

PHASE I GEOTECHNICAL DATA REPORT  
ROBERTSON BOULEVARD OVER INTERSTATE 395  
BRIDGE NO. 1560, MAINEDOT WIN 029484.00  
BREWER, MAINE

by  
Haley & Aldrich, Inc.  
Portland, Maine

for  
Maine Department of Transportation  
Augusta, Maine

File No. 0210037-000  
May 2026





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May 15, 2026  
File No. 0210037-000

Maine Department of Transportation  
16 State House Station  
Augusta, Maine 04333-0016

Attention: Laura Krusinski, P.E.  
Senior Geotechnical Engineer

Subject: Phase I Geotechnical Data Report  
Robertson Boulevard over Interstate 395  
Bridge No. 1560, MaineDOT WIN 029484.00  
Brewer, Maine

Ladies and Gentlemen:

This Phase I Geotechnical Data Report presents the compilation of subsurface data and results of the historical geotechnical field investigations completed for construction of the existing Robertson Boulevard bridge (existing bridge) in Brewer, Maine (see Figures 1 and 2). This report is intended to provide MaineDOT and their bridge subconsultant (HNTB Corporation; HNTB) with initial geotechnical information for the proposed bridge replacement. This work has been completed in accordance with our proposal dated March 5, 2024, which was authorized on March 18, 2024. This report supersedes our November 21, 2025 report.

A site-specific field investigation has been conducted to support development of the design build (DB) request for proposals (RFP) document and is summarized in the Phase II Geotechnical Data Report dated May 15, 2026.

## **Project Background**

### **EXISTING BRIDGE STRUCTURE**

The existing 202-foot (ft)-long, two-span bridge carries Robertson Boulevard over Interstate 395 (I-395; see Figure 2). Based on our review of the 1984 historical bridge drawings, the existing bridge substructures are supported on plumb and battered (inclined at 3 horizontal:12 vertical) steel H-piles end-bearing in bedrock. The historical bridge drawings (Sheet No. 5) indicate that the maximum calculated pile load is 118 tons (236 kips). A summary of the existing bridge piles (based on the historical bridge drawings Sheet No. 2 and the pile notes on Sheet No. 5) is provided below.

Existing Bridge Substructure	Steel H-Pile Section	Number of Piles	Estimated Pile Length (ft)
Abutment No. 1	HP14x73	19	56
Pier	HP14x89	10	33
Abutment No. 2	HP14x73	23	49



Photograph 1 – Abutment No. 1 (north side of existing bridge) supported on steel HP14x73 H-piles.





Photograph 2 – Pier supported on steel HP14x89 piles.



Photograph 3 – Abutment No. 2 (south side of existing bridge) supported on steel HP14x73 piles.

## **PROPOSED BRIDGE STRUCTURE**

Based on discussions with HNTB, the project will include a full bridge replacement.

## **Geologic Setting**

According to Maine Geological Survey's Bangor Surficial Geology Quadrangle, Maine (2011), the surficial geologic unit mapped within the site vicinity is the Presumpscot Formation which consists of silt, clay, and sand. According to Maine Geological Survey's Bangor Bedrock Geology Quadrangle, Maine (2011), bedrock at the site vicinity is mapped as the Bangor Formation of the Penobscot River Member which consists of Silurian Age medium- to very fine-grained feldspathic metawacke.

## **Historical Geotechnical Field Investigations**

Two phases of geotechnical field investigations (investigations) were conducted at the subject site by MaineDOT in 1980 and 1982. The results of these investigations are summarized in the report titled, "Soils Report 82-27, Brewer – Penobscot County, Industrial Park Rd. over I-395, Project 395-8(74)," dated June 1982 (Soils Report) and is included for reference in Appendix A. Based on Sheet Nos. 6 to 8 in the Soils Report, the investigations consisted of conducting seven wash borings (borings) to support design and construction of the existing bridge. Please note that a reference elevation datum was not indicated in the Soils Report. Refer to Figure 2 for approximate locations of the historical borings.

## **Generalized Subsurface Conditions**

The subsurface conditions encountered in the investigations generally consisted of the following geologic units presented in order of increasing depth below ground surface (BGS) along the existing bridge alignment: in-situ fill, marine deposit, glacial till, and bedrock.

A general description of each geologic and bedrock unit encountered in the available historical borings is provided separately below.

## GEOLOGIC UNIT DESCRIPTIONS

Geologic Unit	Approximate Range in Encountered Thickness (ft)	Generalized Description
In-situ Fill	0 to 4	Medium dense <sup>1</sup> , brown, silty SAND and GRAVEL.
Marine Deposit <sup>2</sup>	37 to 50	Stiff, brown to grey, sandy Silty CLAY. <i>Note: upper portions of deposit are stiffer than lower portions.</i>
Glacial Till	5 to 22	Loose to dense, grey, gravelly SAND with “rocks.”

### Notes:

1. Please note that field blow counts per foot (i.e., uncorrected N-values) and corresponding densities in the table were based on a Sprague & Henwood soil sampler.
2. The Soils Report did not provide a geologic unit classification for these strata. Based on the descriptions of this stratum on the boring logs, and the surficial geology map of the site, we have classified these strata as a marine deposit.

## BEDROCK CONDITIONS

Bedrock was cored in each of the historical borings at the existing bridge substructures. In these borings, the top of the bedrock surface ranged from approximately 49 ft to 63 ft BGS (approximately El. 19 to approximately El. 5). The cored bedrock was generally described as metasiltstone, phyllite, and calcareous metagraywacke, with calcite veins and high angles of foliation.

The top of bedrock is generally flat at El. 5.3 at Abutment No. 1. At Abutment No. 2, the top of bedrock decreases slightly from east to west from approximately El. 13.6 to El. 11.9. In historical boring CB-18-82, which was drilled within the existing pier footprint, the top of bedrock was encountered at approximately El. 5.

## GROUNDWATER ELEVATIONS

Historical groundwater levels were not recorded. An indication of soil sample saturation was not indicated on the historical boring logs. However, per the global stability analyses provided in the Soils Report, the groundwater level was assumed at El. 60 (approximately 7 to 14 ft BGS).

## HISTORICAL IN-SITU FIELD VANE SHEAR STRENGTH TESTING

The historical in-situ field vane shear strength testing results conducted in the marine deposits are summarized in the table below, and details can be found in the Soils Report included in Appendix A.

Existing Substructure	Historical Boring	Approximate Elevation Range of Field Vane Shear Strengths (ft)	Approximate Range of Field Vane Shear Strengths (pounds per square-foot [psf])
Abutment No. 1	CB-19-82	60 to 27	1,240 to 1,740
	CB-20-82	60 to 19	920 to 1,980
	GP-60-80	67 to 49	1,720
	GP-60-80	49 to 18	1,000 to 1,620
Pier	CB-18-82	62 to 46	1,660
	CB-18-82	46 to 25	1,000 to 1,740
	JC-28-80	60 to 24	1,500 to 2,000
Abutment No. 2	CB-17-82	70 to 27	1,060 to 2,000
	CB-21-82	63 to 25	1,680 to 2,000

### HISTORICAL GEOTECHNICAL LABORATORY TESTING

The historical geotechnical laboratory testing results in the marine deposits are summarized in the tables below, and details can be found in the Soils Report included in Appendix A.

#### Atterberg Limits

Existing Substructure	Historical Boring	Approximate Range of Plastic Limits (%)	Approximate Range of Liquid Limits (%)
Abutment No. 1	GP-60-80	15 to 24	20 to 40
	CB-19-82	16 to 27	23 to 42
	CB-20-82	19 to 22	27 to 38
Pier	JC-28-80	16 to 20	24 to 40
Abutment No. 2	CB-21-82	18 to 22	29 to 33
	CB-17-82	14 to 23	27 to 38

**Strength Testing Results**

Existing Substructure	Historical Boring	Approximate Elevation Range of Field Vane Shear Strengths (ft)	Approximate Range of Lab Vane Shear Strengths (psf)
Abutment No. 1	CB-19-82	60 to 27	500 to 1,600
	CB-20-82	60 to 19	500 to 1,520
	GP-60-80	67 to 18	600 to 1,700
Pier	CB-18-82	62 to 25	600 to 1,140
	CB-18-82		
	JC-28-80	60 to 24	800 to 1,600
Abutment No. 2	CB-17-82	70 to 27	400 to 1,640
	CB-21-82	63 to 25	660 to 2,000

**Consolidation Results**

Existing Substructure	Historical Boring	Sample No.	Approximate Sample Elevation (ft)	Range in Water Content, WC (%)	Preconsolidation Pressure, P <sub>p</sub> (psf)	Virgin Compression Index, C <sub>c</sub> (unitless)
Abutment No. 1	GP-60-80	2U	56	29 to 33	Inconclusive <sup>1</sup>	0.17
	GP-60-80	5U	37	28 to 33	3,900	0.26
Abutment No. 2	CB-17-82	3U	58	26 to 30	4,000	0.17
	CB-17-82	6U	43	25 to 30	4,300	0.22

**Note:**

- The consolidation test results did not provide a preconsolidation pressure but indicated the minimum and maximum preconsolidation pressure was 2,200 psf and 9,800, respectively.



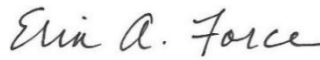
## Closure

We appreciate the opportunity to provide engineering services on this project. Please do not hesitate to contact us if you have any questions or comments.

Sincerely yours,  
**HALEY & ALDRICH, INC.**



Nathan A. Sherwood, P.E.  
Senior Project Manager



Erin A. Force, P.E.  
Senior Associate



Enclosures:

- Figure 1 – Project Locus
- Figure 2 – Historical Boring Location Plan
- Appendix A – Historical Soils Report

<https://haleyaldrich.sharepoint.com/sites/MaineDepartmentofTransportation2/Shared Documents/0210037.MaineDOT-Brewer I-395 Design Build/Deliverables/Phase 1 - Historic Geotech Data Reports/Robertson Blvd over I395 No. 1560/2026-0515-HAI-I395-Robertson Blvd Bridge-Phase I GR-F.docx>

## References

1. Syverson, Kent M., & Thompson, Andrew H., *Surficial Geology Bangor Quadrangle, Maine*, Maine Geological Survey, Department of Conservation, Augusta, Maine, Open File Report No. 11-6, 2011.
2. Pollock, Stephen G., *Bedrock Geology of the Bangor Quadrangle, Maine*, Maine Geological Survey, Department of Conservation, Augusta, Maine, Open File Report No. 11-57, 2011.

