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**MISSISSIPPI EBL & WBL RIVER BRIDGES
HIGHWAY 417, ARNPRIOR, ONTARIO
GEOTECHNICAL INVESTIGATION REPORT**

W.P. 451-90-03/04, SITE 3-594
HIGHWAY 417, DISTRICT 42, OTTAWA
GEOCRES No. 31F-117

Report

To

Ministry of Transportation, Ontario
Foundation Design Section

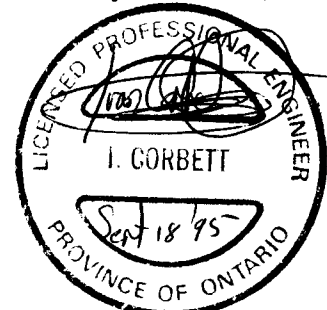
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September 18, 1995
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TABLE OF CONTENTS

1.0	INTRODUCTION	1
2.0	SITE DESCRIPTION	1
3.0	SITE GEOLOGY	3
4.0	INVESTIGATION PROCEDURE	4
4.1	Field Program	4
4.2	Laboratory Testing	6
5.0	SUBSURFACE CONDITIONS	7
5.1	General	7
5.2	Stratigraphic Units	8
5.2.1	Topsoil	8
5.2.2	Floodplain Deposits	8
5.2.3	Silty Clay	9
5.2.3.1	Overview	9
5.2.3.2	Silty Clay to Clay	10
5.2.3.3	Silty Clay	13
5.2.4	Glacial Till	15
5.2.5	Bedrock	15
5.3	Groundwater	16
6.0	ENGINEERING DISCUSSIONS AND RECOMMENDATIONS	18
6.1	General	18
6.2	Approach Embankment Design	20
6.2.1	Overview	20
6.2.2	Stability Analysis	20
6.2.2.1	Design Parameters	20
6.2.2.2	Short Term Stability	21
6.2.2.3	Long Term Stability	22
6.2.2.4	Discussion	22
6.2.3	Settlement Analysis	23
6.2.3.1	Design Parameters	23
6.2.3.2	Magnitude of Primary Settlement	24
6.2.3.3	Rate of Settlement	26
6.2.3.4	Secondary Consolidation	27
6.2.3.5	Discussion	27
6.2.4	Abutment and Road Approach Fill Construction	28
6.2.4.1	Abutment Fills	28
6.2.4.2	Road Approach Fills	30
6.3	Foundation Design Recommendations	30
6.3.1	Overview	30
6.3.2	End Bearing Steel H Piles	31

6.3.2.1	End Bearing Capacity	31
6.3.2.2	Negative Skin Friction	31
6.3.3	Installation	32
6.3.4	Lateral Pile Resistance	33
6.3.5	Abutment Backfill	35
6.4	General Construction Recommendations	36
6.4.1	Concrete Sulphate Requirements	36
6.4.2	Temporary Excavations	36
6.4.3	Road Approach Fill Construction	37
6.4.4	Construction Inspection and Monitoring	37
6.4.5	Frost Depth	38
6.4.6	Floodplain Access Road Construction	38
6.4.7	Seismic Design	38
6.5	Environmental Considerations	39
7.0	CONCLUSIONS AND RECOMMENDATIONS	40
8.0	REFERENCES	42

LIST OF TABLES

Table 1	Borehole Summary
Table 2	Borehole Stratigraphic Summary
Table 3	Summary of Water Level Readings
Table 4	Summary of Sulphate Content Determinations
Table 5	Summary of Consolidation Test Data

LIST OF FIGURES

Figure 1	Site Location Plan
Figure 2	Borehole Location Plan
Figure 3	Summary Plot of Engineering Properties of Silty Clay Layer - West Approach Area
Figure 4	Summary Plot of Engineering Properties of Silty Clay Layer - East Approach Area
Figure 5	Summary Plot of Engineering Properties of Silty Clay Layer - Central Floodplain Area
Figure 6	Summarized Design Stratigraphy - East Approach Area
Figure 7	Summarized Design Stratigraphy - West Approach Area
Figure 8	Design Shear Strength Envelopes - East Approach Area
Figure 9	Design Shear Strength Envelopes - West Approach Area
Figure 10	Pre-Consolidation Design Profiles
Figure 11	Summarized Stability Calculations - East Approach Area
Figure 12	Summarized Stability Calculations - West Approach Area
Figure 13	Summarized Settlement Calculations - East Approach Embankment
Figure 14	Summarized Settlement Calculations - East Abutment Embankment
Figure 15	Summarized Settlement Calculations - West Approach Embankment
Figure 16	Summarized Settlement Calculations - West Abutment Embankment
Figure 17	Abutment Embankment Settlement Versus Fill Contact Pressure
Figure 18	Approach Embankment Settlement Versus Fill Contact Pressure
Figure 19	Rate of Settlement Analysis - Approach Embankments
Figure 20	Rate of Settlement Analysis - Abutment Embankment Fills
Figure 21	Rate of Settlement Analysis - Abutment Embankment Fills

