W-2632-01

LANDSLIDE INVESTIGATION

FAIRVIEW RESERVOIR NUMBER ONE

ABERDEEN, WASHINGTON

TO

JOHN W. CUNNINGHAM AND ASSOCIATES
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FOR

ENGINEERING DEPARTMENT
CITY OF ABERDEEN
CITY HALL
ABERDEEN, WASHINGTON

MAY 9, 1974

BY

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LANDSLIDE INVESTIGATION
FAIRVIEW RESERVOIR NO. 1
ABERDEEN, WASHINGTON

1.0 SUMMARY AND CONCLUSIONS

The water supply for the City of Aberdeen consists of two reservoirs, Fairview No. 1 and Fairview No. 2, located on a ridge, approximately at elevation 250 feet about one mile north of the business district as shown on the Vicinity Map, Figure 1.

A landslide has developed in a ravine downslope from the north side of Reservoir No. 1 which is an embankment constructed across the head of the ravine. This landslide has advanced in a progressive headward direction to the extent that it has caused a partial failure of the embankment and could, if left unchecked, cause a total failure of part of the embankment and rupture of the reservoir lining. The present landslide has been active in the lower section of the ravine for more than a year and extended upslope to the toe of the embankment in late 1973. Increased slide activity took place in January and February of 1974, causing further damage to the embankment. The presently active slide area is indicated on the Geologic Map, Figure 2.

Records of the City Engineering Department indicate that a similar landslide occurred about 1920. This landslide caused a nearly complete failure of a section of the embankment and destroyed a section of the concrete lining. The embankment and lining were repaired after construction of a timber sheet piling bulkhead and drainage facilities. The present landslide is,

therefore, considered to be a reactivation of the old slide rather than a new slide. The recent slide appears to have disturbed the drainage facilities installed during earlier reconstruction.

hardened

The ridge is underlain by moderately well indurated marine sedimentary rocks, including shale, siltstone, sandstone and conglomerate. In the vicinity of the landslide, these rocks dip about 24 degrees to the east and strike in a northerly direction, essentially parallel to the axis of the ravine and present landslide. A typical geologic section D-D' is shown on Figure 2, Geologic Map and Cross Section. The location of the ravine is controlled by the presence of a siltstone stratum which is more easily eroded than other more granular phases of the rock. These rocks are deeply weathered and everywhere covered by a mantle of residual soil overburden.

It is reported that the ravine was used as a spoil disposal area during original construction of the reservoir. The present landslide appears to be involved with old spoil material, soil and highly weathered rock, together with a section of the embankment forming the north side of Reservoir No. 1.

The active landslide now has a factor of safety slightly less than 1.0. This factor is somewhat, but not strongly, influenced by the presence of groundwater in the slide mass. It is our opinion that a factor of safety of 1.4 should be achieved to stabilize the upper area of the landslide and embankment failure. The results of our analysis indicate that a factor of this order of magnitude could be achieved by construction of a buttress fill for which design recommendations

are included in this report. The outlet for previously established drainage facilities in the embankment should be tied into the drainage system recommended for the buttress fill.

It is also possible that an adequate safety factor could be achieved by chemical stabilization although further testing and analysis would be required to thoroughly evaluate this scheme. It is our opinion that it is only practical to consider stabilizing the upper part of the present slide area.

The topographic knoll to the north of adjacent Reservoir No. 2 may provide a suitable source of borrow for a buttress fill.

2.0 INTRODUCTION

This report contains the results of a geotechnical investigation that has been completed in connection with the study of a landslide at Fairview Reservoir No. 1, a water supply reservoir for the City of Aberdeen, Washington. The purpose of this work was to define the nature and limits of the existing slide, evaluate its potential with respect to causing further damage to the reservoir, and develop sufficient information to support the design of a scheme to stabilize the slide and protect the reservoir from further damage.

This investigation has been completed under the direction of John W. Cunningham and Associates, consulting engineers at Portland, Oregon for the City of Aberdeen, in accordance with the terms and conditions contained in our proposal letter dated January 28, 1974.

3.0 FIELD INVESTIGATION AND RESULTS

The field investigation associated with this study was started on February 5, 1974 and included a geological reconnaissance of the landslide and adjacent area, a geophysical survey, the establishment of several stake lines to determine the lateral extent of the effected area, the completion of several test borings and the installation of inclinometer casings at selected locations to define the vertical dimension of the slide mass and rate of movement. The location of the test work is shown on the Geologic Map, Figure 2, and Detailed Site Plan, Figure 3.

3.1 Geologic Investigation

A reconnaissance was made in the vicinity of the reservoir to determine geologic factors that may be related to the present landslide.

The ridge on which the reservoir is located is composed of Miocene sedimentary rocks of the Montesano Formation. These rocks consist of interbedded shale, siltstone, sandstone and conglomerate and are generally deeply weathered. The general dip of the Montesano Formation in the vicinity of the reservoir is 0 to 24 degrees to the east and southeast as shown on the Geologic Map and Cross Section, Figure 2. A geologic section across the head of the present landslide is also shown on Figure 2. Evidence of faulting in the bedrock was found at several locations, however, the faults appear to be old and well healed and not to be related to the present landslide.

The bedrock is generally deeply weathered forming residual soil overburden of varying thickness. On oversteepened slopes, the development of landslides in residual soil overburden

and weathered rock is a typical element of the normal process of erosion in this type of rock. Nearby areas of ancient landsliding have been identified and are indicated on the Geologic Map, Figure 2.

The ravine in which the present landslide is located was used to deposit spoil from the excavation associated with construction of the reservoir before 1920. This fill material together with underlying residual soil and weathered rock are associated with the present slide that is up to 150 feet in width and extends for a distance of more than 500 feet downslope from the reservoir into the ancient slide area beyond the mouth of the ravine. A similar landslide is reported to have occurred about 1920. The earlier slide breached the embankment and reservoir lining.

A small spring emanates from the base of a slide scarp, approximately 150 feet downslope from the top of the embankment, forming a stream that flows down the ravine into the lower area of the present slide. The source of the spring is probably from drainage facilities that were installed in connection with reconstruction of the embankment in 1928 and have since become disrupted by the recent sliding. The stream is reported by Mr. Robert Salmon, Water Superintendent, to flow throughout the year and not to vary noticeably in volume of flow when the reservoir is emptied indicating that it is probably a natural spring.

3.2 Geophysical Survey

A seismic refraction survey was completed in the upper slide area for the purpose of defining the approximate depth and configuration of the unweathered rock surface beneath the landslide and on the knoll to the north of Reservoir No. 2. Six individual

seismic lines were completed for a total length of 1,375 feet of seismic traverse. This work was accomplished with a portable 12-channel seismograph using small explosive charges as a source of seismic energy.

The location of this work is shown on the Geologic Map, Figure 2, and Detailed Site Plan, Figure 3. The results are shown on the Seismic Velocity Profiles, Figure 4.

A comparison of the seismic velocity profiles to the logs of test borings indicate the following probable correlation between seismic velocity and material type:

Velocity ft./sec.	Material Type
1000	Unconsolidated residual soil overburden, spoil and embankment fill, undifferentiated
2800 to 3500	Highly weathered bedrock
5000 to 7500	Moderately weathered to unweathered bedrock

The seismic velocities do not appear to be clearly influenced by disturbance of material within the slide mass although there is good correlation at boring B-1 between the base of sliding indicated by inclimoneter measurements and the base of the 2800 to 3500 ft./sec. boundary. The profiles appear to be related closely to the degree of weathering and the thickness of soil overburden.

Seismic results obtained on the knoll to the north of Reservoir No. 2 indicate the rock to be weathered to an elevation below that of the existing reservoir. Suitability of this material to provide construction material for a buttress fill has not been

established but could be readily determined by drilling or trenching. It is our opinion that the 3200 ft./sec. weathered rock can probably be excavated without blasting. Removal of the knoll would also eliminate the possibility of a future slide of surficial material into Reservoir No. 2.

3.3 Test Borings

Four exploratory borings, designated B-1, B-2, B-3 and B-4, were drilled to determine the general subsurface conditions. Boring locations are shown on Figures 2 and 3. Depths ranged from 23 to 52 feet. A total of 158 lineal feet of drilling was accomplished.

Drilling and sampling was subcontracted to Pacific Testing Laboratories of Seattle, Washington. Their work was continuously observed by Mr. John Musser, a geologist with our firm. The boring work was accomplished using a truck-mounted Mobil Model B-61 drill rig and 3-3/8-inch I.D. hollow-stem continuous flight auger.

All borings were located by taping from existing survey stakes at the site. Ground surface elevations at the boring locations were established by a survey crew from the City of Aberdeen Engineering Department.

The borings were drilled by adding 5-foot sections of auger to the initial cutter bit. When the auger was being advanced, the stem hole in the cutter bit was plugged with the sampler attached to an inner string of rods. When the selected sampling depth was reached, the inner rod and sampler was removed, a new sampler attached to the rods, inserted into the stem hole and the sample obtained by driving or hydraulically pushing on the rods.

For drilling the final 15 feet of borings B-1 and B-2, boring equipment was converted from augering to the rotary method to provide a tight hole to secure the bottom of the inclinometer casing. A 3-inch diameter tri-cone bit with water as a drilling fluid was utilized.

The Standard Penetration Test (ASTM D-1586) was conducted in conjunction with obtaining split spoon drive samples. This test consists of driving a 2-inch O.D. split spoon sampler 18 inches, or to refusal (100 blows), by a 140-pound hammer freely falling a distance of 30 inches. The number of blows required to penetrate each 6-inch increment, or smaller under refusal conditions, was recorded by our geologist. The number of blows for the last two 6-inch increments were totaled and recorded as the Standard Penetration Resistance. This number, known as the N-value, provides an approximate means for evaluating the consistency of plastic soils and the relative density of granular soils. A sampling interval of 5 feet was maintained throughout most of the boring program although continuous samples were taken at selected locations.

All samples were visually classified and logged in the field by our geologist and then sealed in airtight jars.

Undisturbed samples were taken in addition to the drive samples at the direction of our geologist. Generally these were taken where more compressible soils were encountered and where more detailed information was required. These samples were obtained by using the hydraulic pulldown on the drill rig to push 3-inch O.D. thin-walled steel tubes ahead of the auger bit a distance of about 2 feet. The tube was extracted, slough material removed from the ends and the exposed material classified and logged. The ends were then sealed with molten wax and identified for shipment to our laboratory.

The results of these test borings are shown on the Boring Logs, Figures 5 through 8. Slope indicator casing was installed in borings B-1 and B-2 for inclinometer and water level measurements. Three-quarter-inch PVC pipe was installed in borings B-3 and B-4 to preserve them for water level measurements.

3.4 Stake Lines

On February 1, at our request, a survey crew from the City Engineers Office set four lines of stakes across the slide area for the purpose of determining the areal extent and rate of surface movement. These stake lines were designated Lines "E", "F", "G", and "H" and were also used as reference points for the seismic survey and drilling program. Subsequent measurements of horizontal and vertical position were made on February 6, 12, and 25, March 13 and April 22. The locations of the stake lines are shown on the Detailed Site Plan, Figure 3.

The following table shows the relationship of surface movement based upon these measurements:

Line	Maximum Vertical Movement	Maximum Horizontal Movement	Ratio Vertical/Horizontal
E	-1.03'	.38'	2.8
F	-0.45'	.57'	0.79
G	-0.13'	.80'	0.19
H	+ .08'	.78'	0.11

These measurements have served to clearly define the limits of the slide area and have been helpful, together with inclinometer data, in evaluating the slide mechanism. These data indicate that the slide has moved at an average rate of about 0.3 ft./month through the period from early February to late April.

3.5 Inclinometer Installations

After completion of borings B-1 and B-2, inclinometer (Slope Indicator) casing was installed through the hollow-stem auger. Concrete sand was poured from the surface to backfill around the inclinometer casing as the auger was withdrawn.

The inclinometer casings consist of 10-foot sections of special, grooved, 2.75-inch O.D. x 2.32-inch I.D. ABS plastic pipe with special grooves placed at 90 degree angles to each other. The sections were installed flush jointed with 6-inch couplings. This casing is especially made for the operation of the Digitilt Inclinometer instrument, designed and manufactured by Slope Indicator Company in Seattle.

The initial readings of the inclinometers were obtained on February 19, 1974, using a biaxial Digitilt inclinometer model No. 50301. Subsequent readings were taken on February 19, 20, 21 and 27, March 8 and April 29.

The function of the Digitilt instrument is to detect the change in inclination of the casing from its original near-vertical position. By taking readings at regular depth intervals (every 2 feet) inside the casing, a change in profile of the casing may be constructed. Repeating such measurements periodically provides data on the locations, magnitude, direction, and rate of movement of the casing.

The biaxial Digitilt inclinometer records tilt in two coordinate directions simultaneously. For vertical installations the resolution is $\pm$ 0.005 foot per 100 feet, repeatability $\pm$ 0.010 foot per 100 feet, and total system accuracy is $\pm$ 0.025 foot per 100 feet.