<https://haleyaldrich.sharepoint.com/sites/MaineDepartmentofTransportation2/Shared Documents/0210037.MaineDOT-Brewer I-395 Design Build/Deliverables/Phase 1 - Historic Geotech Data Reports/Robertson Blvd over I395 No. 1560/2026-0515-HAI-I395-Robertson Blvd Bridge-Phase I GR-F.docx>

## FIGURES



0210037.001 LOCUS HALEYALDRICHUBOIS



SITE COORDINATES: 44°46'57"N, 68°45'54"W



MAP SOURCE: USGS

**HALEY  
ALDRICH**

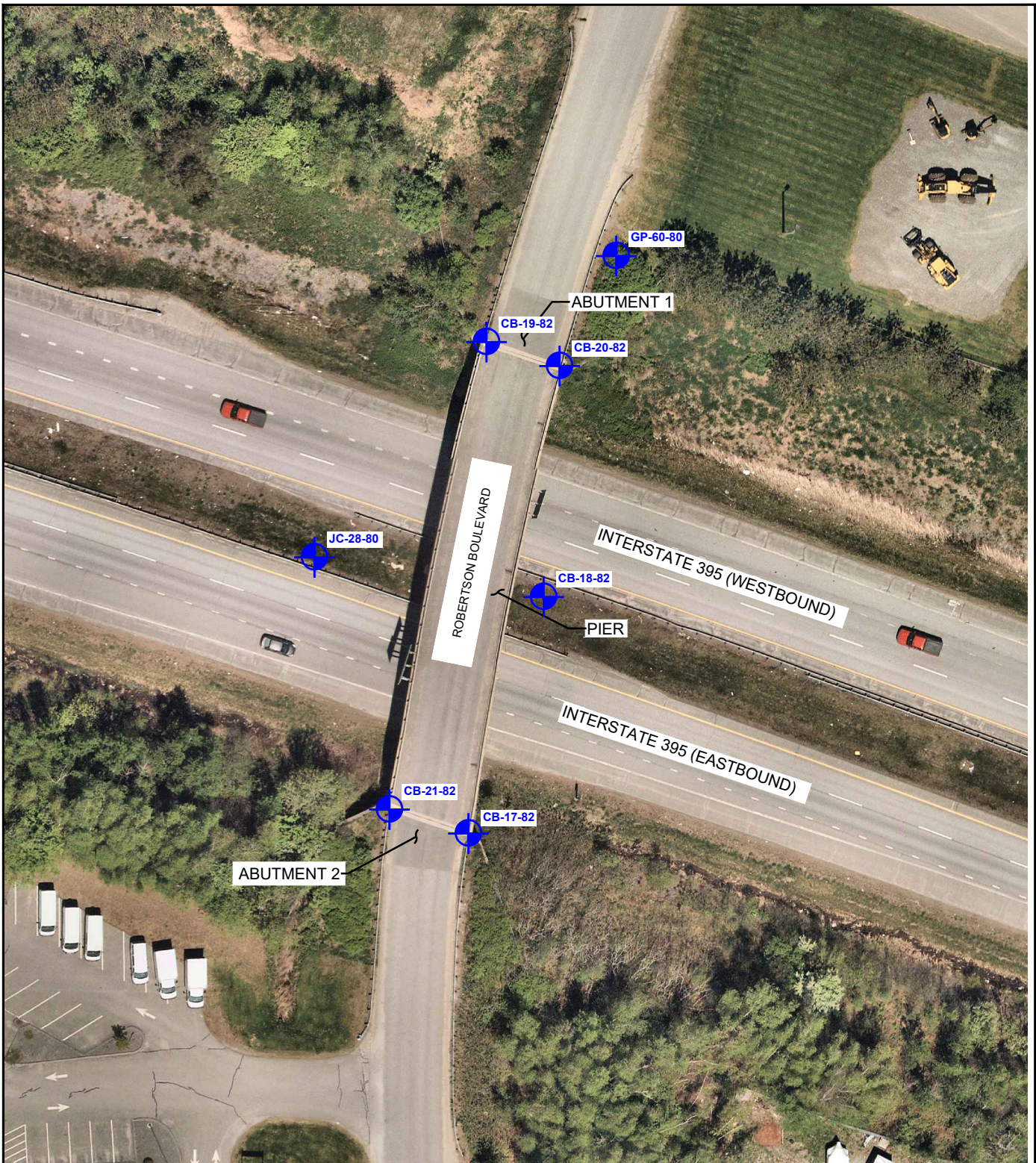
ROBERTSON BOULEVARD OVER INTERSTATE 395  
BRIDGE NO. 1560, MAINEDOT WIN 029484.00  
BREWER, MAINE

## PROJECT LOCUS

APPROXIMATE SCALE: 1 INCH = 2,000 FEET  
MAY 2026

**FIGURE 1**





#### LEGEND



CB-21-82  
 APPROXIMATE LOCATION OF  
 HISTORICAL BORING  
 BASED ON JUNE 1982 SOILS REPORT

#### NOTES

1. AERIAL IMAGE SHOWN IS DATED MAY 22, 2023 AND WAS DOWNLOADED FROM THE NEARMAP ONLINE DATABASE.



0 30 60  
 SCALE IN FEET

**HALEY  
 ALDRICH**

ROBERTSON BOULEVARD OVER INTERSTATE 395  
 BRIDGE NO. 1560, MAINEDOT WIN 029484.00  
 BANGOR, MAINE

## HISTORICAL BORING LOCATION PLAN

SCALE: AS SHOWN  
 MAY 2026

FIGURE 2

## **APPENDIX A**

### **Historical Soils Report**



BBFWEF 395 8 74

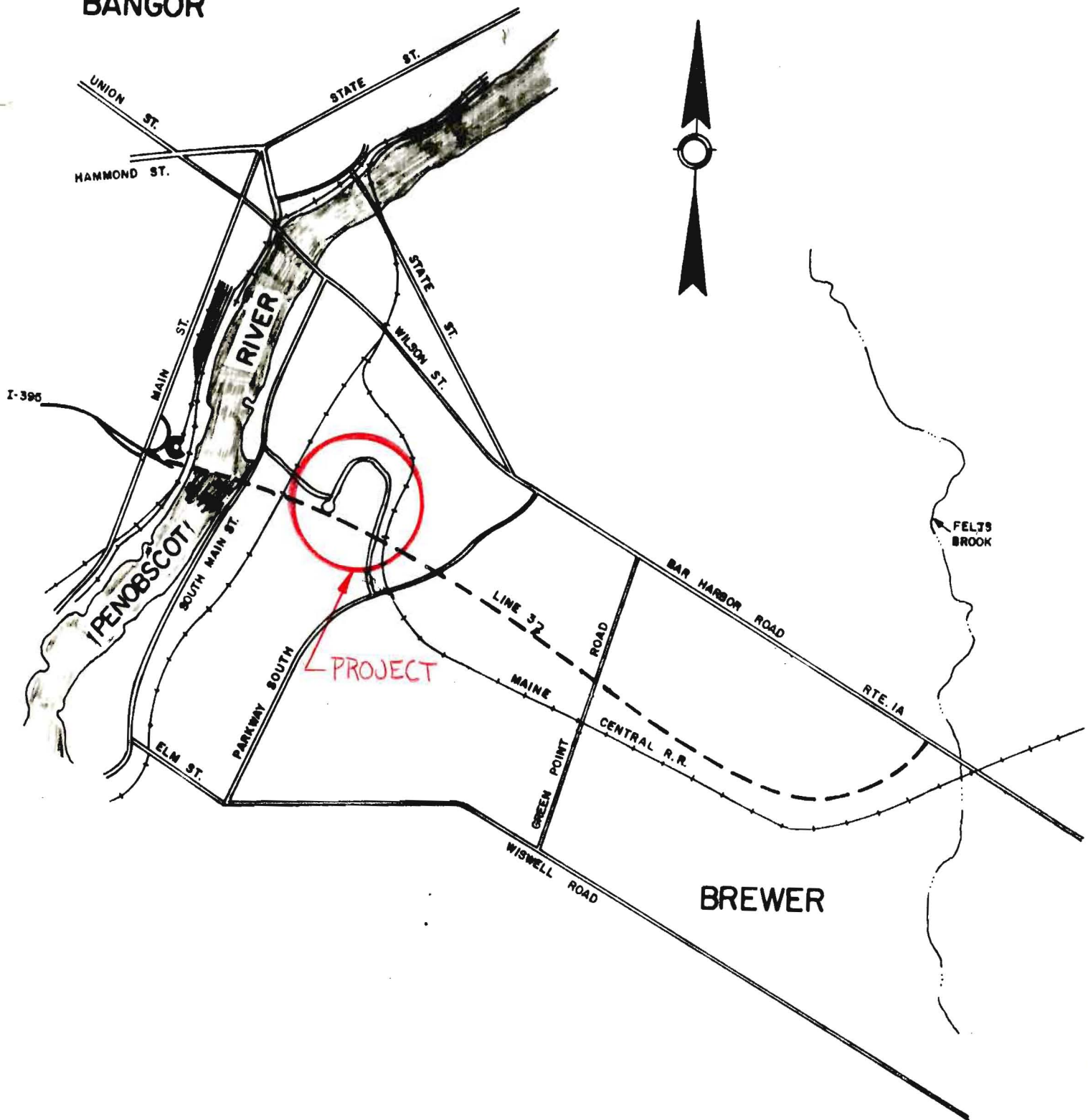
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Soils Report 82-27  
Brewer - Penobscot County  
Project 395-8(74)  
Industrial Park Rd. over I-395  
June 1982

Maine Department Of Transportation  
Materials and Research Division  
Soils Section

SUBSURFACE INVESTIGATION FOR THE PROPOSED  
CONSTRUCTION OF A STRUCTURE TO  
CARRY THE INDUSTRIAL PARK ROAD  
OVER INTERSTATE 395 IN THE CITY OF  
BREWER

# BANGOR



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## INTRODUCTION

A subsurface soils investigation has been completed for a proposed structure which will carry the relocated Industrial Park Road over I-395 in Brewer. Several washborings were made at or near the three proposed substructure locations in April and May, 1982, by a crew under the supervision of Mr. Chris Bark. Two additional washborings were made near this structure in 1980 as part of preliminary soils explorations for route alignment studies.