LIST OF DRAWINGS

Drawing No. 4519003/04-A	Borehole Locations and Soil Strata-WBL
Drawing No. 4519003/04-B	Borehole Locations and Soil Strata-EBL

LIST OF APPENDICES

Appendix A	Borehole Logs
Appendix B	Grain Size Analyses
Appendix C	Atterberg Limits Determinations
Appendix D	Consolidation Test Results
Appendix E	Selected Stability Analyses
Appendix F	Settlement Analyses
Appendix G	Factual Report on Consolidation Test Results as submitted by Golder Associates Ltd., Mississauga, Ontario, dated May 1995

FOUNDATION INVESTIGATION REPORT

for

Mississippi River Bridges (EBL/WBL)

WP 451-90-03/04, Site 3-594

Hwy 417, District 42, Ottawa

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out by Thurber Engineering Ltd. (TEL) on behalf of the Ministry of Transportation of Ontario (MTO), Foundation Design Section, at the site of the proposed Highway 417 twin bridges over the Mississippi River, near Arnprior, Ontario. The purpose of this investigation was to establish the subsurface conditions at the site and, based on that information, provide pertinent geotechnical recommendations for the design and construction of the proposed bridge structures and associated approach embankments. The contents of this report apply to the design of the Westbound Lanes between Station 17+800 and 18+225 and between Station 17+775 and 18+200 of the Eastbound Lanes.

The work presented within was completed in general accordance with our proposal letter to the MTO dated February 23, 1995, and under MTO Consultants Agreement No. 9490-4444-9713 dated June 12, 1995.

2.0 SITE DESCRIPTION

The site of the proposed works is located in the Regional Municipality of Ottawa-Carleton, Ontario in the Township of West Carleton, approximately 10 km south of the Town of Arnprior (see Figure 1). More specifically, the proposed new bridges will be located approximately midway between the existing Highway 17 bridge over the Mississippi River and an existing bridge over the same river along Upper Dwyer Road some 1350 m upstream of the Highway 17 bridge.

The existing Highway 17 bridge structure is a through truss structure with a span of 54.3 m and two low truss approach structures each with spans of 23.7 m built in 1954. Review of the available construction drawings for this structure indicates that both abutments and the two central piers are supported on 15.3 m long (50 feet) timber piles. Available borehole data indicates the presence of soft silty clay to at least 23 m (75 feet) at each foundation element. Review of MTO inspection records from 1982 indicate that the bridge has generally performed well, although there was

some concerns expressed over closing of expansion joints at the east pier and west abutment. These may have been a result of bridge settlement although there are no reported settlement problems with the bridge.

The existing Dwyer Hill Road Bridge, located along the 3rd line, Lot 15 between Concessions 2 and 3, Township of West Carleton, consists of a three span, two lane, continuous concrete structure with a 32 m long centre and two 23 m long exterior spans. The bridge was constructed in 1983 to replace an existing smaller structure. It is understood that both abutments and central piers are supported on 273 mm diameter, 9.3 mm thick steel tube piles filled with concrete and end-bearing on bedrock. Field drilling results indicate the presence of deep deposits of silty clay overlying bedrock at approximate elevation 47 m. Approach embankments to the initial structure at this location were in the order of 5 m high and 5 m wide and were raised by about 1 m and widened to 10 m as part of the bridge construction in 1983. Bridge inspection records from 1987 indicates that the bridge has performed well and was in good condition.

The site of the proposed bridges is dominated by the north flowing Mississippi River and associated adjacent flood plain which is some 5 to 8 m below the surrounding flat tablelands. The total width of the river valley is some 250 m which consists of an 80 m wide river channel adjacent to the toe of the west valley slope and a 170 m wide poorly drained floodplain area to the east of the main channel. Available information provided by the MTO, indicates that the river channel is in the order of 7 to 8 m deep, or the channel base is some 15 m below the elevation of the adjacent west tablelands. The river water elevation was reported to be 83.0 m in October 1992, although this level is anticipated to vary seasonally, and also as a result of the river flow control related to a downstream hydro dam. A low berm (approximately 1 m high by some 25 m in width) covered by mature trees separates the east floodplain from the main river channel.