At boring B-1 the inclinometer data indicates a zone of deformation at a depth of 23 to 26 feet. The average rate of movement is approximately 2.5 inches per month between the period February 19 through April 29. A summary of the inclinometer data is shown on B-1, Lateral Deformation - Slide Plane, Figure 9. No deformation is indicated in boring B-2 over the same period of time.

3.6 Chemical Stabilization Tests

Two soil samples were taken of slide material and weathered rock for the purpose of making preliminary tests to evaluate the susceptibility of these materials to chemical stabilization. The samples were sent to Ion Tech, Incorporated at Daly City, California for analysis. The results of these tests are preliminary, however, they do indicate that the soil and rock chemistry is probably favorable for chemical stabilization. A copy of their letter dated February 15 is appended to this report.

It is our opinion that chemical treatment could be a reasonable approach to stabilizing this landslide providing conditions were found by further examination and testing to be suitable. If it was found to be effective, this scheme would probably be less expensive and could be accomplished more quickly than stabilization by construction of a buttress fill. However, this is a relatively new method of landslide treatment with which we do not have direct experience. For this reason, we are reluctant to recommend it as a viable solution to this problem.

4.0 ENGINEERING STUDIES

4.1 Stabilization Analysis

Based on our study and interpretation of subsurface conditions, we conclude that landsliding has occurred in an uphill progressive manner on a relatively deep surface corresponding to the overlying contact between the weathered and decomposed bedrock and overlying less weathered sedimentary rock. In our opinion, sliding has occurred on the approximate surface shown on Figure 10. Using this slide surface, the stabilization analyses has been made assuming the following boundary conditions:

1) Sliding has occurred near the base of the clayey silt soil.

2) Groundwater levels are below the slide surface, similar to the conditions shown on boring logs B-1 and B-4. Groundwater may have been higher, however, when using the same level in determining both the existing resisting force and remedial design resisting force, we found no great effect on the resulting factor of safety.

3) The slide mass was idealized as a series of "blocks" having straight lines. Each block weight was resolved along the base of the slide zone.

4) The existing resisting force is slightly less than the calculated driving force (factor of safety = 0.99). Cohesion values of the soil at the slide plane were established as ranging between 0.19 and 0.29 tsf (tons per square foot), as determined in the laboratory. Values of

internal friction determined from the analysis to achieve the 0.99 factor of safety ranged from 6.6 (cohesion of 0.19 tsf) to 4.2 (cohesion of 0.29 tsf) degrees.

5) A minimum factor of safety against sliding of 1.4 was required for long-time stability.

The present slide can be expected to enlarge and continue moving unless measures are taken to control the activity. To stabilize the slide, we recommend construction of a high strength granular buttress at the location shown on Figure 3. The size and configuration of the buttress is a function of the desired factor of safety against sliding, together with the buttress soil strength, unit weight, upland topography and location of the slide plane. We believe that the construction of the buttress shown on Figure 10 will increase the factor of safety of the slope against sliding to at least 1.4. In our opinion, this is the minimum factor acceptable for stabilization.

4.2 Buttress Design Recommendations

We recommend that the high strength granular buttress be constructed to a minimum height of 20 feet above the slide surface and extend at least 2 feet below the slide plane, as determined in the field at the time of construction. The area between the buttress and reservoir embankment should be refilled and graded to a slope of 1 vertical on 2.5 horizontal, or flatter, similar to that shown on Figure 10. The buttress should extend full height across the present limits of the slide area as shown in the longitudinal section C-C, Figure 3. The surface of the buttress should not extend below the top boundary shown on Figure 10. However, the base of the buttress is approximate and some variation in depth should be expected during construction.

Figure 10 shows the details of the 20-foot high buttress. In our opinion, a 3-foot filter zone adjacent to the on-site clayey silt soil is required. The filter zone should consist of free-draining granular materials such as that conforming to the requirements of "Gravel Backfill for Drains" as specified in Section 9-03.12(4) of the State of Washington, Standard Specifications for Road and Bridge Construction, 1969. The remainder of the buttress fill could consist of pit-run sand and gravel or granular soil from the proposed adjacent borrow area. The buttress fill should be placed in maximum 12-inch loose lifts and compacted to at least 90 percent of the Modified Proctor maximum density as determined by the ASTM D-1557-70 laboratory test procedure. Samples of proposed borrow soils should be obtained and laboratory tests conducted to determine the soil strength and compaction characteristics. The results of these tests should be reviewed and the above compaction recommendation modified, if necessary.

We recommend that the buttress be keyed into the sloping bedrock surface as shown on Figure 10. We consider the three perforated drains, on approximate 20-foot centers, as shown on Figure 10, to be necessary to control groundwater and maintain strength within the buttress. The free-draining granular soil surrounding each drain is necessary to prevent infiltration of backfill into the pipe. The perforated drains should be connected to a tight drain at the low point of the buttress to carry collected water off the slope.

To minimize the danger of triggering additional sliding, excavation and backfilling for the buttress should be accomplished in trenches about 15 to 20 feet wide. Further, construction must be, in our opinion, accomplished during the drier summer and early fall months. One possible construction scheme would be, we believe, to proceed from the center area (deepest area) of the buttress, and proceed uphill. The sequence of construction for this scheme

is illustrated on Figure 11. Each trench excavation should be cut in near-horizontal steps into the sloping bedrock surface parallel to the buttress. This would result in the base of the buttress being keyed into the sloping bedrock surface both parallel and perpendicular to the axis of the buttress.

We do not intend that blasting of the bedrock be required, although ripping may be necessary. If the rock cannot be excavated by conventional ripping methods, it is our opinion that the competency of the rock would not necessitate keying to the extent recommended above. This decision should be made in the field during excavation by an experienced soils engineer or technician. Excavation slopes in the overburden soils should be made the responsibility of the contractor to provide safe working conditons.

Prior to buttress construction, we recommend that removal of overburden soils required in regrading the slope above the buttress area be accomplished. Following buttress construction, regrading should be completed and the slope groomed and seeded. Further, an asphalt gutter should be constructed at the intersection of the groomed side slopes, terminating at the low point of the buttress into a catch basin and tight drain, to facilitate surface drainage.

4.3 Construction Observations and Inspection

In our opinion, it is essential that construction of the buttress be monitored by a qualified individual to see that work is accomplished in accordance with the preceding recommendations. Further, there are several aspects of construction that must be decided in the field; determination of the slide surface is one such aspect.

Samples of proposed borrow soils should be tested to evaluate the strength and compaction characteristics upon which design parameters can be based. We recommend that construction inspection include close observation of buttress foundation preparation. Backfill placement and compaction requires continuous inspection to establish and maintain satisfactory procedures.

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February 15, 1974

Mr. Sigmund D. Schwarz
Shannon & Wilson, Inc.
1105 North 38th Street
Seattle, Washington 98103

Re: City of Aberdeen Reservoir Landslide

Dear Mr. Schwarz:

Testing has been completed on the "preliminary" samples you submitted to us for testing. Based on the results of the testing, it appears that the chemical environment of the landslide is compatible with the Ion Tech Method and, therefore, warrants further consideration.

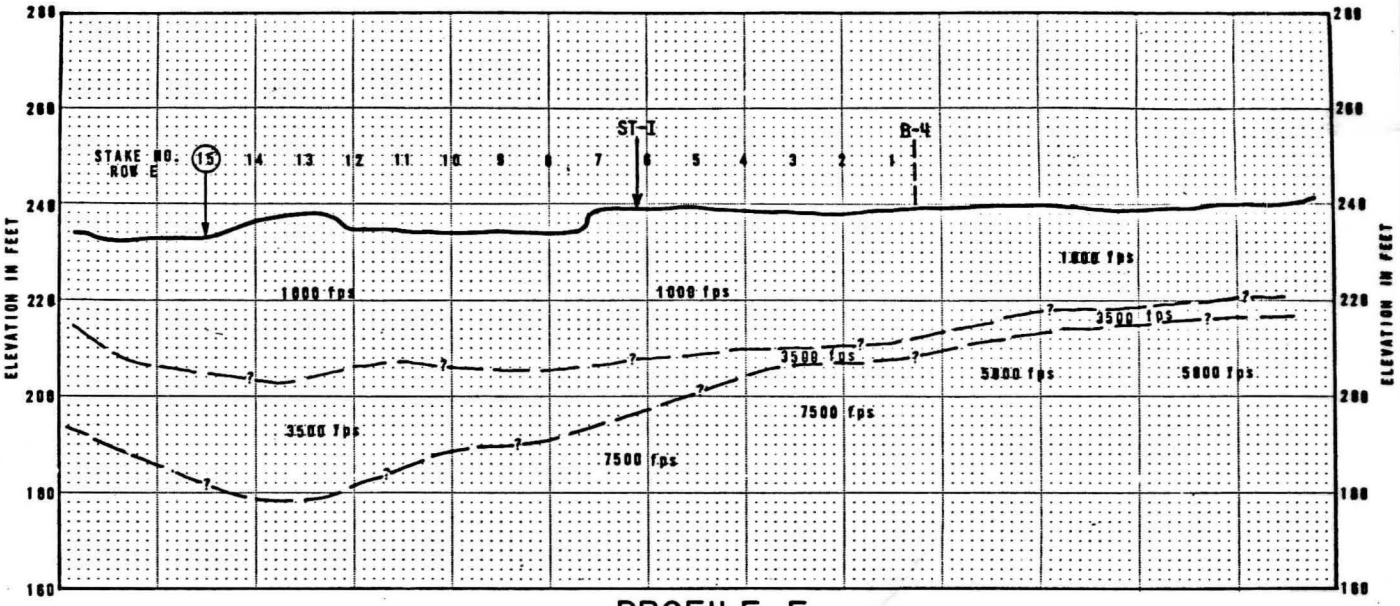
Geologic and engineering factors must now be evaluated in addition to obtaining selected samples for final evaluation and for laboratory testing of critical clays to determine the proper chemicals to achieve optimum strength increases. This phase of the work will require a field evaluation and selection of samples by a member of the Ion Tech staff. Our fee for such work is $175.00 per day and expenses. There is no charge for laboratory work.

Test Result Data:

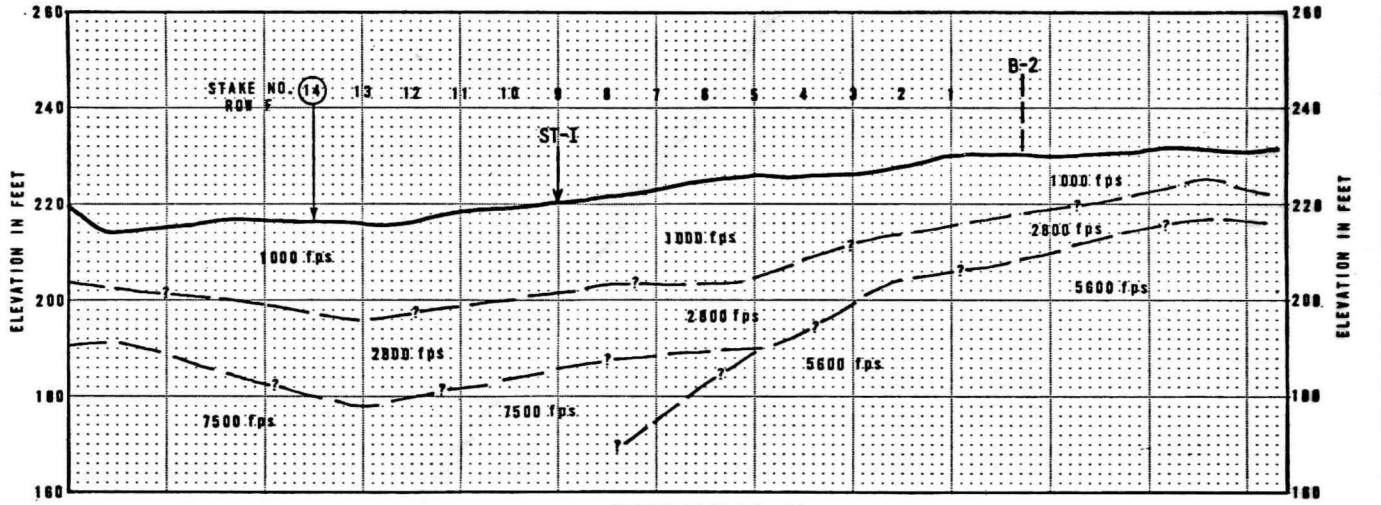
Sample #1 - (8' deep from slide) - Clayey silt with lenses or zones of fat clay. pH 5.2. Natural Moisture 45.4%.

Samples were prepared by drying and remolding. Vane shear tests were run on samples at about 45% moisture. A 132% increase (using optimum chemical) in strength versus untreated control sample was obtained in 24 hours. Unconfined compression tests were run on treated and untreated samples compacted to a moist unit weight of 116 pounds/ft^3, 15 minutes after chemical application. Untreated had a U_c of 2519 psf at a strain of 11.4% and the treated had a U_c of 4076 psf at a strain of 8.9%.