The locations and details of these borings are shown on the foundation survey and detail sheets. The masters of these sheets will be forwarded to the Bridge Design Section for inclusion in the final plans.

## GENERAL CONDITIONS

The proposed route of the relocated Industrial Park Road is approximately 700' to the west of the existing Industrial Park Road. The relocated Industrial Park Road centerline follows the alignment of an existing deadend paved road, continues beyond the present deadend and intersects Parkway South. The elevated topography is wooded to the south of the I-395 centerline and there are open fields on the north side of the I-395 centerline.

The soils explorations indicate that there is a significant thickness of soil overburden above the ledge surface at this site. The surficial deposit is a layer of brown and gray sandy clay. Along the existing paved road, this clay has been excavated and

replaced with brown silty sand and gravel fill. Underlying this surface deposit is 32' to 47' of stiff, sensitive, gray sandy silty clay over 5' to 21' of variable density gray silty gravelly sand and rocks. Ledge was encountered in all of the borings and core samples were described as calcareous metasilstone with a high angle of foliation.

#### SUBSTRUCTURE DETAILS

##### Abutment No. 1

The proposed centerline of bearing of this abutment intersects the construction centerline at Station 31+26.08, and is skewed  $7^{\circ}52'$  ahead measured from the long chord.

The approach fill to this abutment reaches a maximum height of 13 $\pm$  feet and this occurs over the existing left ditch.

Washboring CB-19-82 (Elev. 68.60) was made through the existing pavement at Station 31+25, 16' right of the Industrial Park Road construction centerline. The upper 9' of soil consists of a mixture of gray and brown silty clay and sand and gravel. From the depth of 9' to 42', stiff, sensitive gray sandy silty clay with many sand lines and lenses and black specks exists. Samples of this gray clay were obtained and consequently tested. At the 42' depth, a boulder was core drilled and this drilling continued to 63' where ledge was encountered and core drilled for 4'. Gray gravelly sand and rocks exists between depths 42' and 54' and the material changes to brown sand from 54' to 63'. Ledge was described as calcareous metasilstone with a high angle of foliation.



Shear strengths of the in situ clay were measured as between 0.62 and 0.87 TSF with the vane shear device. Lab vane strengths were typically lower. Natural water contents averaged 30%.

Washboring CB-20-82 (Elev. 66.91) was made 19' left of Station 31+27 in the existing ditch. There exists 7' of brown and gray silty clay underlain by 40'6" of stiff, sensitive gray silty clay with sand lines and lenses and black specks and bands. From depth 47'6" to 61'9" medium density gray sandy silt with gravel and rocks was encountered. Solid ledge was core drilled from 61'9" to 66'9" and described as calcareous metasiltstone with a high angle of foliation. Field vane shear strengths of the stiff gray clay varied from 0.99 to 0.46 TSF and natural water contents averaged 30%.

A preliminary washboring was made in the proposed approach embankment area in 1980. Washboring GP-60-80 (Elev. 67.04) is located at Station 30+75, 26' left of the Industrial Park Road construction centerline. There exists 18' of very stiff gray silty clay with a few sand or silt lines over 31' of stiff, sensitive gray silty clay with numerous small silt lines and black specks and bands. The lower few feet of this stratum contained more sand layers. From depths 49' to 55', medium density gray silty gravelly sand was found. Ledge was encountered at Elevation 12.0 and core drilled 5' and described as metasiltstone and phyllite with calcite

veins and a high angle of foliation.

Field vane shear strengths of the upper 18' stratum were 0.86 TSF and higher and the corresponding lab vane values were typically lower. The underlying clay stratum exhibited field vane strengths ranging from 0.81 TSF to 0.50 TSF and water contents averaged 30%. Consolidation tests were performed on two clay samples from this boring and the resultant pressure-void ratio diagrams are shown on Sheets 1 and 2. Calculations indicate that this stiff clay is in an overconsolidated state.

Washboring details for these three borings are shown on Sheet 6 and a transverse section across Abutment No. 1 illustrates the subsurface stratification on Sheet 9.

#### Pier

The proposed centerline of bearing of this pier is coincidental with the I-395 mainline centerline and intersects the Industrial Park Road construction centerline at Station 32+23.62. This centerline of bearing is skewed ahead 7°52' measured from the long chord.

One washboring was completed within the pier footing location. Washboring CB-18-82 (Elev. 68.03) was made through the existing pavement at Station 197+30, 3' left of the I-395 centerline. Six feet of brown sand and gravel fill overlies 16' of very stiff gray sandy silty clay with sand lenses and layers. From the depth of 22' to 43', there exists stiff, sensitive gray sandy silty clay with black bands and sand lines. The following 20' consisted of drilling through variable densities of gray gravelly sand and rocks. Ledge

was encountered at Elevation 5.0 and core drilled 4' and described by the geologist as calcareous metasiltstone and metagraywacke that exhibited a high angle of foliation.

Field vane shear strengths in the upper clay layer were greater than 0.83 TSF and in the lower clay layer they ranged from 0.50 to 0.87 TSF. Natural water contents averaged 29%.

Another washboring was made for preliminary soils studies in 1980 in the vicinity of the proposed pier location. Washboring JC-28-80 (Elev. 67.45) was made 9' right of Station 196+58 on the I-395 centerline. Below the existing pavement there exists 4'4" of brown silty sand and gravel fill and then a thin 2'8" layer of medium consistency gray sandy silty clay. From the depth of 7'4" to 43'4", there exists stiff, sensitive, gray sandy silty clay with sand lines and black specks. Underlying this clay is 5'4" of dense gray gravelly sandy clayey silt. Ledge was encountered at Elevation 18.8 and core drilled 5' and described as tan and light gray meta-siltstone fragments over phyllite.

Shear strengths of the stiff clay as measured using the field vane device ranged from 0.75 to over 1.0 TSF. Unconfined compressive tests ranged in value from 0.49 to 0.66 TSF for the same material. Natural water contents averaged 28%, plastic limits averaged 18% and the corresponding liquid limits ranged from 24% to 31%.

Details of these two washborings are shown on Sheet 7 and a transverse section at the pier is shown on Sheet 9.

#### Abutment No. 2

The centerline of bearing of this abutment intersects the

construction centerline at Station 33+22.32. This centerline of bearing is also skewed ahead 7°52' measured from the long chord. The approach fill to this abutment reaches a maximum height of 9'±.

Washboring CB-17-82 (Elev. 74.10) was made on top of the existing slope at Station 33+26, 17' left of the construction centerline. There exists 3½' of surficial brown silty sandy clay topsoil over 46½' of stiff, sensitive, gray sandy silty clay with sand and silt lenses and black specks. From the depth of 50' to 60'6", there exists loose gray gravelly sand and at Elevation 13.6 the ledge surface was encountered and core drilled 5'. This ledge core was described as calcareous metasiltstone and metagraywacke with a high angle of foliation.

Natural water contents of the stiff clay samples averaged 30%. Field vane shear strengths decreased from over 1.0 TSF to 0.53 TSF with depth in this clay.

Two consolidation tests were performed on samples of the stiff gray clay and it is apparent that this clay stratum is in an over-consolidated state. The two pressure-void ratio diagrams are shown on Sheets 3 and 4.

The second washboring for this abutment was made at Station 33+20, 16' right of the construction centerline. Washboring CB-21-82 (Elev. 70.38) encountered 38' of stiff, sensitive gray sandy silty clay with many sand lenses and black specks underlying 7' of brown and gray sandy clay. From the depth of 45' to 58'6",

medium density gray gravelly sand and rocks exists and at Elevation 11.9, ledge, described as metasilstone with some phyllite zones and a high angle of foliation was core drilled 5'.

Natural water contents of the gray silty clay averaged 28% and field vane shear strengths ranged from 0.84 TSF to more than 1.0 TSF.

The details of these two washborings are shown on Sheet 8. Also, a transverse section depicting the soil stratification at this proposed Abutment location is shown on Sheet 9.

#### DESIGN CONSIDERATIONS

It is recommended to support the 3 substructure units on steel end-bearing H-piles driven to the ledge surface or practical refusal. At Abutment No. 1, the ledge surface is essentially flat at Elevation 5.3 $\pm$  and at Abutment No. 2, the ledge surface dips slightly from Elevation 13.6 $\pm$  on the left of centerline to Elevation 11.9 $\pm$  on the right of centerline. At the pier location the ledge surface appears to rise from Elevation 3.0 $\pm$  on the left to Elevation 10 $\pm$  on the right.

#### Settlement

The approach fill to Abutment No. 1 will basically follow the alignment of the existing paved road and reach a maximum height of 13' $\pm$ . A settlement analysis was performed using the consolidation data from the stiff clay samples from Washboring GP-60-80. It was determined that this deposit of stiff silty clay is in an over-

consolidated state due to past stresses that exceeded the existing "in situ" stresses. With the addition of the fill material, the increase in stress within the clay layer does not reach the past maximum stress level and thus the change in void ratio is relatively small. Estimated maximum settlement of a 11' fill height along centerline is 3"<sub>±</sub>. An average of 95% of the total primary consolidation is expected to occur within 3 years.

The approach embankment to Abutment No. 2 is on new alignment and the maximum fill height is 8½'<sub>±</sub>. Two samples from Washboring GB-17-82 were tested and two resultant pressure-void ratio curves were developed and indicate that the clay is overconsolidated. The future stress increase caused by the fill addition again results in a small change in void ratio. Settlement estimates indicate 1.5"<sub>±</sub> of consolidation will occur and an average of 95% of this consolidation should occur within 2-3 years after fill placement.

### Stability

The embankment approaches to the bridge combined with the excavation required for the I-395 lanes were believed to be potential stability problems. However, analyses were conducted to study the possibilities of a shear failure along a longitudinal section as well as across a transverse section and it was found that the proposed embankment conditions should be stable as designed.