Review of river flow data supplied by Environment Canada from a monitoring station located at Appleton some 30 km upstream from the present site, indicates the following mean monthly flows in m³/second.

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
25.1	25.6	41.5	90.5	98.0	28.7	13.5	10.3	14.5	12.4	18.3	24.6

However, maximum daily discharges of up to 238 m³/second have been recorded.

For reference, the approximate cross sectional area of the river channel at the Mississippi River Bridge site is approximately 400 m² .

The existing east and west valley slopes are both approximately 4 to 5 horizontal: 1 vertical. Both valley slopes have been eroded by several gullies which drain the tableland into the river channel. One of these gullies is located within the west bank of the river channel along the proposed alignment of the Eastbound Structure.

The Ontario Ministry of Natural Resources has classified the floodplain area as a Class 1 Wetland. Otherwise, the lands surrounding the site are relatively flat and open with few trees, and enjoy a largely agricultural usage.

3.0 SITE GEOLOGY

The site of the proposed works is located within the Physiographic Region known as the Ottawa Valley clay plain (Chapman and Putnam 1984). This area is dominated by extensive deposits of sensitive marine clays, more commonly known as Leda Clay. This deposit was laid down following the retreat of the Laurentide ice sheet during the Wisconsin Age, approximately 13,000 years ago, while much of the Ottawa River Valley was submerged below the Champlain Sea. These deposits were derived from glacial outwash and from the erosion of these deposits as the ice front retreated. They were deposited under marine and estuarine conditions in an initially saline environment, which decreased in salinity as the Champlain Sea regressed (Klugman and Chung, 1976).

The bedrock at the site consists of Paleozoic deposits controlled by a series of northwest striking steeply dipping normal faults which transect the area. One such fault is reported to be present on Ontario Geological Survey (OGS) Map P2726 (1984) roughly 100 m south of the site. This fault divides the Verulam Formation, consisting of interbedded limestone and shale anticipated to be present under the site area, from the Gull River Formation to the south.

4.0 INVESTIGATION PROCEDURE

4.1 Field Program

The field investigation work for this project was carried out between March 1 and March 18, 1995, at which time the Mississippi River and adjacent floodplain areas were iced-over. Ice thicknesses in the order of 400 mm were observed at the borehole locations in the floodplain area. A total of 14 boreholes were advanced to depths ranging between 16.5 m and 49.9 m at the locations shown on the attached Figure 2 and Drawings 45190-03/04-A and B and summarized in the attached Table 1.

Drilling was carried out from the surface of the frozen ice within the floodplain area or from the ground surface of the adjacent tableland areas, using a track-mounted CME-55 drill rig supplied and operated by Marathon Drilling of Gloucester, Ontario. All boreholes were advanced through the overburden soils using 210 mm diameter continuous flight hollow-stem augers. The bedrock type, quality and elevation was confirmed in 10 of the boreholes using rock coring techniques with an N-size core barrel.

During drilling of the boreholes, Standard Penetration Tests (SPT) with associated disturbed soil sampling by means of a 50 mm outside diameter split-spoon sampler were carried out at regular intervals in accordance with ASTM D1586. Additionally, in Boreholes 95-1, 7, 8 and 14, 50 mm and 72 mm diameter thin-walled Shelby tube samples of cohesive soils were retrieved to provide relatively undisturbed samples for more detailed strength and compressibility testing. The undrained shear strength characteristics of the cohesive deposits were also investigated at regular intervals by means of in-situ shear vane testing with an MTO 'N'-size vane, and hand-held springs calibrated in 0.23 kg(0.5 lb) increments. Following each insitu determination of the peak undrained shear strength, the remoulded shear strength was obtained by rotating the vane 10 times and determining the undrained shear strength after an elapsed period of 60 seconds. The soil sensitivity was calculated as the ratio of the insitu peak undrained shear strength to the remoulded undrained shear strength.

A record of encountered soil conditions along with the results and locations of all in-situ testing and sampling, are presented on Borehole Logs 94-1 through 14 provided in Appendix A.

In addition to noting seepage and groundwater conditions during and upon completion of drilling, open-standpipe type piezometers were installed in all boreholes to monitor near-surface and deep piezometric levels. The piezometers consisted of 19 mm diameter Schedule 40 PVC pipe with

0.45 m long slotted tips. Installation details of the piezometers are presented on the Borehole Logs included in Appendix A and are summarized in Table 1.