Sample #2 (cut slope sample) - Clayey silt with some clay. pH 5.2. Natural moisture 46.6%.



PROFILE E



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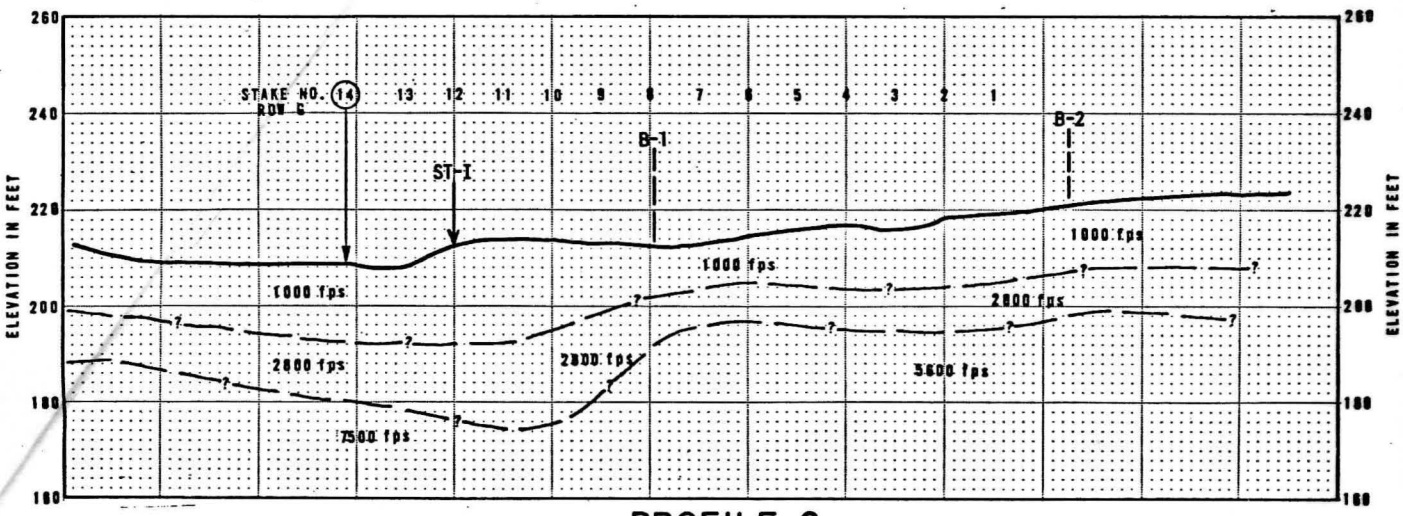

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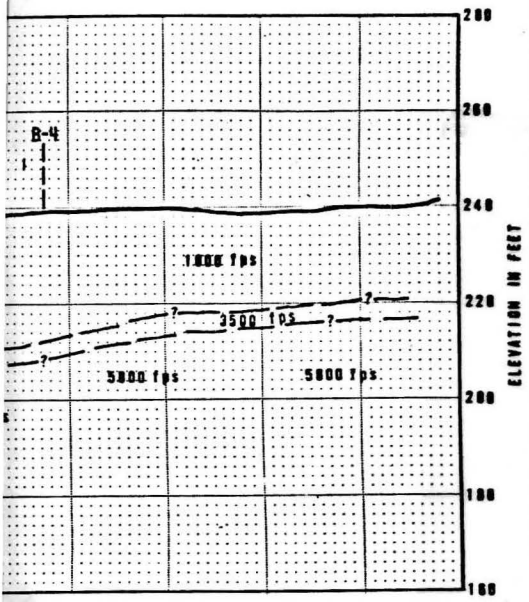

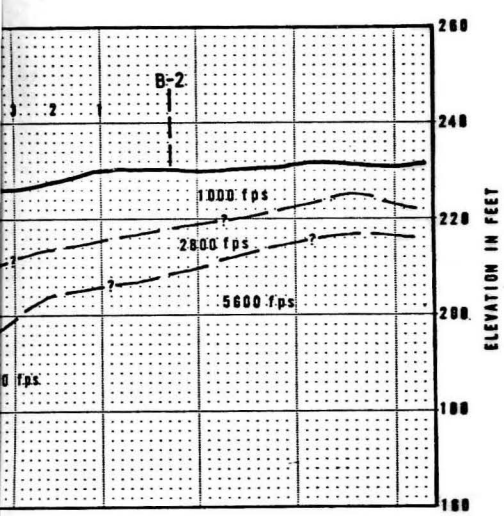

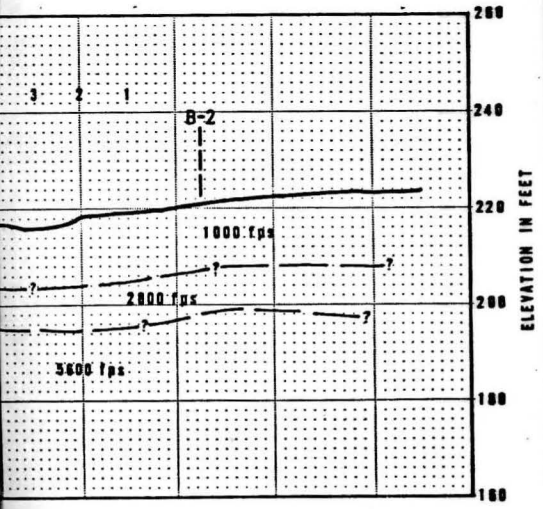

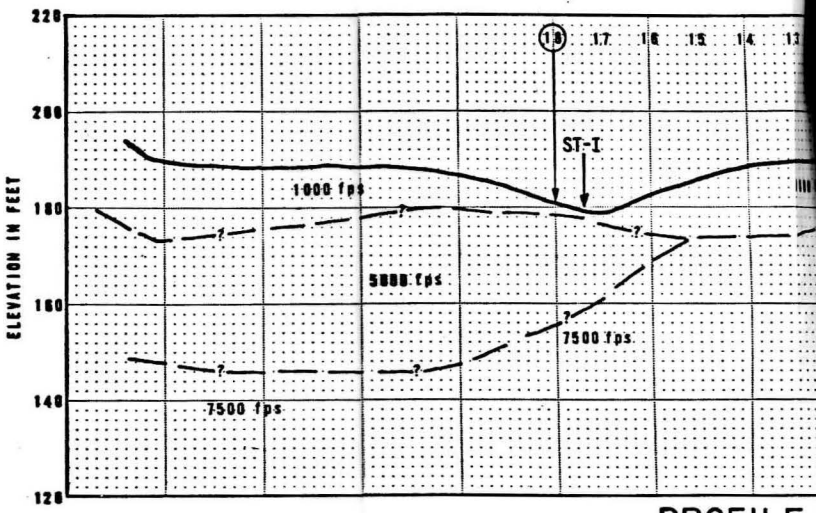

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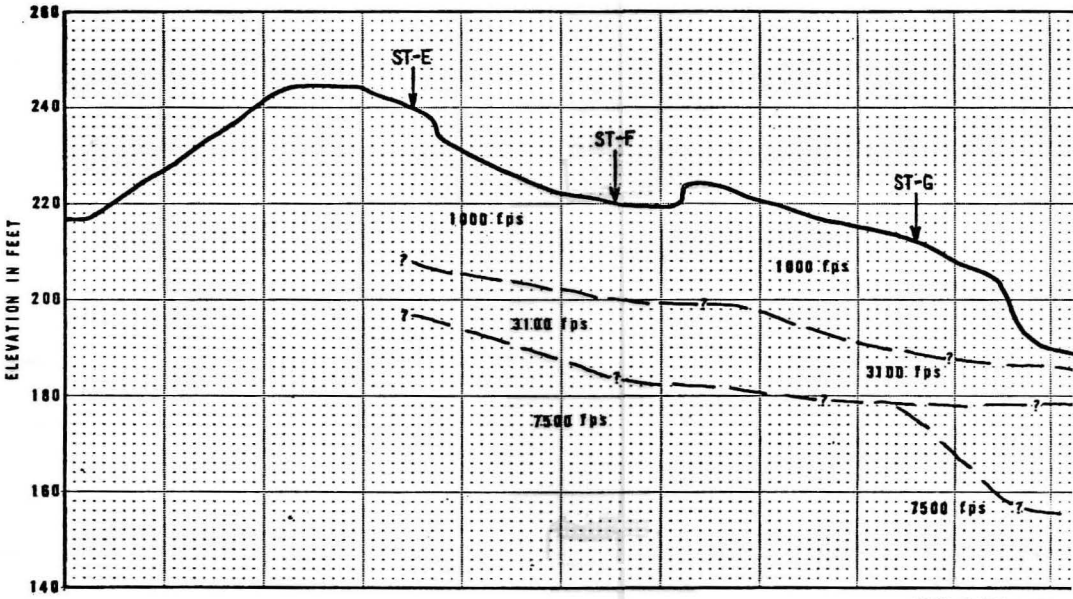

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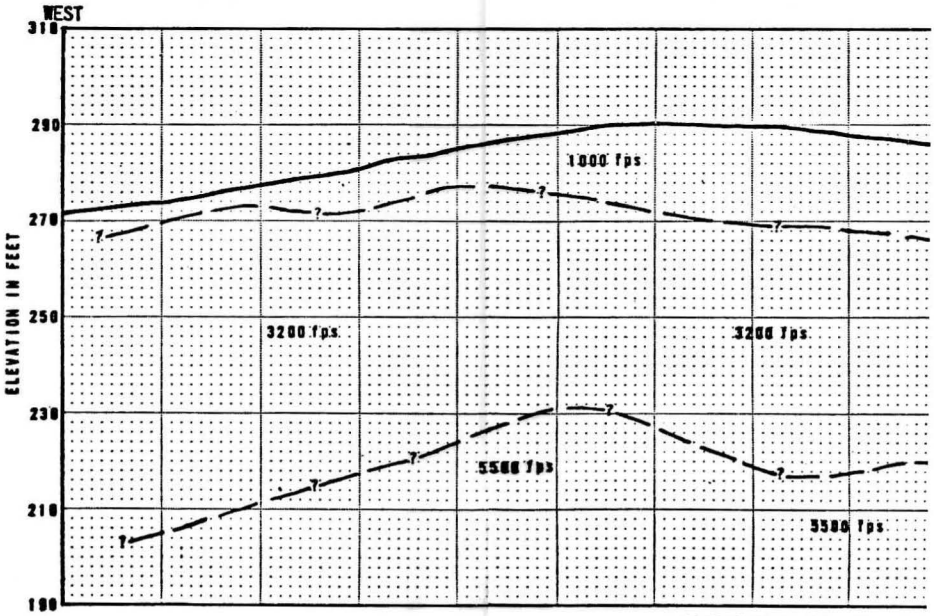
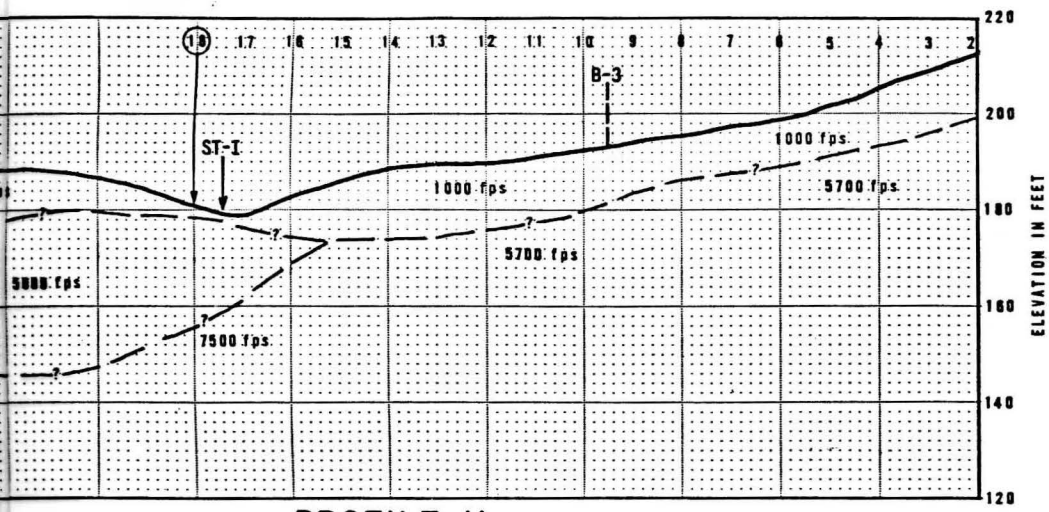


