROUS JOINTS AND SEAMS. |
| | | 68 RQD =0 | | 1595 | |
| | | 60 RQD =0 | | | |
| 62.5 | HARD GRAY GNEISS | | | 1585 | |
| 65.5 | NO RECOVERY OF SAMPLE | 0 RQD =0 | | 1575 | |
| 75.5 | MEDIUM HARD TO SOFT BROWN TO GRAY GNEISS | 55 RQD =0 | | 1565 | HAS WEATHERED SEAM AT 85.5' |
| 88.0 | VERY HARD DARK GRAY GNEISS | 97 RQD =6 | | 1555 | SAMPLE CONTAINS VERTICAL FRACTURES |
| 95.0 | CORING TERMINATED | | | | |

WATER TABLE

## CORE BORING RECORD

BORING NO. B-1

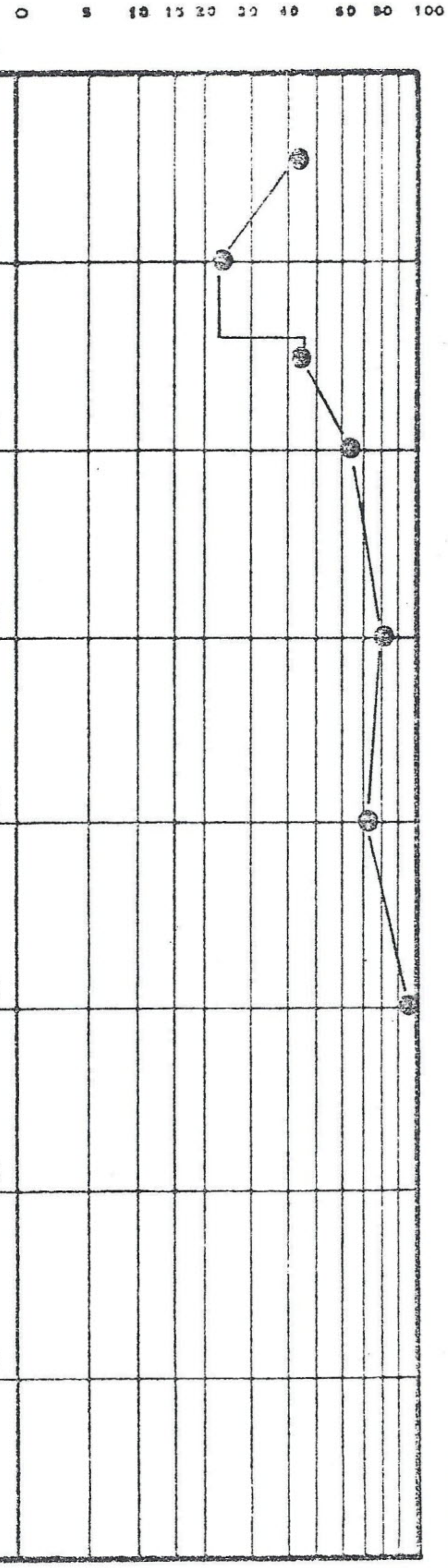
JOB NO. 7502

LAW ENGINEERING TESTING CO.

# TEST BORING RECORD

| DEPTH FEET | DESCRIPTION | ELEV. |
|---|---|---|
| 0 | | 1658 |
| 1.0 | DENSE GRAY AND BROWN SANDY * | |
| | HARD TO VERY STIFF BROWN FINE SANDY MICACEOUS SILT - (FILL) | 1653 |
| 5.0 | HARD BROWN FINE TO COARSE SANDY SLIGHTLY MICACEOUS SILT WITH ** | |
| 7.0 | HARD BROWN FINE SANDY MICACEOUS*** | |
| 8.5 | HARD RED, BROWN AND GRAY FINE SANDY MICACEOUS SILT | 1648 |
| 12.0 | HARD LIGHT BROWN AND GRAY FINE SANDY MICACEOUS SILT - (PARTIALLY WEATHERED ROCK) | 1643 |
| | | 1638 |
| | | 1633 |
| 29.0 | REFUSAL TO SOIL TEST BORING (SEE CORE BORING RECORD) | 1628 |
| | | 1623 |

PENETRATION-BLOWS PER FOOT




REMARKS:
* GRAVEL - (BASE COURSE FILL)
** FINE GRAVEL - (FILL)
*** SILT WITH ORGANIC MATTER

BORING NUMBER B-2
DATE DRILLED 1-2-74
JOB NUMBER 7502

LAW ENGINEERING TESTING COMPANY

| DEPTH FT. | DESCRIPTION | CORE % | BIT SIZE | ELEV. 1629 | REMARKS |
|---|---|---|---|---|---|
| 29.0 | HARD DARK GRAY GNEISS | | | | |
| 30.0 | MODERATELY HARD TO SOFT BROWN AND GRAY WEATHERED GNEISS | 83 RQD =0 | NX | 1619 | CONTAINS NUMEROUS M SEAMS |
| 38.0 | HARD TO VERY HARD DARK GRAY GNEISS | 84 RQD =3 | | 1609 | |
| | | 40 RQD =60 | | 1599 | |
| 59.0 | CORING TERMINATED | | | | |

WATER TABLE

CORE BORING RECORD

BORING NO. B-2

JOB NO. 7502