Borehole backfill consisted of a sand filter around the piezometer tip, a thin bentonite seal (0.3 to 0.45 m thick) followed by cement: bentonite grout mixed at a 3:1 ratio by weight to surface. Where piezometers were installed in the middle of the hole, the portion of the hole up to the proposed piezometer tip was backfilled with grout. This backfilling procedure was adopted to help prevent problems related to environmental cross-contamination, and also to offset potential problems related to artesian water pressures known to be present close to the bedrock surface. However, artesian water flows from the top of the augers was not observed during the fieldwork and did not interrupt or complicate grouting procedures at any of the borehole locations.

Where artesian flow was observed from the top of a piezometer after installation, threaded PVC couplings with a top cap were glued to the top of the piezometer riser pipes to prevent flow. Piezometric head measurements at these locations were obtained by removing the top cap and attaching a pressure gauge to the threaded coupling, which was left in place for several minutes until stabilized pressures were recorded. Piezometer measurements with time are included on the borehole logs of Appendix A and summarized in Table 3.

In addition to the standpipes, six steel 25 mm drive-point piezometers (P1 to P6) were installed approximately 100 m north and 100 m south of the alignment at the approximate locations shown on Figure 2 and summarized in Table 1. The installation of these drive point piezometers was completed at the request of environmental unit of the MTO Eastern Region.

Borehole and drive-point piezometer locations were established by TEL in the field relative to survey stakes at the centre-line of the proposed abutment and pier locations that were established by the MTO in advance of the drilling program. Borehole northing and easting data as presented on the Borehole Logs and summarized in Table 1, were determined by TEL based on measured offsets from the previously referenced known survey points.

Borehole elevations were determined by TEL relative to the following temporary benchmarks established by the MTO:

TBM1: Nail and washer in root of a 0.60 m twin ash tree 18.2 m R.T. of station 18 + 132.9 E.B.L. (elevation: 84.216 m, geodetic)

TBM2: Nail and washer in root of a 0.60 m oak tree 10.8 m R.T. of station 17 + 827.6 (elevation: 89.693 m, geodetic)

Upon completion of all drilling works, the excess drill spoils from boreholes within the floodplain areas (Boreholes 95-3 to 6 and 95-10 to 13) were removed from the surface of the frozen floodplain using a rubber tired front end loader and disposed of at the local municipal landfill. Borehole spoils from the remaining on-land boreholes, were disposed of in adjacent treed/bush covered areas and/or in uncultivated areas around the perimeter of the fields.

4.2 Laboratory Testing

Recovered samples were brought back to TEL's Toronto laboratory and subjected to detailed visual classification and water content determinations. In addition, a number of grain size analyses, Atterberg limit determinations and unit weight measurements were undertaken on selected samples for more detailed soil classification purposes. Ten sulphate content determinations were also carried out for assessment of concrete mix design requirements. The results of the grain size analyses are presented on Figures B1 through B9 of Appendix B, Atterberg limits on Figures C1 through C6 of Appendix C, and summarized on the borehole logs. Moisture content and unit weight data are also summarized on the borehole logs, with a compilation plot of unit weight data versus elevation presented on Figure C7 of Appendix C. The results of the sulphate content determinations are presented in Table 4.

In addition to the above noted index testing, a total of six one-dimensional consolidation tests were completed on thin walled shelly tube samples retrieved from the east and west approach areas (three tests each side). This testing was completed by Golder Associates Ltd., Mississauga, Ontario under contract to TEL.

Samples from each abutment location, were selected to provide compressibility and consolidation data on the upper crust and the underlying softer deposits. Consolidation tests on the upper crust materials were completed using a standard load increment ratio of 1.0, whereas tests on the underlying weaker deposits were completed using a combination of load increment ratios of 0.5 and 1.0. All test load increments were terminated at times less than that required to investigate secondary compression

characteristics of the test samples. Void ratio versus logarithm of applied pressure plots for each test are presented on Figures D1 to D3 of Appendix D in accordance with the MTO format. A complete copy of consolidation test results, including coefficient of consolidation (C_v) data, as reported by Golder Associates Ltd., Mississauga, Ontario, is presented in the attached Appendix G.

In addition to the factual test data presented in Appendices D and G, a summary of test results, including sample locations and derived consolidation parameters, are presented in Table 5. It should be noted, that the data presented in Table 5 includes pre-consolidation (P_c) pressure estimates using the standard Casagrande procedure as well as the construction procedure proposed by Becker et. al. 1987.