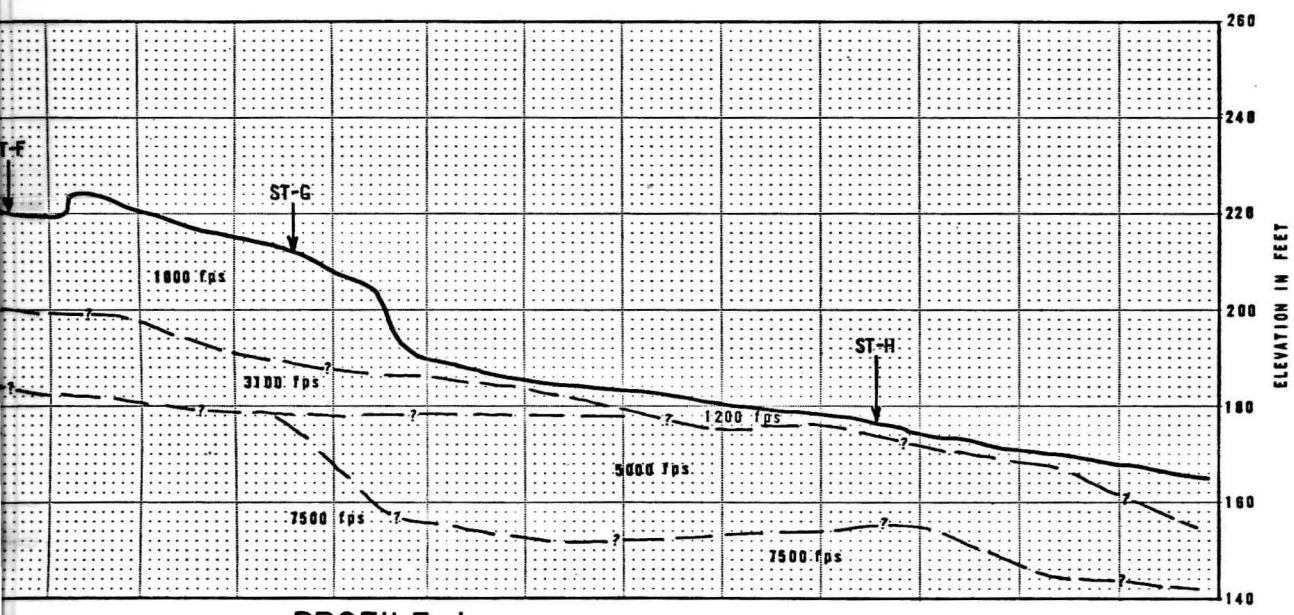
PROFILE J



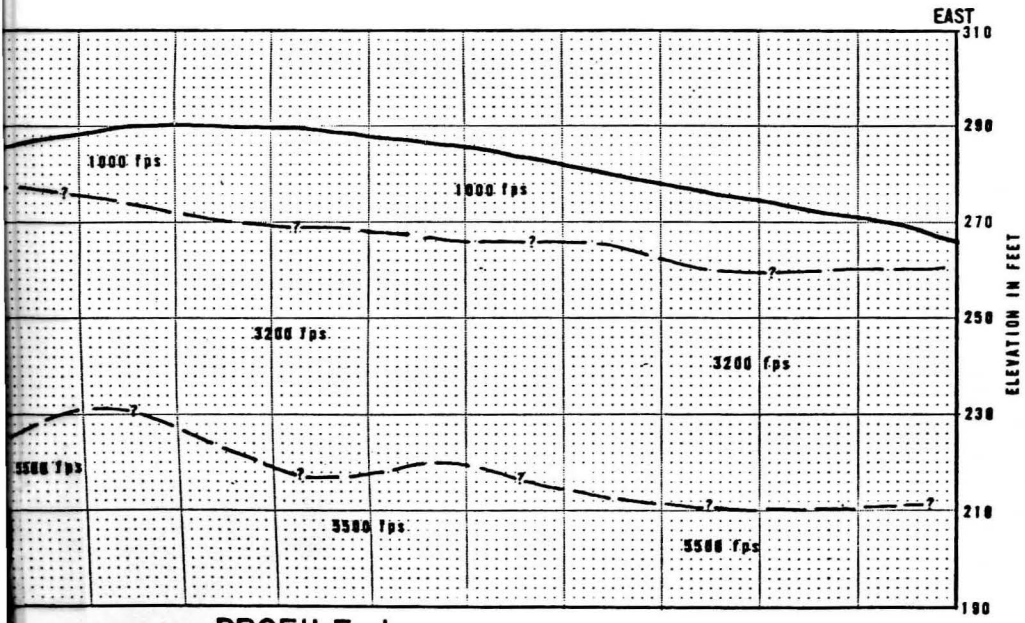
PROFILE H

GEOPHYSICAL INFORMATION IS BASED UPON GEOPHYSICAL MEASUREMENTS MADE BY GENERALLY ACCEPTED METHODS AND FIELD PROCEEDURES AND OUR INTERPRETATION OF THESE DATA. GEOLOGICAL INFORMATION IS BASED UPON OUR BEST ESTIMATE OF SUBSURFACE CONDITIONS CONSIDERING THE GEOPHYSICAL RESULTS AND ALL OTHER INFORMATION AVAILABLE TO US. THESE RESULTS ARE INTERPRETIVE IN NATURE AND ARE CONSIDERED TO BE A REASONABLY ACCURATE PRESENTATION OF EXISTING CONDITIONS WITHIN THE LIMITATIONS OF METHOD OR METHODS EMPLOYED.

fps = VELOCITY IN FEET PER SECOND



PROFILE I



PROFILE J

JOHN W. CUNNINGHAM & ASSOCIATES
CITY OF ABERDEEN

FAIRVIEW RESERVOIR NO. I

SEISMIC VELOCITY PROFILES

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FIG. 4

The chemistry of this sample showed a marked change from Sample #1. The optimum chemicals for Sample #2 were not the same as Sample #1. Samples were prepared by drying and remolding. Vane shear tests were run on samples at about 45% moisture. A 90% increase (using optimum chemical) in strength versus untreated control sample was obtained in 24 hours.

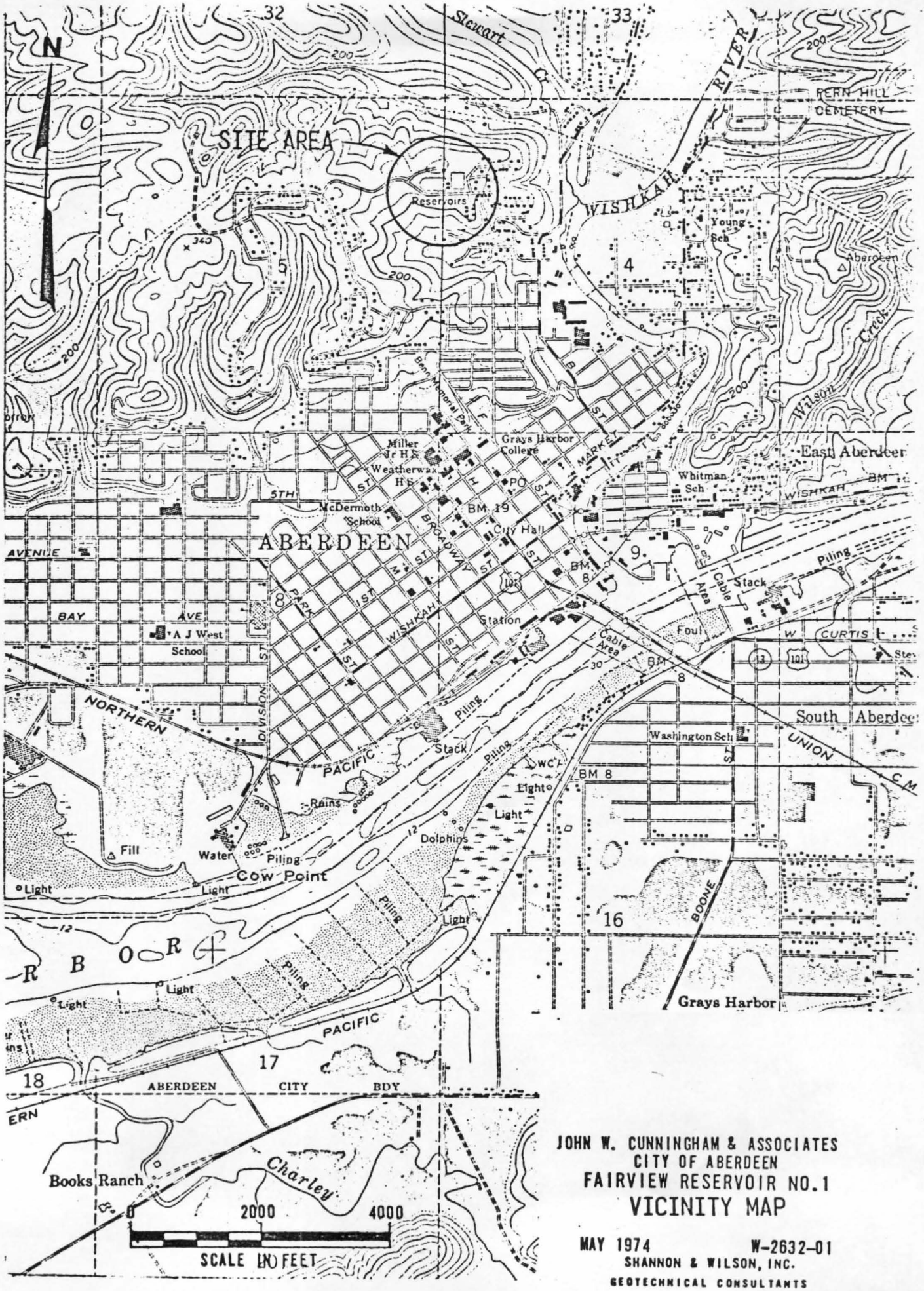
Very truly yours,

ION TECH, INC.