In the longitudinal analysis along the Industrial Park Road centerline with the approach fill at the finish grade and the I-395



roadway excavated to subgrade elevation the resultant safety factor against shearing is 1.94 and the failure arc occurs deep in the stiff silty clay layer. Another analysis was performed which approximates the final appearance longitudinally and includes the I-395 subbase materials. As expected, the safety factor increased; thus, it appears that stability along the roadway centerline is adequate. A diagram of this analysis is shown on Sheet 5.

Transverse stability of the Industrial Park Road embankment was analyzed and the safety factor is very high. This is due to the high shear strength of the subsurface clay layer and the existing high backslopes.

#### SUMMARY

A subsurface soils investigation has been completed for a new bridge structure that will carry the relocated Industrial Park Road over the proposed I-395 lanes. Washborings were made at each of the substructure locations and their individual details are shown on Sheets 6, 7 and 8. The plan, profile, and transverse sections are shown on Sheet 9. Generally there exists a thin surface layer of brown sandy silty clay (or brown silty sand and gravel along the existing roadway) above a thick stratum of stiff and sensitive gray sandy silty clay. This clay layer is very stiff in the upper portions and becomes less stiff with depth. Underlying this compressible clay stratum is a layer of variable density gray gravelly


sand and rocks. Ledge was encountered and core drilled in all of the borings and ranges in Elevation from 5.0+ and 18.8+. It was described as calcareous metagraywacke with a high angle of foliation.

Settlement of the approach fills to each abutment is anticipated. Due to the relatively small fill heights and the resulting low stress increases in the thick clay stratum and the fact that this stiff clay deposit is overconsolidated, settlement ranging from 1.5"+ to 3"+ is expected. An average of 95% of this total settlement is theoretically expected to occur within 3 years.

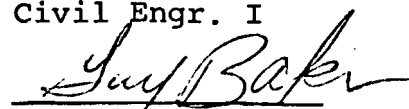
Stability analyses of the approach embankments were undertaken in both the transverse and longitudinal directions. According to these analyses, stability does not appear to be of significant concern either during or after construction. Computed safety factors were well above the desired minimum values.

It is recommended to support the three substructure units on steel end-bearing H-piles driven to the ledge surface or practical refusal. Across Abutment No. 1, the ledge surface appears to be flat at Elevation 5.3+ and across Abutment No. 2, the ledge surface dips slightly from left to right from Elevation 13.6+ to Elevation 11.9+. At the pier location, the ledge appears to rise from left to right from Elevation 3.0+ to Elevation 10+.

Prepared By

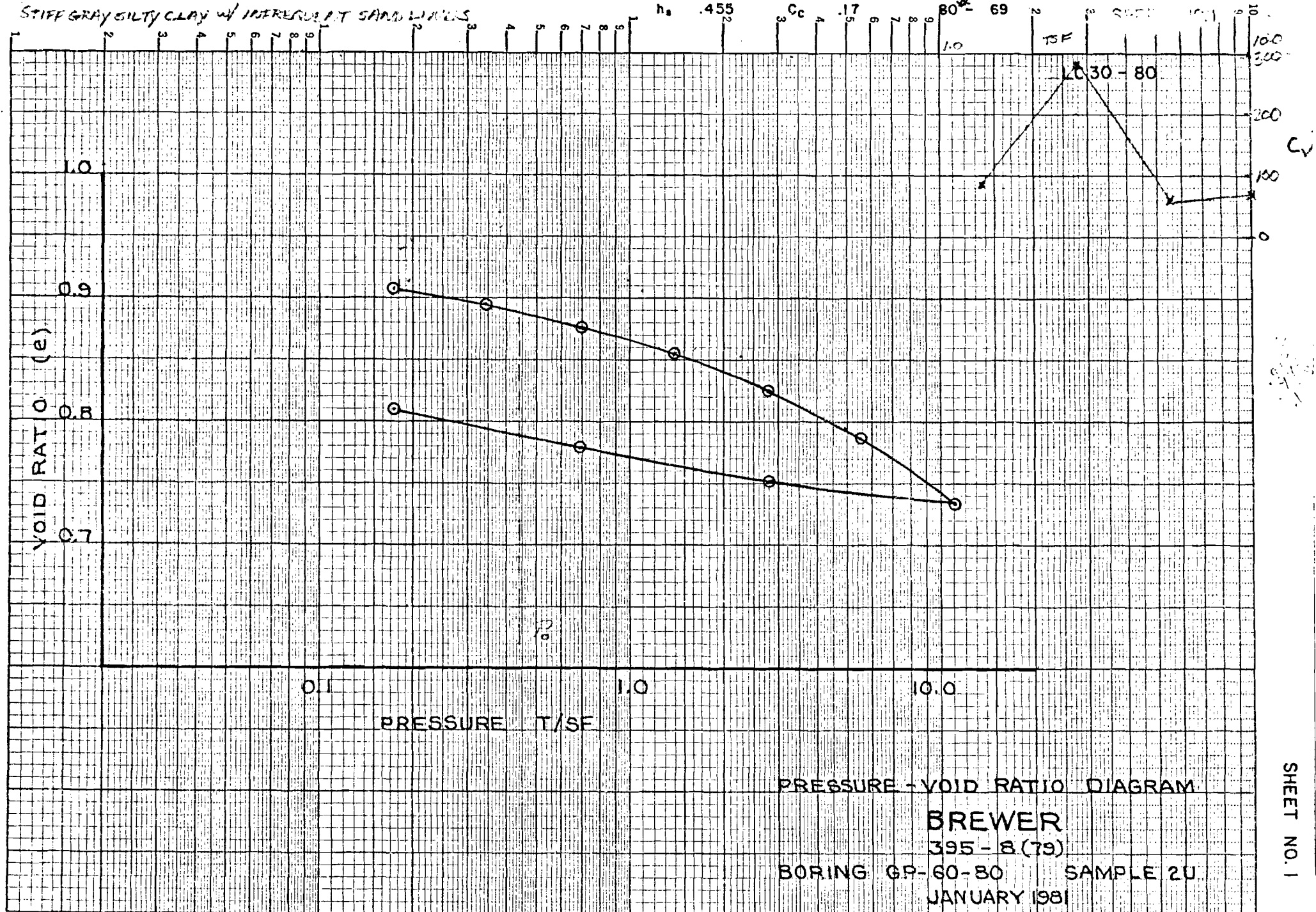
  
Peter Coughlan  
Civil Engr. I

Approved By

  
Guy Baker  
Asst. Soils Engr.

VANE .720, .816 PMIN 1.1 Cv 10% - 92  
WC's 33 - 29 PMAX 4.9 20% - 279  
G 2.82 e .85 40% - 61  
h<sub>s</sub> .455 Cc .17 80% - 69

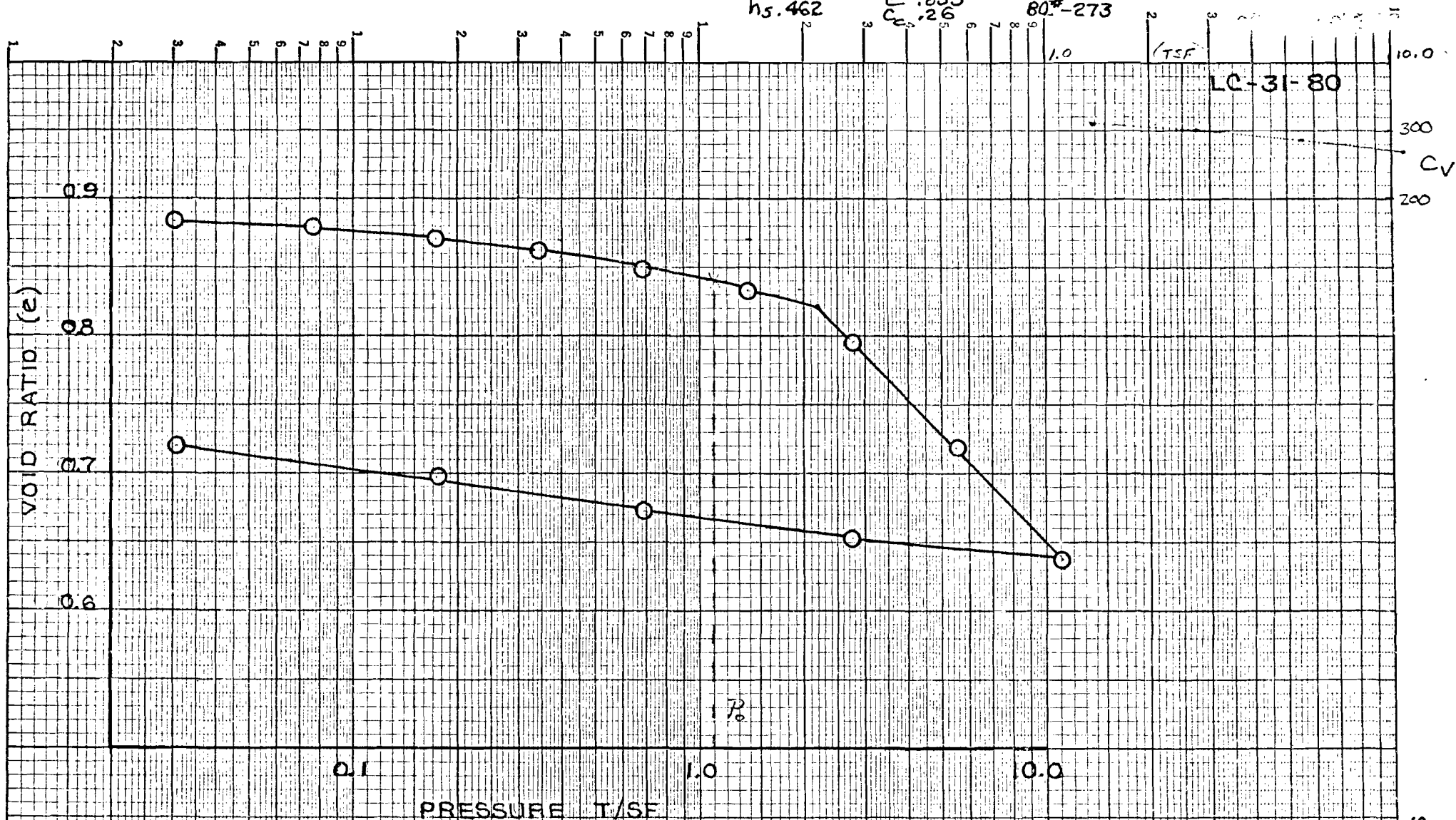
STIFF GRAY SILTY CLAY w/ INTERMITTENT SAND LENSES



VANE 384-432  
WC's 33-28  
G 2.74  
h<sub>s</sub> 462

P<sub>min</sub> 1.4  
P<sub>max</sub> 2.2  
P<sub>p</sub> 1.95  
e .833  
C<sub>c</sub> .26