5.0 SUBSURFACE CONDITIONS

5.1 General

The soils at the site consist of a thick layer of silty clay (Leda Clay) overlying a thin discontinuous layer of glacial till, overlying limestone bedrock at a depth of some 45 m below the floodplain level of the site. The silty clay consists of an upper layer of higher plasticity, overlying an approximately 20 m thick layer of less plastic material at depth. The upper layer of the clay has a stronger, brown dessicated crust in the order of 4.5 m in thickness present below both approach embankment areas, overlying weaker clay. However, in the central floodplain area of the site, the crust has been eroded by past river action, and the deposits in this region consist of 3 to 4 m of peat and recent alluvial deposits overlying weak silty clay.

A description of these individual stratigraphic units and their key engineering properties is provided within. However, for more detailed information, the reader should refer to the borehole logs of Appendix A and the stratigraphic and geotechnical information presented on the attached Drawings Nos. 45190-03/04-A and B and Figures 3, 4 and 5 and stratigraphic summary data presented in Table 2.

5.2 Stratigraphic Units

5.2.1 Topsoil

A layer of topsoil was encountered in the proposed east and west approach areas. As indicated in the attached Table 2, topsoil thicknesses at the borehole locations in this area of the site (Boreholes 95-1, 2, 6, 7, 8, 9, 13 and 14) varied from 150 to 300 mm.

5.2.2 Floodplain Deposits

Recent deposits varying in thickness from 3.1 to 4.9 m were encountered within Boreholes 95-3 to 5 and 95-10 to 12 drilled in the floodplain area, with maximum thicknesses of 4.3 to 4.9 m encountered in Boreholes 95-3 and 10, drilled from the river bank area adjacent to the main river channel. As indicated in the attached Drawing Nos. 45190 03/04-A and B, the floodplain deposits consists of an upper silty clay alluvium, overlying peat which in turn overlies a lower sandy clay alluvium.

Silty Clay (Alluvium)

Immediately adjacent to the main river channel, a layer of silty clay alluvium with a trace of sand and some organics, was encountered in Boreholes 95-10 and 11 along the EBL alignment and Borehole 95-3 along the WBL alignment. Maximum thicknesses of 2.3 m and 3.3 m, were identified in Boreholes 95-3 and 10 respectively adjacent to the river, decreasing to 1.8 m in Borehole 95-11. SPT values in the layer varied from 2 to 5 and as such it is considered to be soft to firm in consistency.

Peat

A layer of peat was encountered below the upper alluvial deposits in Boreholes 95-3, 10 and 11 and from ground surface in the remainder of the floodplain boreholes. Observed thicknesses of peat varied from 2.5 m in Borehole 95-5 to 1.3 m at Borehole 95-3. The peat is described as fibrous to woody near surface, although it becomes more amorphous at depth. In addition, in Borehole 95-11, deposits of organic marl were encountered at the base of this layer. Measured water contents in this layer varied from 125 to 315%.

SPT values in the peat were generally less than 1 with insitu vane undrained shear strengths typically between 25 and 50 kPa and accordingly, the peat is described as soft.

Sandy Clay (Alluvium)

A layer of sandy clay alluvium with some silt and shell fragments was encountered below the surficial peat deposits in Boreholes 95-4, 5 and 12 and on top of the underlying silty clay within the eastern portion of the floodplain area. Observed thicknesses varied from 0.6 to 1.5 m. SPT values of 2 were obtained in Boreholes 95-4 and 5 and based on this data, the layer is described as soft.

5.2.3 Silty Clay

5.2.3.1 Overview

A thick deposit of silty clay was encountered in each borehole and extended essentially from the surface to bedrock at a depth of some 45 m below the floodplain and is the dominant site deposit. This deposit is consistent with regional geological information which indicates the presence of deep deposits of Champlain Sea clay deposits, more commonly referred to as Leda Clay.

At this site, the silty clay consists of two major sub-layers above and below approximate elevation of 60 m, or 24 m below the elevation of the floodplain. The upper deposits are generally higher in plasticity and contain a higher percentage of clay particles relative to the lower clay deposit.

The upper silty clay has an upper stronger, brown dessicated crust below the east and west approach areas overlying weaker deposits. However, within the floodplain area the upper crust has been eroded. The shear strength of the weaker deposits increases with depth from a minimum value immediately below the upper crust in the approach areas and from the top of the deposit in the floodplain area. The shear strength in the lower layer below approximate elevation of 60 m is greater than that of the upper clay layer and the strength also increases more quickly with depth.

A discussion of these layers is presented in Sections 5.2.3.2 and 3 respectively. However, these sections should be read in conjunction with the borehole logs of Appendix A, stratigraphic summary presented in Table 2 and summary plots of geotechnical properties for this layer provided in Figure 3, 4 and 5.