James B. Scott
vm

James B. Scott
Chief Engineering Geologist

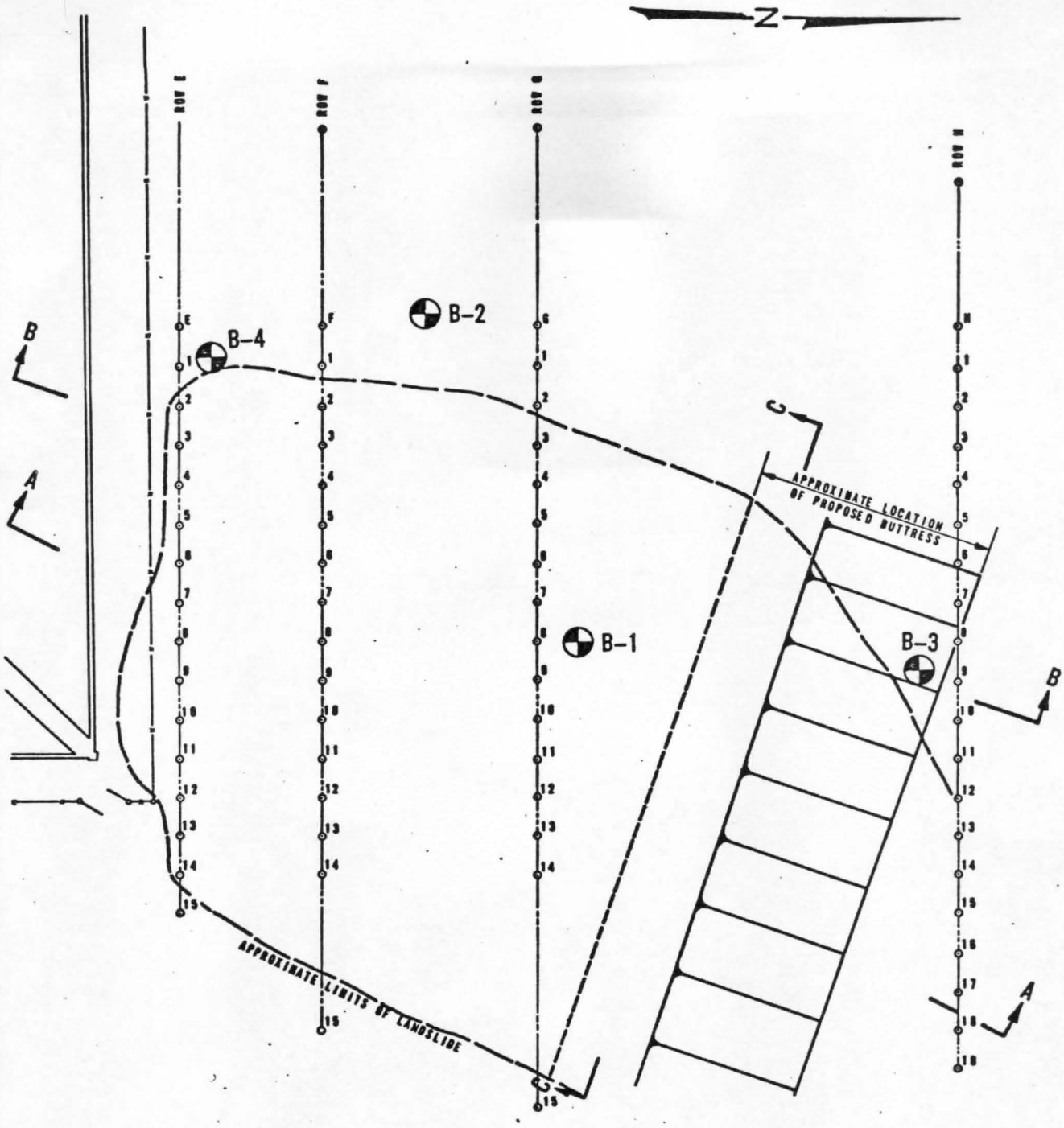
JBS/vm



JOHN W. CUNNINGHAM & ASSOCIATES
CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO.1
VICINITY MAP

MAY 1974 W-2632-01
SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

FIG. 1



LEGEND

ROW E = SEISMIC VELOCITY PROFILE LINE

0 20 40 60 80 100

SCALE IN FEET

JOHN W. CUNNINGHAM & ASSOCIATES
CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO.1
DETAILED SITE PLAN

MAY 1974 W-2632-01

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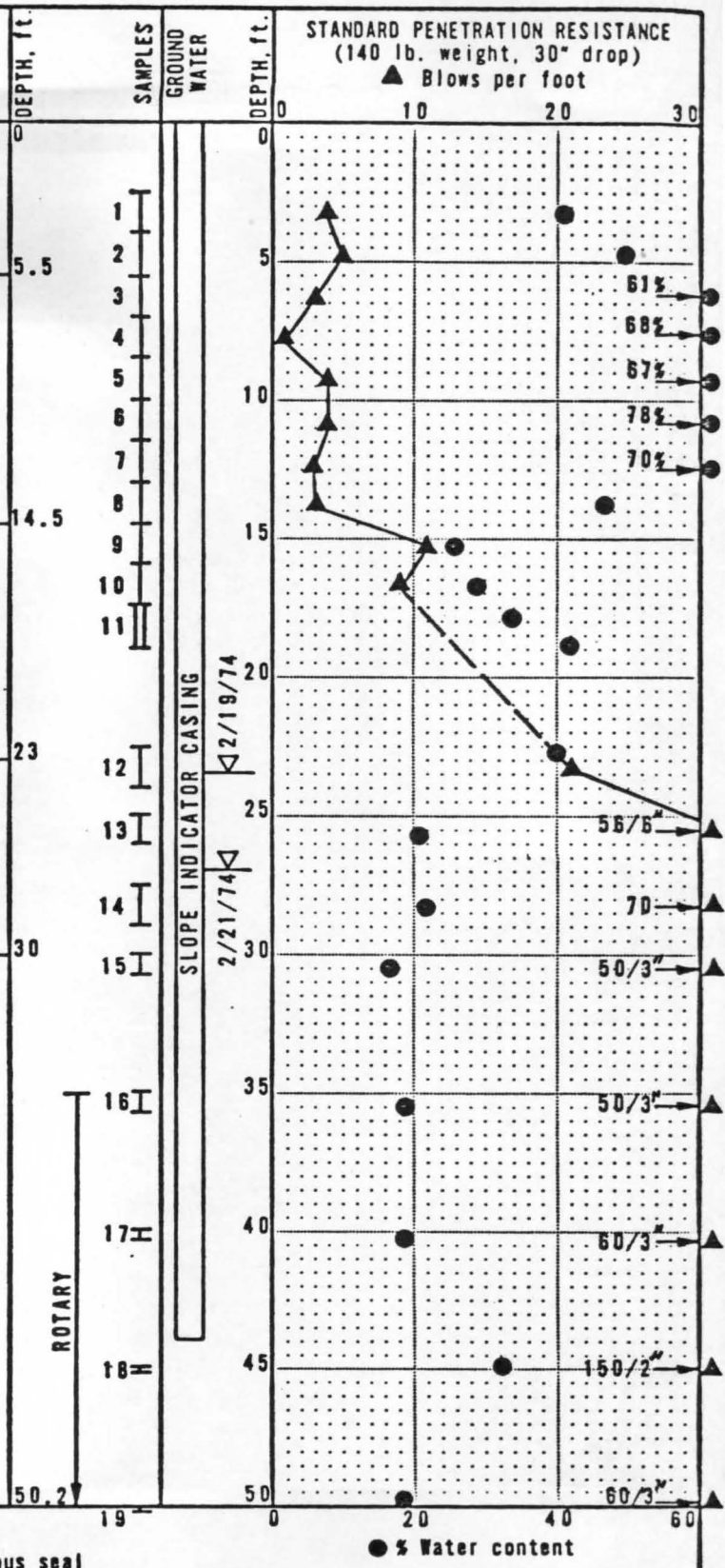
FIG. 3

SOIL DESCRIPTION

Surface Elevation: 211.85 feet

Description	Depth, ft.
Medium stiff, brown, clayey SILT, scattered fine gravel, trace of fine sand & organics (FILL)	0 – 5.5
Very soft to soft, dark brown, organic, clayey SILT w/roots, trace of fine sand (Soil)	5.5 – 14.5
Stiff, light gray-brown, clayey SILT w/ trace of fine sand & black organics (Soil and highly weathered rock)	14.5 – 23
Hard, gray, silty CLAY to clayey SILT, trace of fine sand & black organics (Weathered SILTSTONE)	23 – 30
Hard, dark gray, fine sandy, clayey SILT (Weathered SILTSTONE)	30 – 50.2

Bottom of Boring
Completed 2/18/74



LEGEND

- I 2" O.D. split spoon sample
- II 3" O.D. thin-wall sample
- * Sample not recovered

Atterberg Limits:

- Liquid limit
- Natural water content
- Plastic limit

- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed
- USC Unified Soil Classification

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

JOHN W. CUNNINGHAM & ASSOCIATES
CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO.1

LOG OF BORING B-1

MAY 1974 W-2632-01

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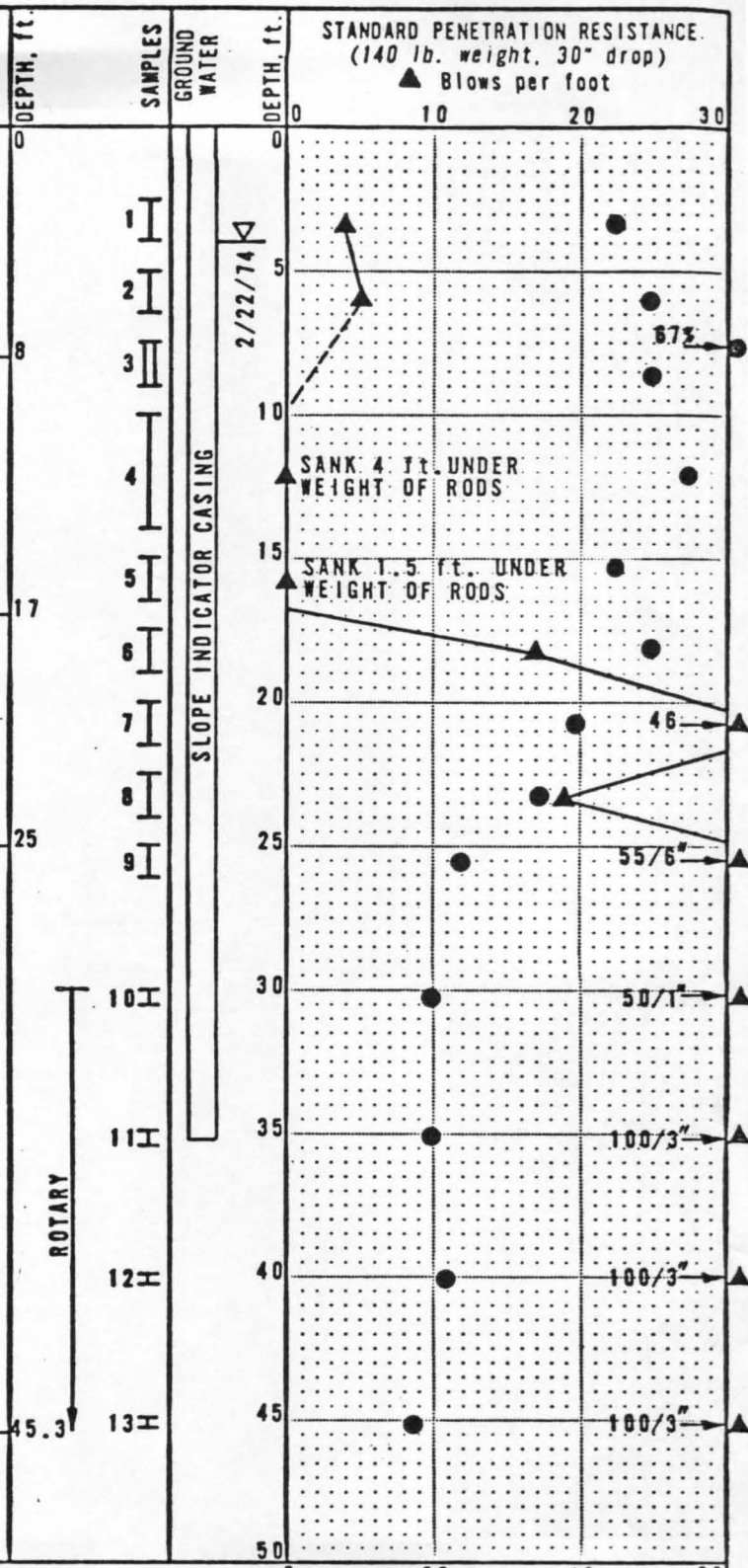
GEOTECHNICAL CONSULTANTS

FIG. 5

SOIL DESCRIPTION

Surface Elevation: 224.43 feet

Description	Depth, ft.
Medium stiff, brown, clayey SILT, scattered fine gravel, trace of sand & organics (FILL)	0 – 8
Very soft, dark brown, fine sandy, clayey SILT w/numerous organics (Soil)	8 – 17
Very stiff to hard, brown & gray, below 21 feet, clayey SILT (Weathered SILTSTONE)	17 – 25
Very dense, dark gray, silty, fine SAND (SANDSTONE)	25 – 45.3
Bottom of Boring Completed 2/19/74	45.3



LEGEND

- I 2" O.D. split spoon sample
- II 3" O.D. thin-wall sample
- * Sample not recovered
- Atterberg Limits:
  - Liquid limit
  - Natural water content
  - Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed
- USC Unified Soil Classification

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CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO.1

LOG OF BORING B-2

MAY 1974 W-2632-01

SHANNON & WILSON, INC.
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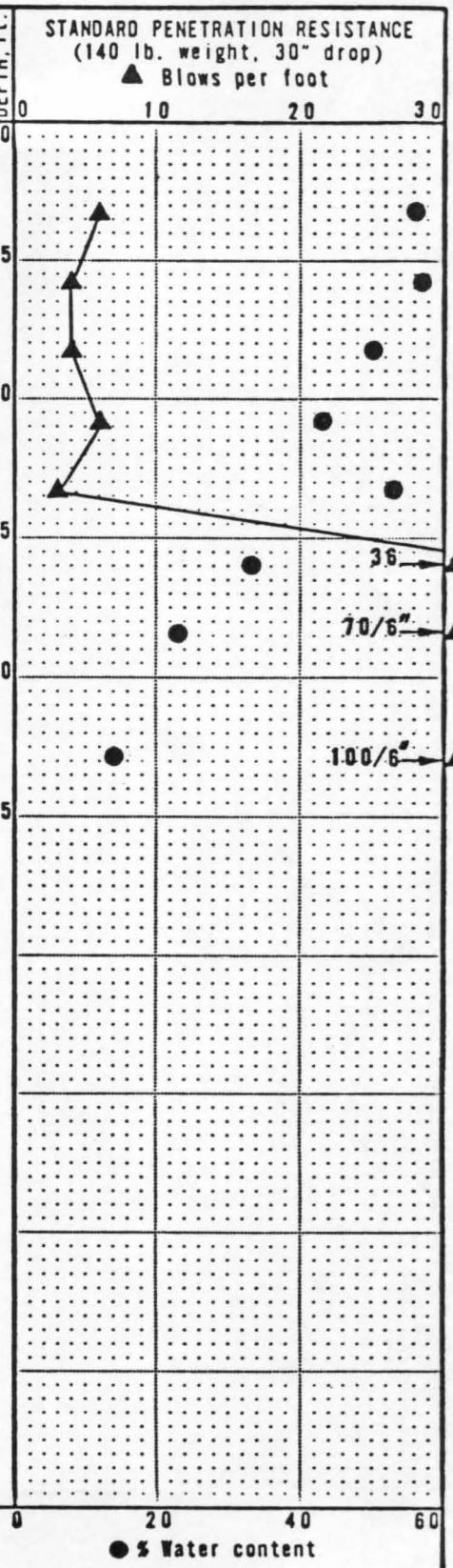
NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