C<sub>v</sub> 10# - 308  
20# - 301  
40# - 285  
80# - 273



PRESSURE-VOID RATIO DIAGRAM

BREWER

395 - 8 (79)

BORING GP-60-80 SAMPLE 60

JANUARY 1981

VANE -672  
WC'S 30-26  
G 2.78  
h<sub>s</sub> .4819

P<sub>MIN</sub> 1.3  
P<sub>MAX</sub> 4.5  
P<sub>p</sub> 2.0  
e .77  
C<sub>c</sub> .17

CV10<sup>3</sup>-78  
20<sup>3</sup>-105  
40 -105  
80 -114

Stiff sens. gray silty clay w/numerous  
black specks

LC-13-82

VOID RATIO (e)

PRESSURE T/SF

PRESSURE-VOID RATIO DIAGRAM

BREWER

395-8(79)

BORING CB-17-82

SAMPLE 3U

MAY 1982

SHEET NO. 3

VANE .450  
WCS 30-25  
G 2.75  
h<sub>s</sub> .4815

P<sub>min</sub> 1.35  
P<sub>max</sub> 4.8  
P<sub>p</sub> 2.15  
e .770  
c<sub>c</sub> .22

CV 10<sup>#</sup> - 313  
20 - 386  
40 - 272  
80 - 254

Med cons. sens. sandy silty clay w/numerous small  
sand & silt lenses / 6 tack specks

LC-14-82

VOID RATIO (e)

PRESSURE T/SF

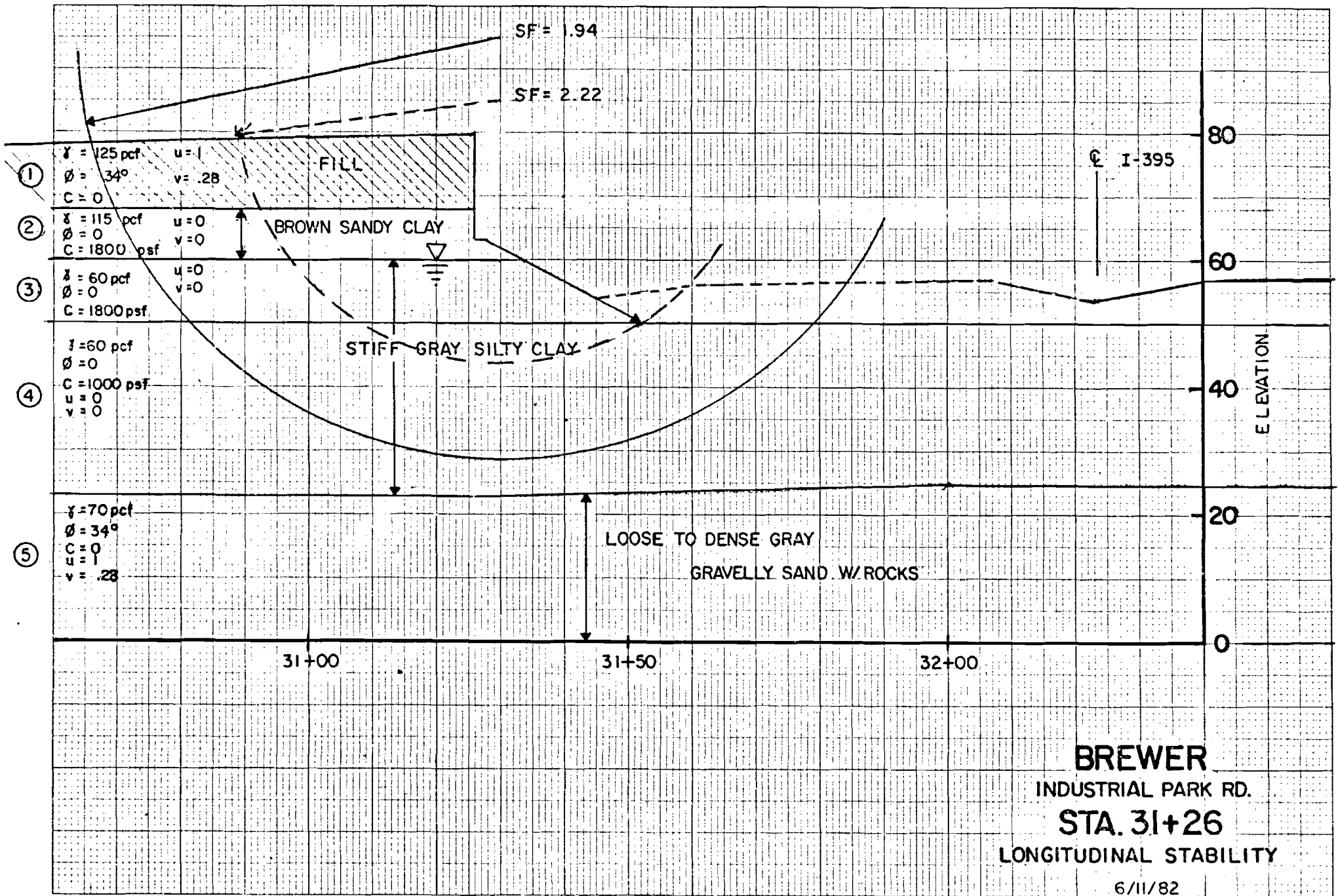
PRESSURE-VOID RATIO DIAGRAM

BREWER  
395-8(79)