5.2.3.2 Silty Clay to Clay

Dessicated Crust

Encountered in Boreholes 95-1, 2, 8 and 9 in the west approach area, the upper crust in this area of the site is of similar thickness in all boreholes and varied from 3.4 to 4.5 m. The base elevation of this layer within Boreholes 95-1 and 8 drilled from the flat tableland area was 87.0 m. The base elevation of the crust within Boreholes 95-2 and 95-9, drilled closer to the edge of the river, was 82.6 m and 83.6 m respectively (see Table 2).

In the east approach area, the thickness of this layer within Boreholes 95-7 and 14 was 5.0 m and 4.7 m respectively whereas within Boreholes 95-6 and 13 drilled from the toe area of the existing valley slopes, the thickness was 1.8 m and 1.7 m respectively. The base elevation of this layer was 83.0 m and 83.4 m at Boreholes 95-7 and 14 respectively and elevation 81.4 m and 81.7 m at the toe of the slope within Boreholes 95-6 and 13 respectively (see Table 2).

The results of 2 grain size analyses and 1 Atterberg Limit test on samples retrieved from the upper crust of the west approach area are presented on Figures B1 and C1 of Appendix B and C respectively. Similar test results from 1 grain size analysis and 1 Atterberg Limit test performed on samples of the crust obtained from the east approach area, are presented on Figures B2 and C2. The results of this and other index data, together with other key engineering data are presented on Figures 3 and 4 and are summarized as follows:

	<u>West Approach Area</u>	<u>East Approach Area</u>
Plastic Limit (%)	25	27
Liquid Limit (%)	57	61
Plasticity Index	32	34
Natural Moisture Content	32-58	29-42
Silt (%)	25-30	32
Clay (%)	70-75	68
Unit Weight (kN/m ³)	16.5-17.8	17.2-19.7
SPT	2-11	2-12
Undrained Strength (kPa)	92 to >115	90 to >115
Sensitivity	4 to 5	6

Based on the above data the crust materials under the east and west approach areas are quite similar, and are described as stiff to very stiff, medium sensitive, clay of high plasticity. This layer is predominantly brown in colour becoming grey towards the base of the crust.

Silty Clay to Clay

Located below the upper dessicated crust in the east and west approach areas of the site, and below the floodplain deposits in the central portion of the site, this portion of the layer represents the bulk of the upper layer of Leda Clay deposits. The thickness of this layer at the east and west abutment locations and at the central pier locations varied from 17.3 to 23.8 m with an estimated lower elevation of between 62.5 to 57.6 m (Table 2). Boreholes 95-1 and 8 at the west approach area and 95-7 and 14 within the east approach area were terminated within this layer.

The results of 9 grain size analyses on samples of this portion of the upper silty clay retrieved from boreholes along the WBL alignment are presented on Figures B3 and B4 of Appendix B with similar information from 7 Atterberg Limit tests presented on Figure C3 of Appendix C. Similarly, the results of 8 grain size analyses on samples of this layer retrieved from boreholes along the EBL alignment are presented on Figures B5 and B6 of Appendix B with similar information from 6 Atterberg Limit tests presented on Figure C4 of Appendix C. A summary of this data is presented below.

	<u>WBL</u>	<u>EBL</u>
Liquid Limit (%)	39-64	38-61
Plastic Limit (%)	20-27	16-33
Plasticity Index	19-37	22-28
% Silt	22-47	28-47
% Clay	53-78	53-72
Unit Weight (kN/m ³)	15.1-17.2	15.5-17.2

Based on the above data, this layer is described as a silty clay of intermediate to high plasticity. This portion of the layer is predominantly grey in colour, although it contains occasional black bands of totally decomposed organic matter. Methane gas was noted to vent through the augers from elevation 65.0 m while drilling within Borehole 95-6.

Summary plots of soil properties, including moisture content, Atterberg limits, shear strength, sensitivity and 'N' values versus depth are provided in Figures 3, 4 and 5 for the West Approach Area, East Approach area and Central Floodplain Area respectively. A summary of consolidation test data from the east and west approach areas, as presented in the attached Table 5 is also summarized on Figures 3 and 4 respectively. The data presented on these drawings indicates the following trends:

- 1) Increase in water content with depth through the upper crust material at the east and west abutment areas, achieving maximum values in the order of 70 to 80% at the base of the crust.
- 2) Decrease in water content with depth through the layer below the crust.
- 3) Water contents below the upper crust that are in excess of the liquid limit of the soil.
- 4) Decrease in undrained shear strength and SPT values with depth through the upper crust materials, achieving minimum values a short distance below the underside of the upper crust.
- 5) Linear increase with depth of undrained shear strength from minimum values at the underside of the upper crust within the east and west abutment areas, and from the top of the layer in the Central Floodplain area.
- 6) Decrease in estimated maximum pre-consolidation pressures (P'_c) through the upper crust materials.