FIG. 6

SOIL DESCRIPTION Surface Elevation: 197.16 feet	DEPTH, ft.	SAMPLES
Medium stiff, brown, fine sandy, clayey SILT w/trace of organics (FILL) and Soil	0	1, 2, 3, 4
Soft, brown, sandy SILT (13.5-14 ft. disturbed-possible slide plane)	12	5
Very stiff, brown, clayey SILT (highly weathered SILTSTONE) trace of fine sand	14.5	
Hard, dark gray, clayey SILT, trace of fine sand (Weathered SILTSTONE)	16	6, 7
Bottom of Boring Completed 2/15/74	23	8

GROUND WATER: ¼ in. PVC



LEGEND

I 2" O.D. split spoon sample

II 3" O.D. thin-wall sample

* Sample not recovered

Atterberg Limits:

- Liquid limit
- Natural water content
- Plastic limit

Impervious seal

Water level

Piezometer tip

P Sample pushed

USC Unified Soil Classification

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

JOHN W. CUNNINGHAM & ASSOCIATES
CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO.1

LOG OF BORING B-3

MAY 1974 W-2632-01

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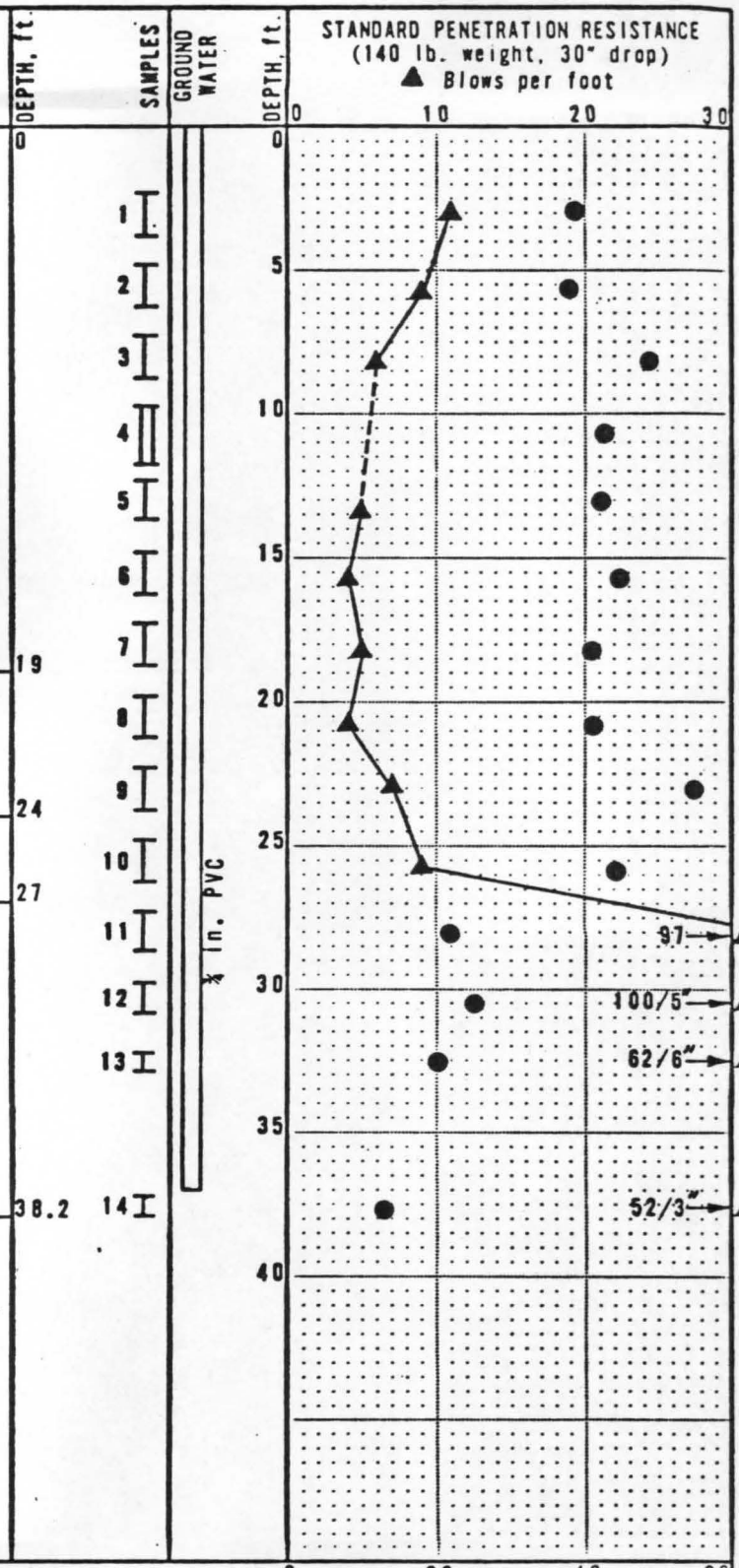
FIG. 7

SOIL DESCRIPTION

Surface Elevation: 236.5 feet

Description	Depth, ft.
Stiff to medium stiff, brown & gray, fine sandy, clayey SILT w/chunks of hard, gray, clay & trace of fine gravel (FILL)	0 – 19
Medium stiff, brown & gray, fine sandy, clayey SILT w/trace of fine sand & numerous organics (Soil)	19 – 24
Loose, gray, slightly clayey, silty, fine SAND & stiff, gray-brown, silty CLAY, intermixed (Highly weathered SILTSTONE)	24 – 27
Hard, gray, fine sandy, clayey SILT (SILTSTONE)	27 – 38.2
Bottom of Boring Completed 2/15/74	38.2



LEGEND

I 2" O.D. split spoon sample

II 3" O.D. thin-wall sample

* Sample not recovered

Atterberg Limits:

- Liquid limit
- Natural water content
- Plastic limit

Impervious seal

Water level

Piezometer tip

P Sample pushed

USC Unified Soil Classification

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual.

JOHN W. CUNNINGHAM & ASSOCIATES
CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO.1

LOG OF BORING B-4

MAY 1974 W-2632-01

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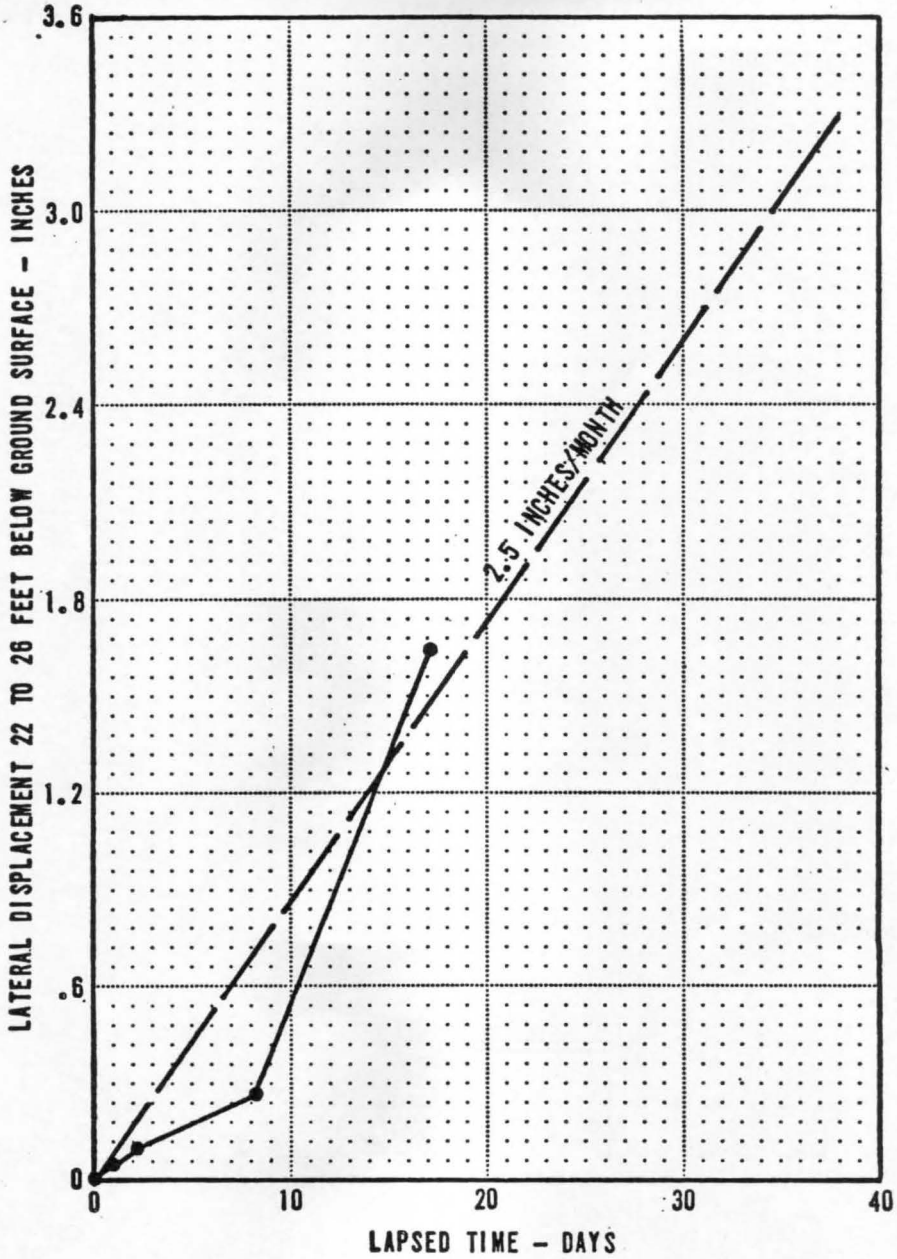
GEOTECHNICAL CONSULTANTS

FIG. 8



NOTE: At 69 days (4/29/74) the probe was unable to bypass the zone of deformation at 23.3 feet

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CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO.1

B-1 LATERAL DEFORMATION - SLIDE PLANE

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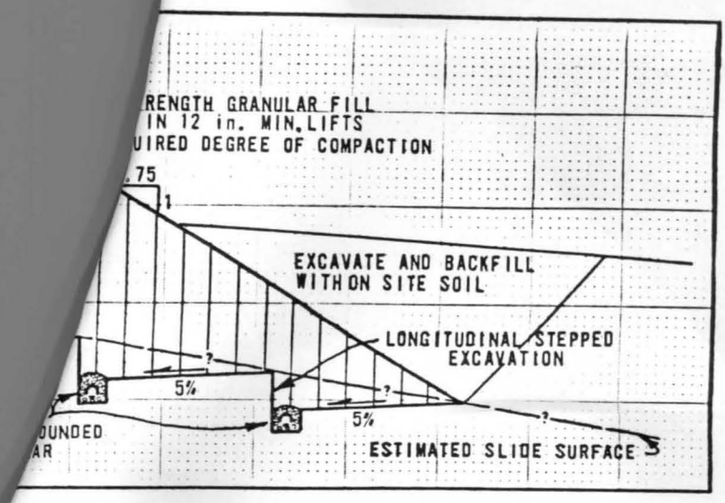
GEOTECHNICAL CONSULTANTS

FIG. 9



GRANULAR MATERIAL CONFORMING
CKFILL FOR DRAINS" AS SPECIFIED
-03.12(4) OF THE STATE OF WASHINGTON
ECIFICATION FOR ROAD AND BRIDGE
ON, 1972.