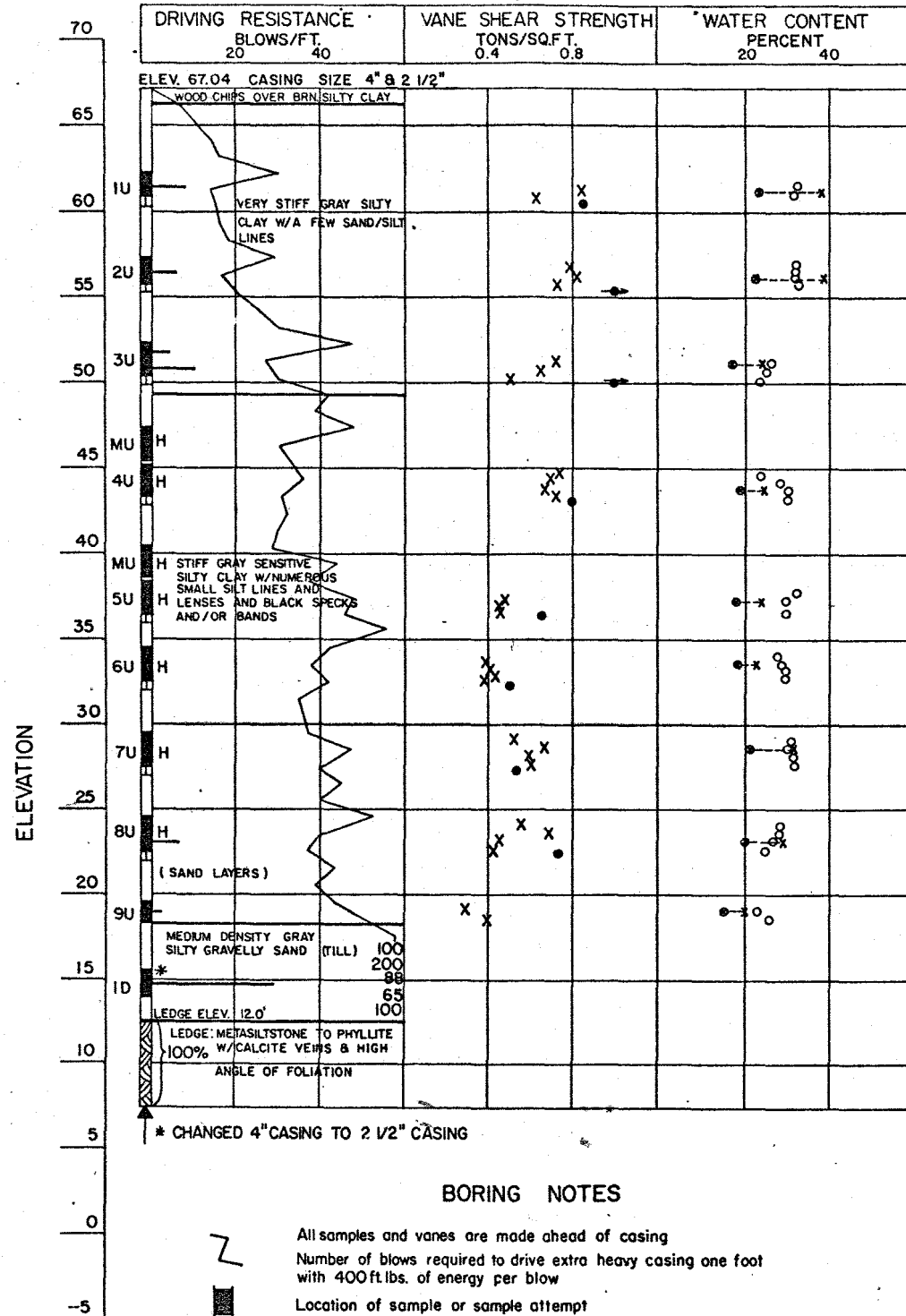
BORING CB-17-82 SAMPLE GU  
MAY 1982

SHEET NO. 4





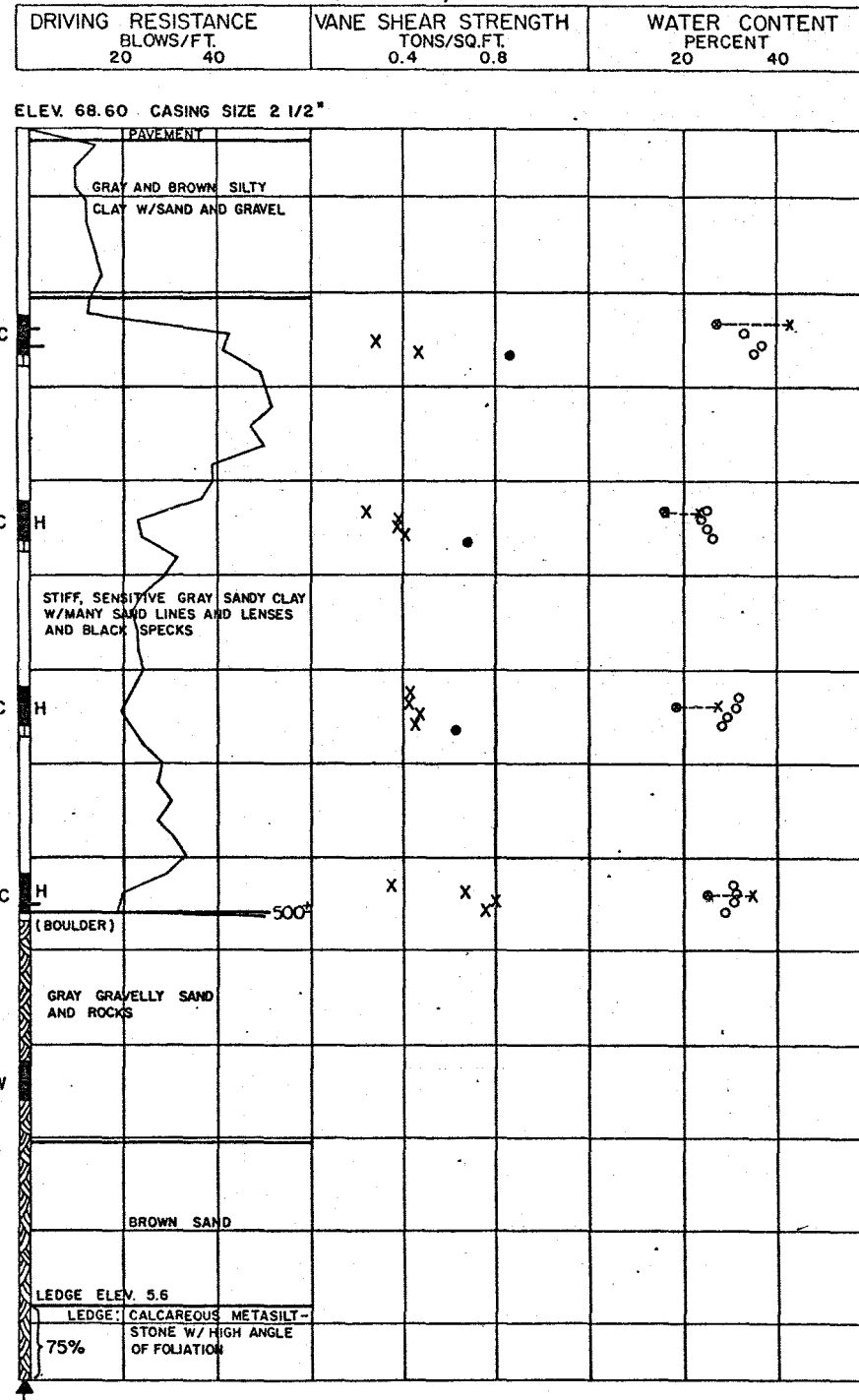
# BORING GP-60-80 STA. 30+75, 26' LT.



## BORING NOTES

- All samples and vane are made ahead of casing
- Number of blows required to drive extra heavy casing one foot with 400 ft. lbs. of energy per blow
- Location of sample or sample attempt
- Number and type of dry sample
- ID S&H Sampler #1290's
- IC 2" O.D. 16 ga. seamless tubing
- IU 3 1/2" O.D. 16 ga. seamless tubing
- IW Wash sample and number
- MD Unsuccessful sample attempt and type of sampler
- Number of blows required to drive spoon or tubing one foot with 350 ft. lbs. of energy per blow
- H Sampling spoon or seamless tubing driven by static weight of drill rods and hammer
- Field vane test
- Bottom of boring (may not be bottom of soil strata)
- 71% Locations cored by diamond bit and percent recovery of rock

# BORING CB-19-82 STA. 31+25, 16' RT.



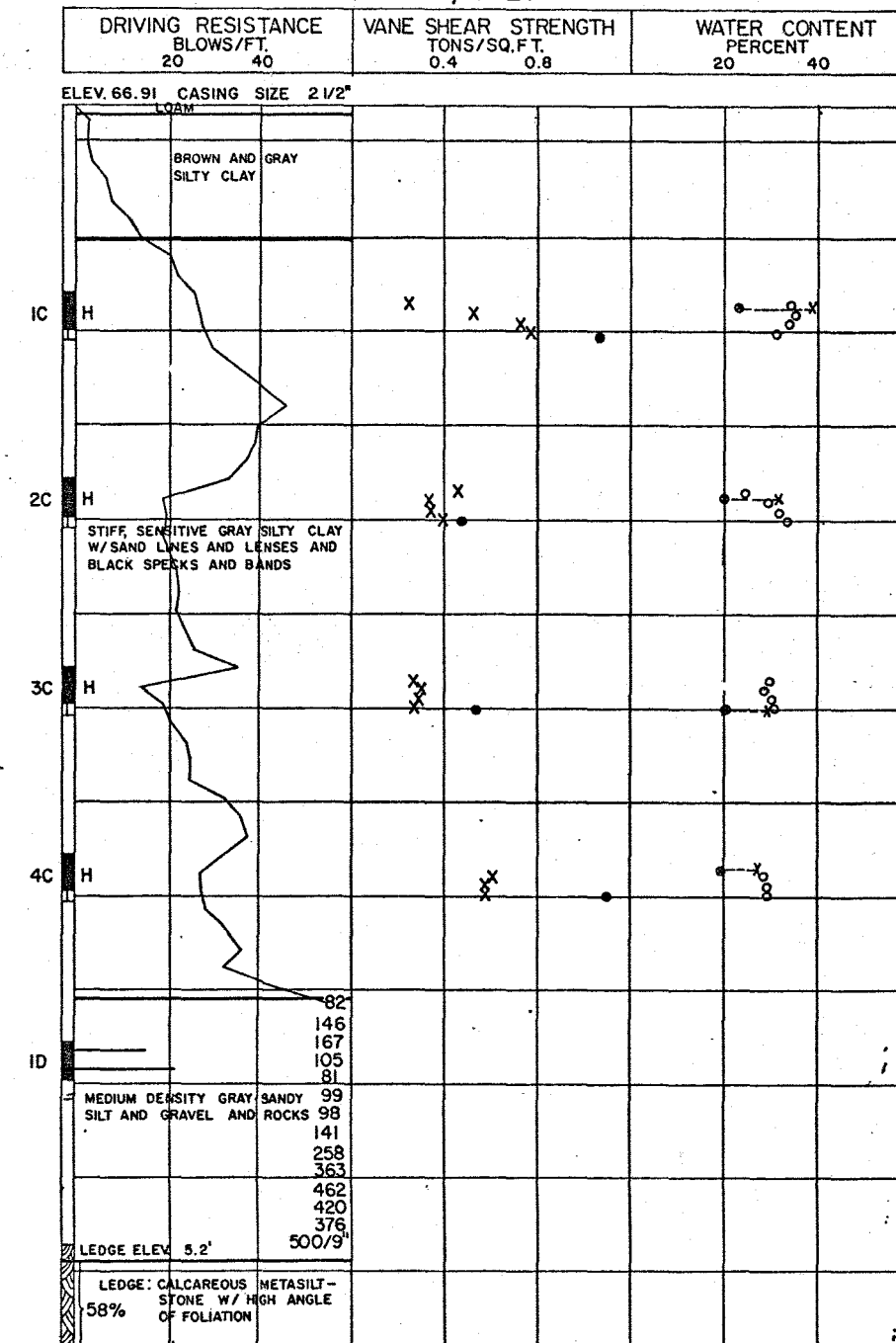
## SHEAR NOTES

- Field vane shear strengths
- Laboratory vane shear strengths
- Shear strengths in excess of capacity of equipment
- One half unconfined compressive strengths

## WATER CONTENT NOTES

- Natural water contents given as percent of dry weight
- Plastic and liquid limits
- Ignition losses are given as percent of dry weight