With respect to the undrained shear strength data, the insitu vane data from the west and east approach areas (Figures 3 and 4) indicates minimum shear strengths of 32 kPa and 25 kPa respectively, increasing to approximately 60 kPa at the base of the layer at approximate elevation 60 m. Within the central floodplain area, the data presented on Figure 5 indicates a thin layer (2 m \pm) immediately below the floodplain deposits with shear strength in the order of 25 to 30 kPa which gives way to strength data that exhibits a similar trend and magnitudes to that within the east and west approach areas. Based on this data, this layer is described as firm to stiff in consistency. There is no discernable difference between shear strength data obtained within boreholes along the WBL and EBL alignments.

Finally, as a check on the insitu undrained shear strength (C_u) data, Mesri (1975), suggests that the shear strength of clay deposits similar to those present at the site, can be estimated using the relationship $C_u = 0.22 P'_c$ where P'_c is the maximum pre-consolidation pressure. Accordingly, C_u values obtained using this relationship based on information presented in Table 5, and presented on Figures 3 and 4, compare favourably with that obtained using the insitu vane.

Soil sensitivity values show quite a scatter for all three areas, with values through the upper clay layer (crust excluded) of between 2 and 10.5, with average values in the order of 5 below elevation 70 m and 6 to 8 above this level. Accordingly, this layer is described as sensitive to extra-sensitive.

Consolidation test results from within the east and west approach areas as presented in Table 5, indicate very similar properties. The data indicates relatively high over consolidation ratios (OCR) for those tests located in the upper crust deposits (Test 1, 4 and 5 of Table 5). However, for the remaining tests conducted on samples below the upper crust area, the estimated OCR values vary between 1.0 and 1.5.

Compression indices (C_c) for those tests completed on lightly over consolidated samples (Tests 1 to 3 and 5 to 6 of Table 5), show relatively constant values of between 1.18 and 1.72 with the exception of Test No. 6 which has a value of 0.60. Reloading indices (C_R) are relatively constant between all tests with values of between 0.021 and 0.074, with average values in the order of 0.060.

As presented in the attached Appendix G, coefficient of consolidation values (C_v) are considerably higher for load levels less than the estimated P_c' than those values obtained for load increments with stress levels greater than P_c' . Average C_v values at loading levels less than P_c' are typically in the order of 5×10^{-3} cm/sec, with average values of 5×10^{-4} cm/sec for loading levels beyond P_c' . Accordingly, the rate of consolidation related settlements for loadings less than the existing P_c' will be considerably greater than that for virgin consolidation at loadings greater than P_c' .

5.2.3.3 Silty Clay

Encountered below the upper silty clay to clay layer at all proposed abutment and pier locations, this layer extends to the underlying bedrock and/or glacial till. The estimated upper elevation of this layer within the boreholes varied from elevation 62.5 to 57.6 m with total layer thicknesses of between 23.0 m and 17.4 m (see Table 2). The lower elevation of this layer varied from 40.8 to 37.4 m.

The results of 4 grain size analyses on samples of this layer retrieved from boreholes along the WBL alignment are presented on Figure B7 of Appendix B with similar information from 5 Atterberg Limit tests presented on Figure C5 of Appendix C. Similarly, the results of 5 grain size analyses on samples of this layer retrieved from Boreholes along the EBL alignment are presented on Figure B8 of Appendix B with similar information from 4 Atterberg Limit tests presented on Figure C6 of Appendix C. A summary of this data is presented below:

	<u>WBL</u>	<u>EBL</u>
Liquid Limit (%)	26-44	28-43
Plastic Limit (%)	13-21	15-22
Plasticity Index	13-23	13-21
% Silt	41-36	59-37
% Clay	59-64	41-63
Unit Weight (kN/m ³)	17.8-19.3	17.6-17.9

Based on the above data, this layer is described as silty clay of low to intermediate plasticity. In addition, occasional silt partings were also noted in the samples. This layer is predominantly grey in colour.