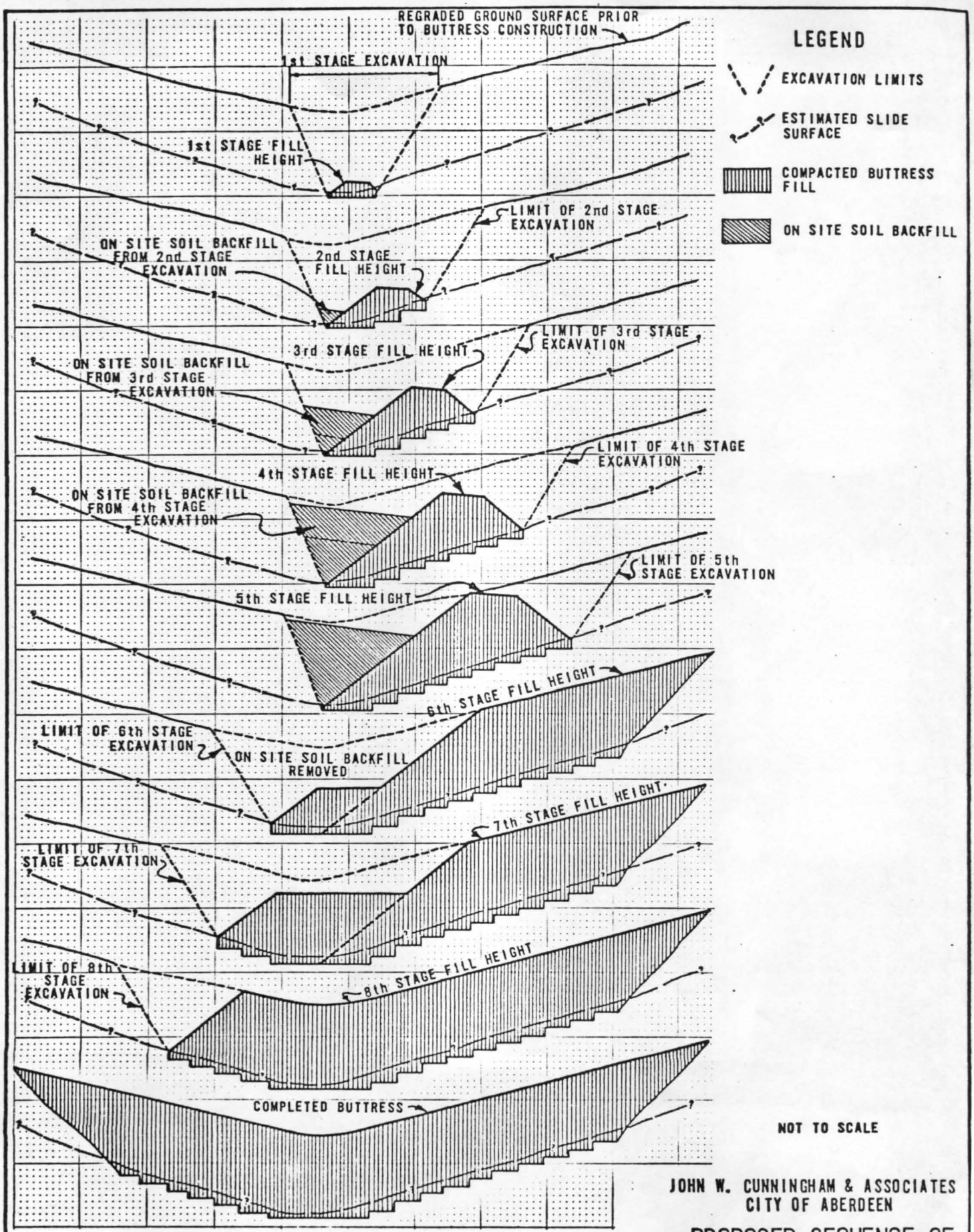
BUTTRESS DETAIL

JOHN W. CUNNINGHAM & ASSOCIATES
CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO.1
SUBSURFACE PROFILES AND
BUTTRESS DETAIL