# BORING CB-20-82 STA. 31+27, 19' LT.



STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

INDUSTRIAL PARK ROAD  
OVER  
I-395  
IN THE TOWN OF  
BREWER  
PENOBSCOT COUNTY

BORING DETAILS

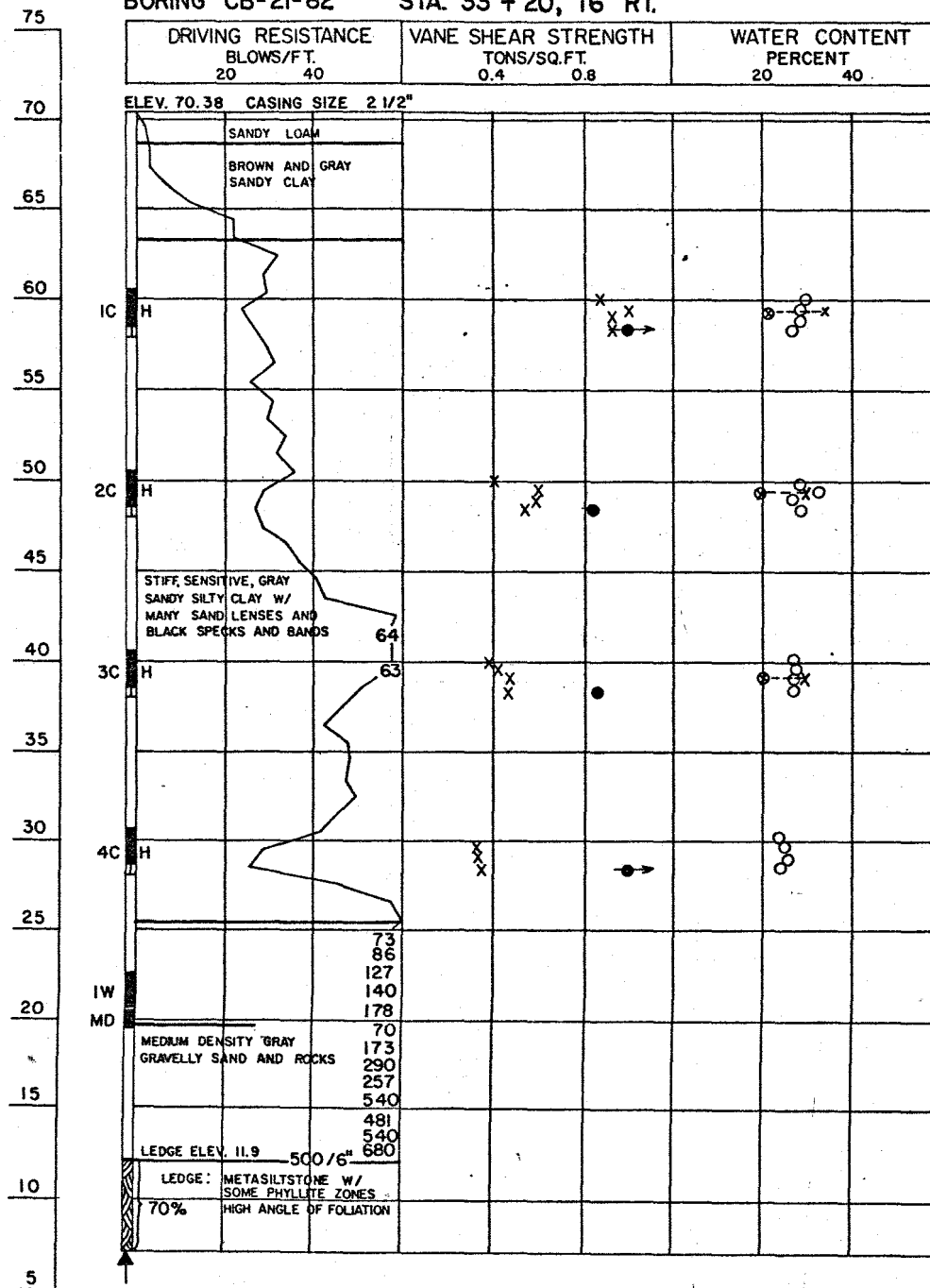
SHEET OF AUGUSTA, MAINE

PROJECT DESIGN ENGINEER  
BY  
DATE  
DESIGN - DETAILED  
CHECKED  
REVISIONS  
FIELD CHANGES  
PLANS

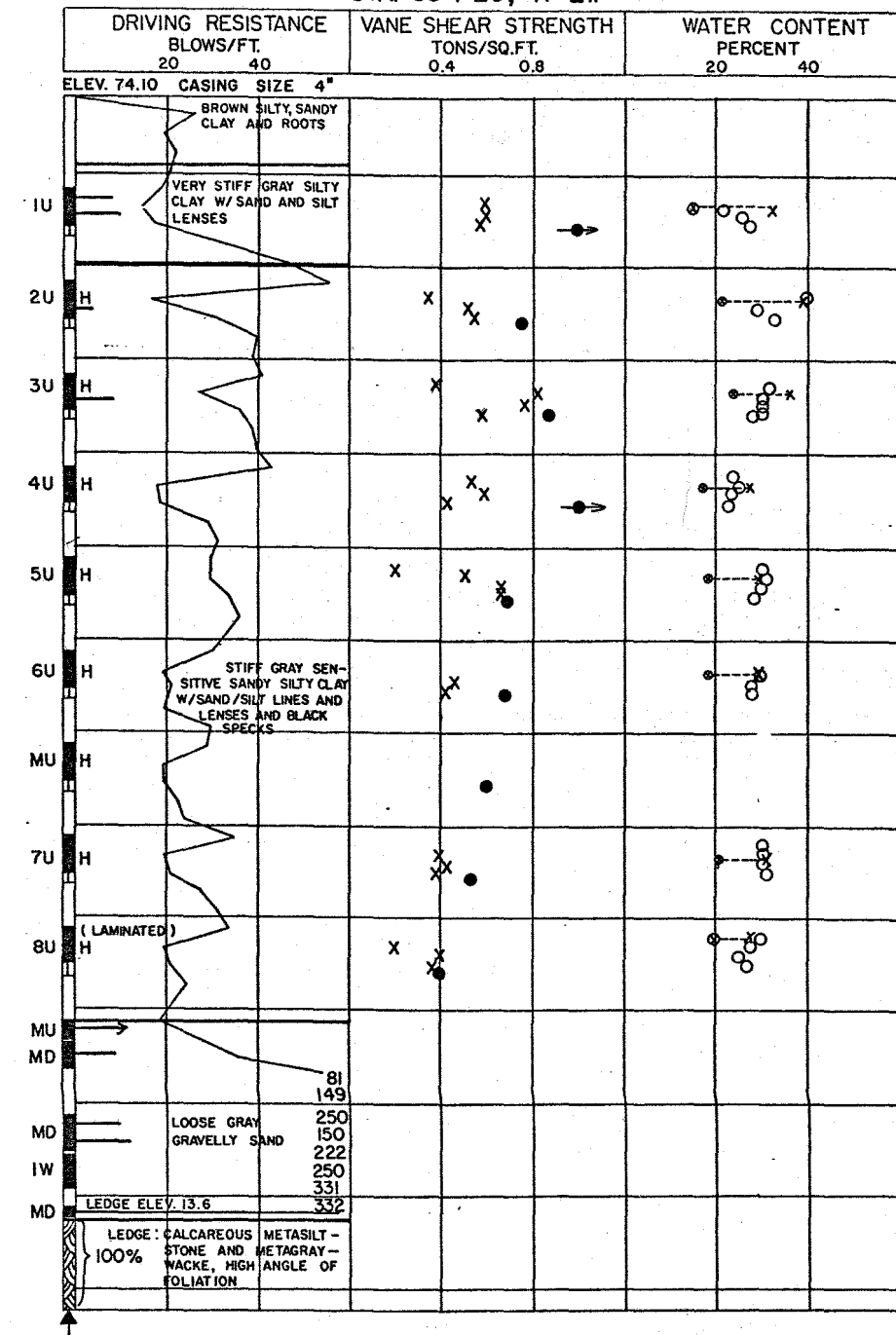
BORING 44-132-45710



BORING CB-21-82 STA. 33 + 20, 16' RT.

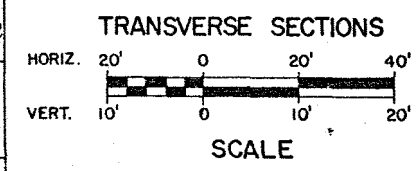
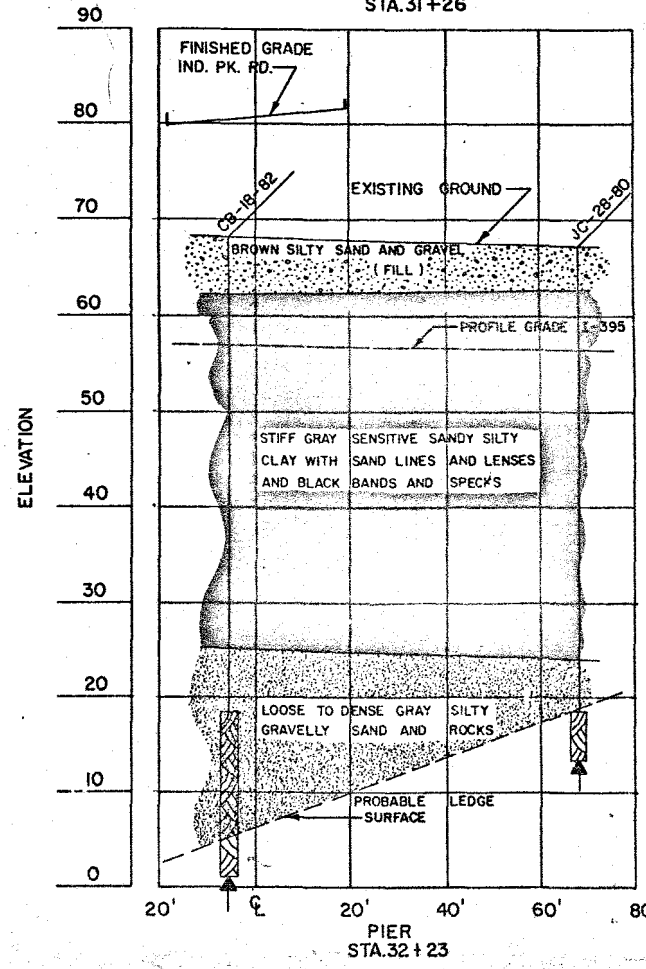
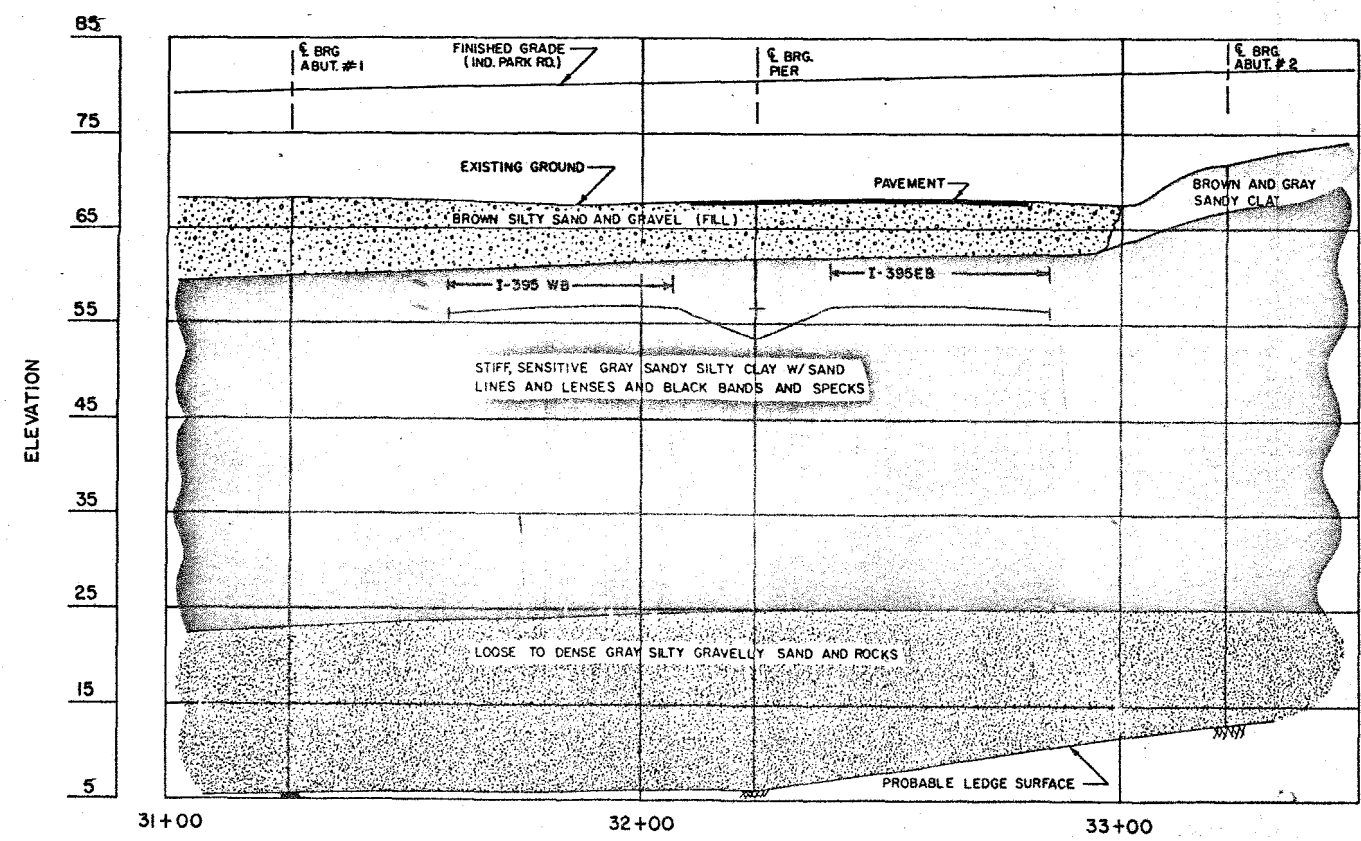
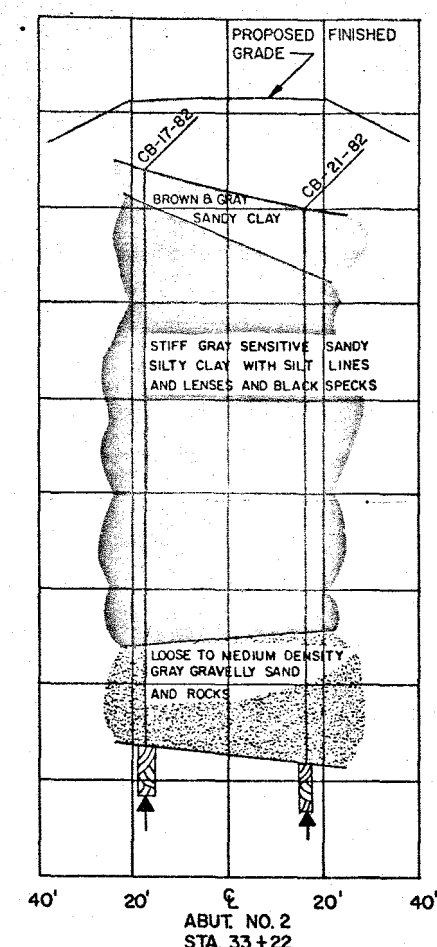
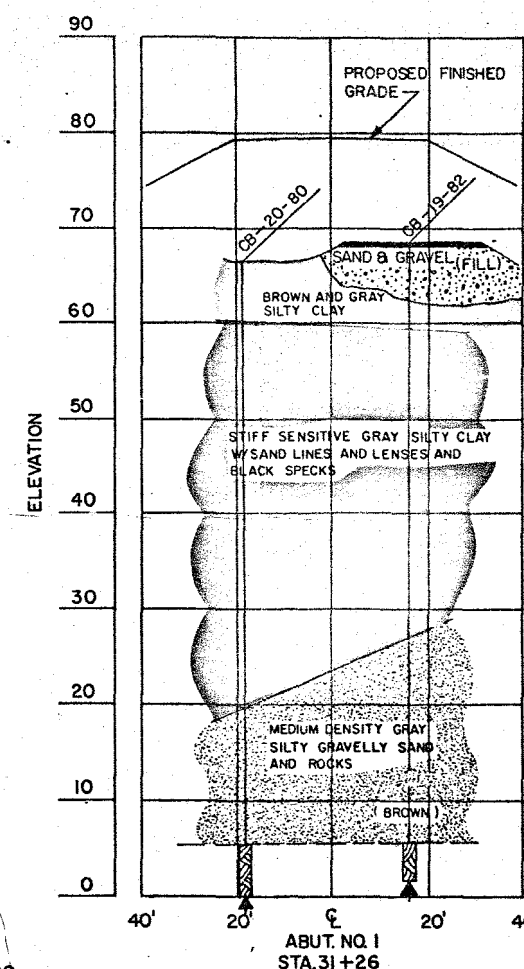
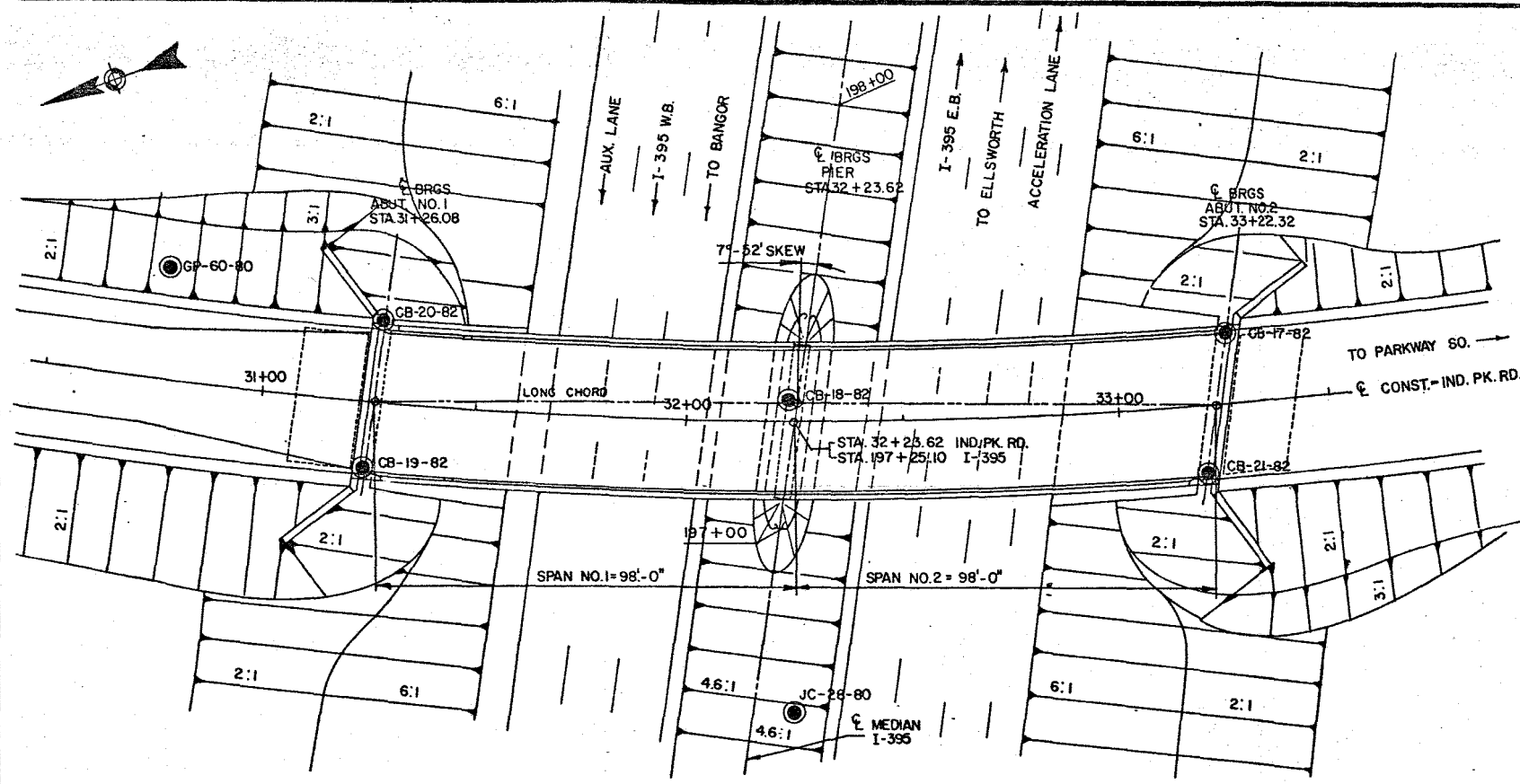


BORING CB-17-82 STA. 33 + 26, 17' LT.



PROJECT DESIGN ENGINEER  
 BY DATE  
 DESIGN - DETAILED  
 CHECKED  
 REVISIONS  
 FIELD CHANGES  
**PLANS**

BORING 44-132-45710



PROJECT DESIGN ENGINEER	BY	DATE
DESIGN - DETAILED		
CHECKED -		
REVISIONS		
FIELD CHANGES		

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

INDUSTRIAL PARK ROAD  
OVER  
I-395  
IN THE TOWN OF  
BREWER  
PENOBSCOT COUNTY  
FOUNDATION SURVEY

SHEET OF AUGUSTA, MAINE