MAY 1974 W-2632-01

SHANNON & WILSON, INC. FIG. NO.
GEOTECHNICAL CONSULTANTS

FIG. 10



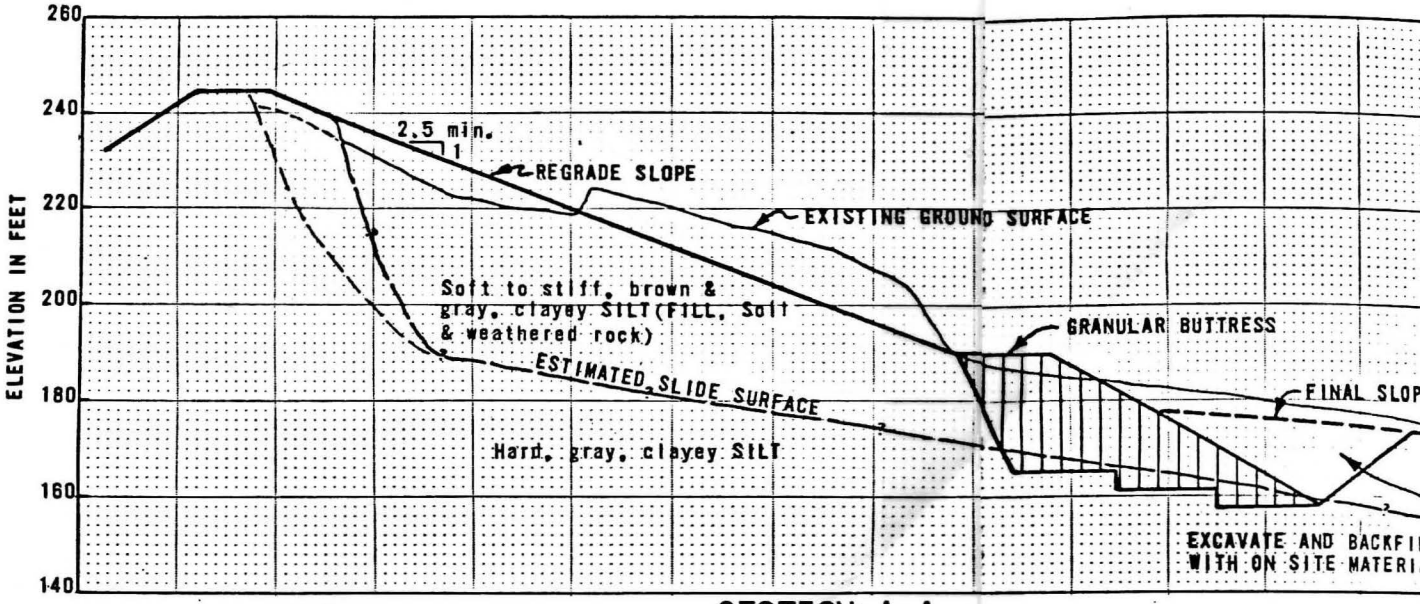
JOHN W. CUNNINGHAM & ASSOCIATES
CITY OF ABERDEEN

PROPOSED SEQUENCE OF BUTTRESS CONSTRUCTION

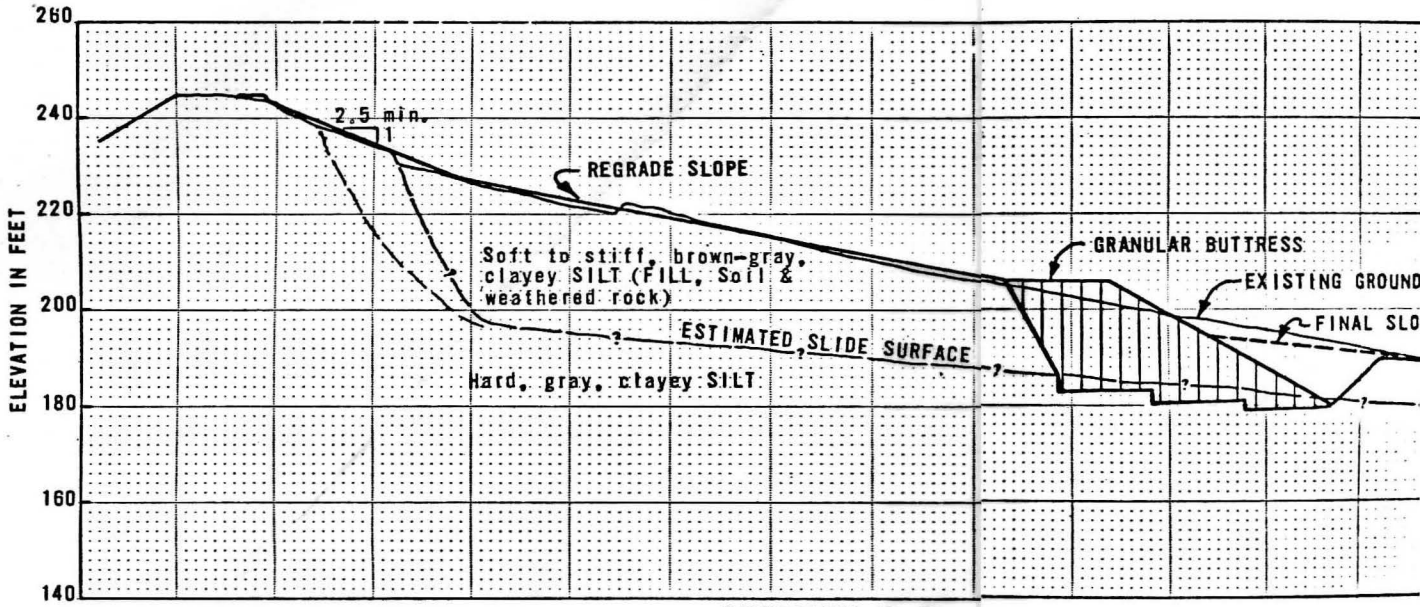
MAY 1974 W-2632-01

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

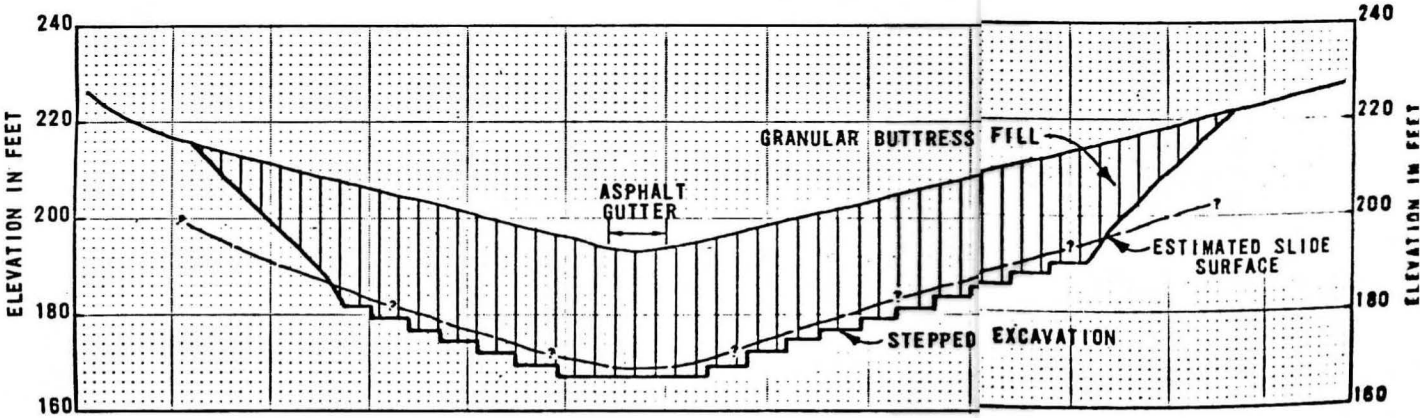
FIG. 11



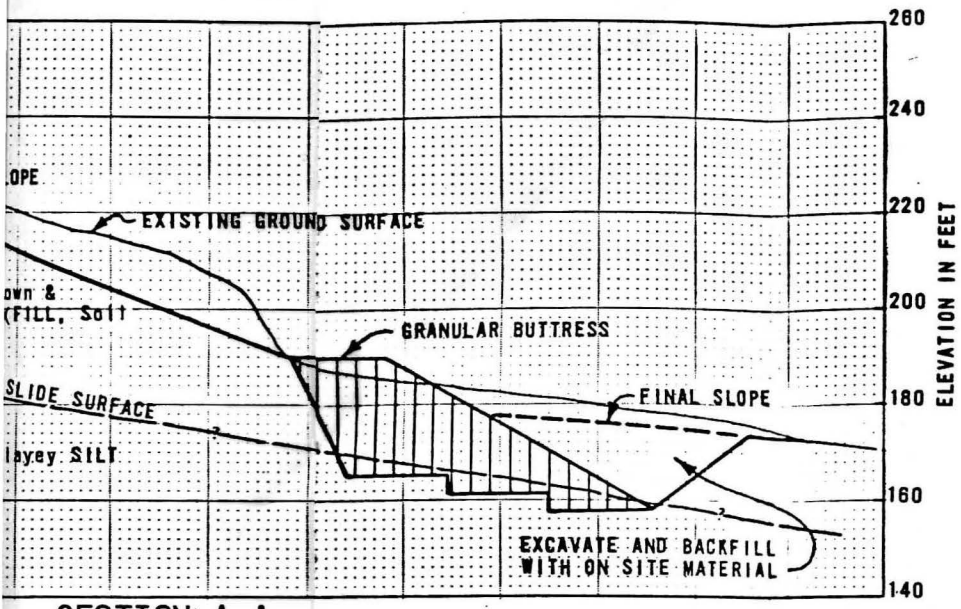
SECTION A-A



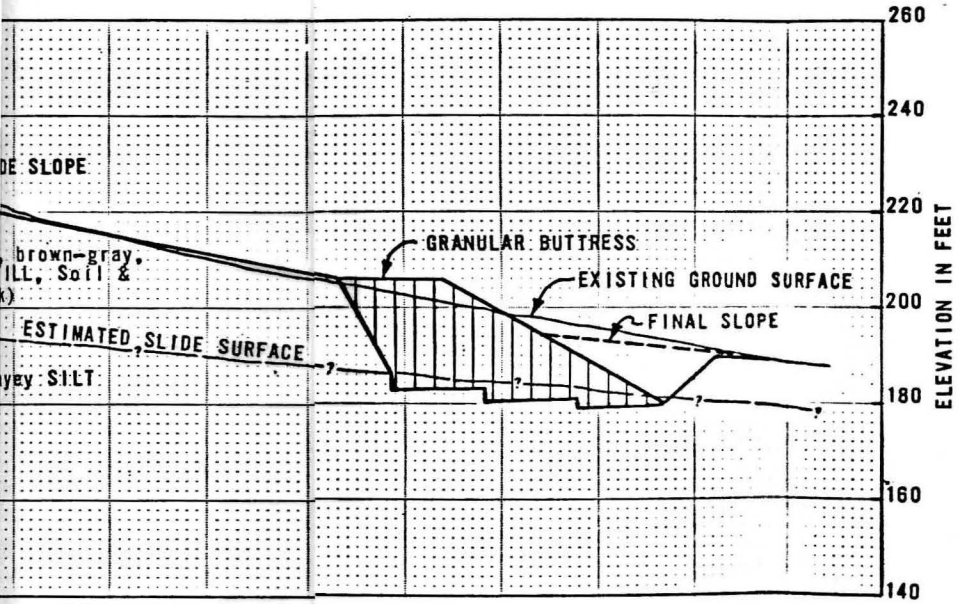
SECTION B-B



SECTION C-C



SECTION A-A



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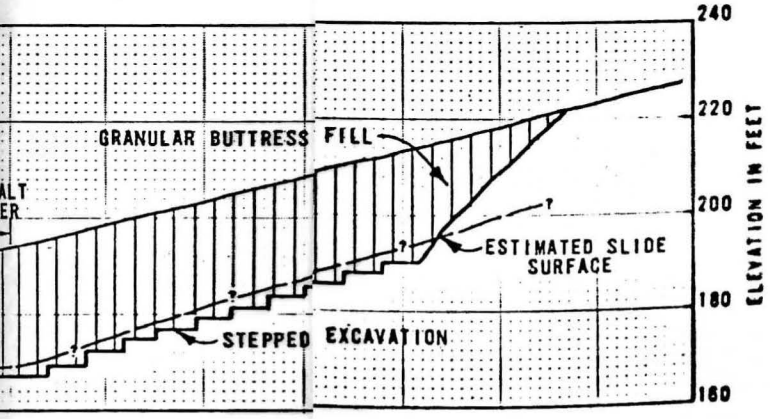
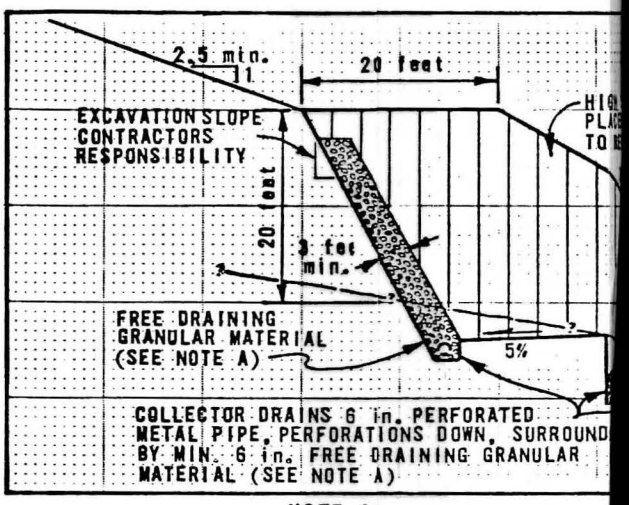


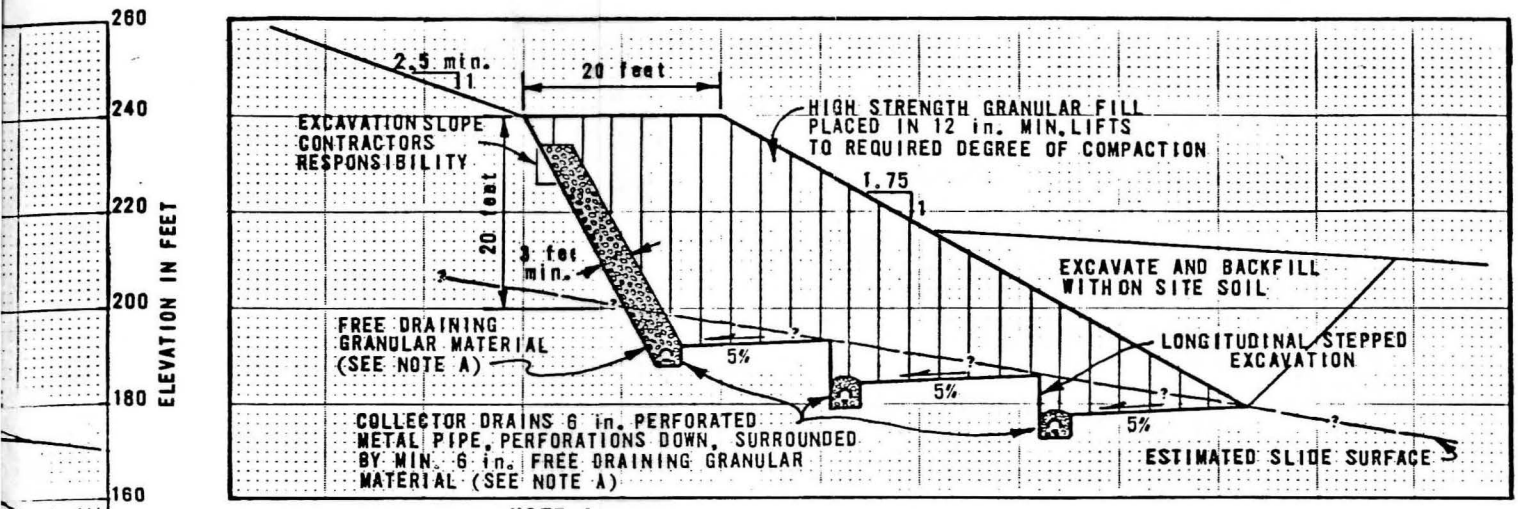
ECTION C-C



NOTE A:

FREE DRAINING GRANULA
TO "GRAVEL BACKFILL F
IN SECTION 9-03.12(4)
STANDARD SPECIFICATIO
CONSTRUCTION, 1972.

BUTTRES



NOTE A:

FREE DRAINING GRANULAR MATERIAL CONFORMING TO "GRAVEL BACKFILL FOR DRAINS" AS SPECIFIED IN SECTION 9-03.12(4) OF THE STATE OF WASHINGTON STANDARD SPECIFICATION FOR ROAD AND BRIDGE CONSTRUCTION, 1972.

BUTTRESS DETAIL

JOHN W. CUNNINGHAM & ASSOCIATES
CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO. 1

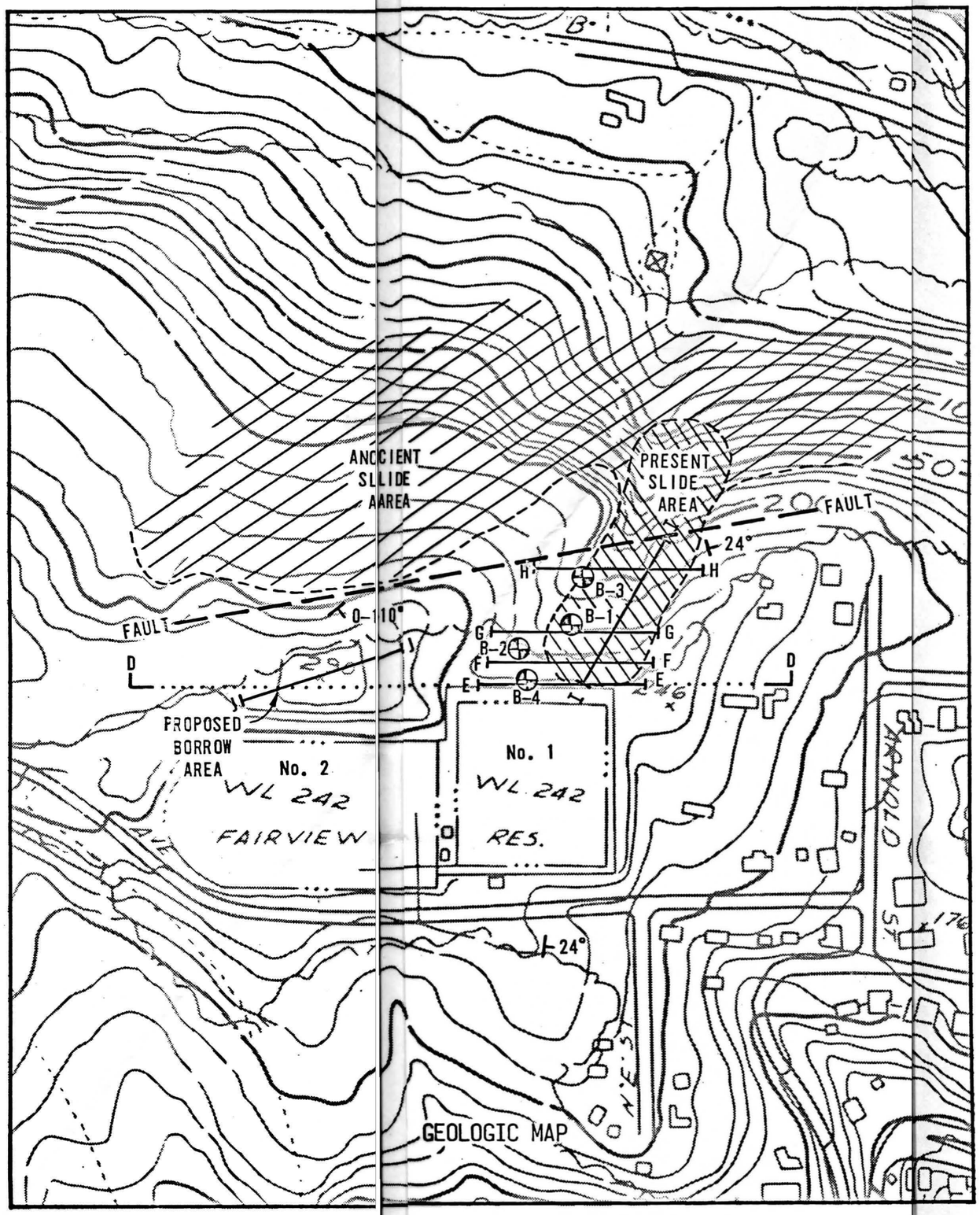
SUBSURFACE PROFILES AND BUTTRESS DETAIL

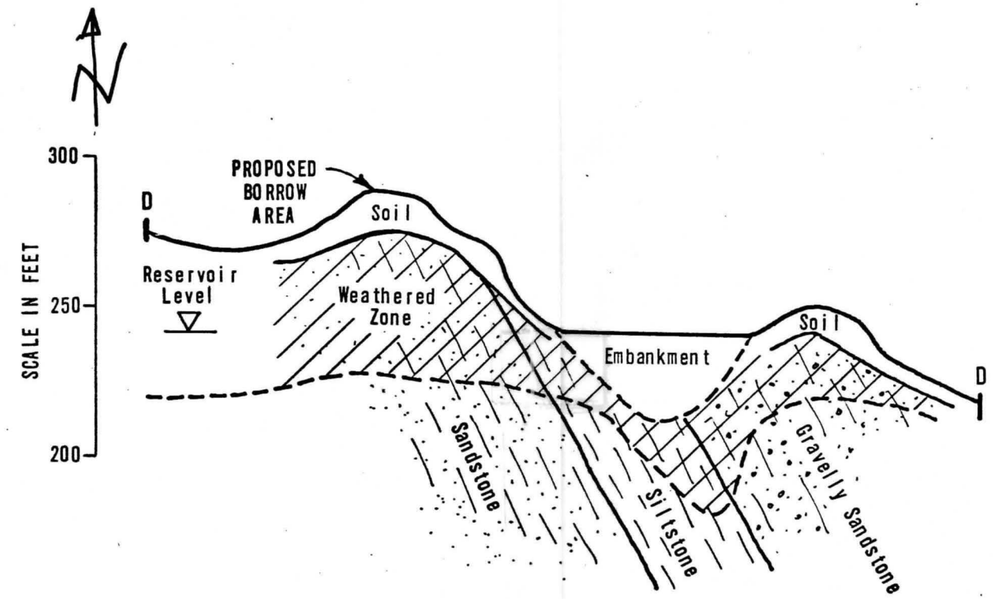
MAY 1974 W-2632-01

SHANNON & WILSON, INC.
GEOTECHNICAL CONSULTANTS

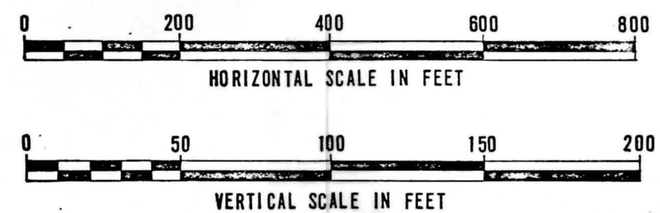
FIG. NO.

FIG. 10





TYPICAL GEOLOGIC SECTION



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CITY OF ABERDEEN
FAIRVIEW RESERVOIR NO.1
GEOLOGIC MAP
AND
CROSS SECTION

MAY 1974 W-2632-01

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FIG. 2