Summary plots of soil properties, including moisture content, Atterberg limits, shear strength, sensitivity and 'N' values versus depth are provided in Figures 3, 4 and 5 for the West Approach Area, East Approach area and Central Floodplain Area respectively. The data presented on these plots indicates the following trends:

- 1) Apparent slight increase in water content with depth between approximate elevation 60 and 55, although there is insufficient data to more accurately confirm this aspect of this layer.
- 2) Gradual decrease in water content values from their maximum value in the order of 50% at Elevation 55 m to approximately 30% at the base of the layer.
- 3) Water contents in excess of the liquid limit of the soil.
- 4) Linear increase with depth of undrained shear strength values from top of layer at approximate elevation 60. Rate of shear strength increase with depth greater than the trend noted through the upper clay deposits.

The undrained shear strength data presented on Figures 3, 4 and 5, indicates minimum shear strength values of approximately 60 kPa increasing to greater than 115 kPa (limit of insitu vane equipment used at the site) at approximate elevation 50, or 10 m above the base of the layer. Based on this data, this layer is described as stiff to very stiff in consistency, with no discernable differences between data obtained along the WBL and EBL.

Soil sensitivity values in this layer are quite consistent and vary between 2 and 5. Accordingly, this layer is described as medium sensitive to sensitive.

5.2.4 Glacial Till

A thin layer of non-cohesive silty sand till with some gravel and clay was encountered below the overlying silty clay in Boreholes 95-6 and 95-13. The till layer is underlain by bedrock. The thickness of this till layer at these locations varied from 1.0 to 1.5 m. A grain-size distribution for the sample of till from Borehole 95-6 is presented in Figure B9 of Appendix B and consists of 11% gravel, 46% sand, 29% silt and 14% clay sizes. The natural water content of the till was 12% and 8% measured in samples from Boreholes 95-6 and 95-13 respectively.

One SPT completed in this layer within Borehole 95-6, recorded split spoon advance under the weight of hammer and based on this data, this layer is described as very loose. However, it is anticipated that this SPT value was affected by the presence of boiling sand conditions at the base of the hole, and may not accurately reflect the insitu relative density of this layer.

5.2.5 Bedrock

Limestone bedrock of the Middle Ordovician Verulam Formation was encountered in all boreholes put down at the proposed abutment and pier locations between elevation 40.8 and 36.8 m (Table 2). However, as indicated on the attached Drawing Nos. 45190 03/04-A and B, the bedrock elevation appears to have a gentle dip towards the east.

Based on visual assessment of recovered rock core from the upper 1.9 to 3.0 m of this unit, it is comprised of fine-grained, grey, bioclastic limestone with occasional thin shale partings. Typically, the discontinuous shale partings are wavy, 0.5 to 10 mm in thickness and are spaced 5 to 400 mm. The limestone is sound, unweathered and has an estimated intact uniaxial compressive strength in the order of 40 to 75 MPa. Core recovery in all runs was 100% with the Rock Quality Designation (RQD) determination generally above 80%. Based on this data, the rock quality is described as good.

One sub-vertical joint set was noted to be present; however, most of these fractures were calcite-infilled. Few to no horizontal joints were identified with the exception of partings along the shale contacts.

5.3 Groundwater

Piezometer details and monitoring dates are shown on the borehole logs and are summarized in Table 3. Based on these measurements and on-site observations, key features of the groundwater conditions at the site are the presence of standing water within the floodplain area, and the presence of artesian pressures with respect to the floodplain level, within the till unit.

Based on observations within the shallow piezometers within Boreholes 95-3 to 5 and 10 to 12, the standing water in the floodplain was at approximate elevation 83.15 on May 24, 1995. However, this level is anticipated to vary seasonally, and to change quite rapidly depending on the flow conditions in the river.

Artesian water pressures with respect to the elevation of the central floodplain area were recorded in all piezometers sealed directly above bedrock or within the till unit. The piezometric elevations across the site were as follows:

	Borehole	Measured Piezometric Level (m)	Ground Surface Elevation (m)
East Abutment	95-6	92.65	83.69
	95-13	90.94	83.45
West Abutment	95-2	90.11	86.92
	95-9	88.78	87.16
Central Piers	95-3	92.68	84.30
	95-10	85.65	84.42

With the exception of the deep piezometer in Borehole 95-10, these readings are relatively similar across the entire site, with minor variations believed to be related to the discontinuous nature of the thin till layer immediately above the bedrock surface. Also, the monitoring procedure of attaching a pressure gauge to the top of the piezometers may also account for the small differences.

With respect to water pressures within the silty clay deposits, the following piezometric level measurements were obtained within piezometers installed in the east and west approach areas on May 24, 1995.

	Borehole	Piezometric Elevation (m)	Ground Surface Elevation (m)
East Approach	95-7	85.72	88.06
	95-14	85.79	88.35
West Approach	95-1	89.47	91.45
	95-8	89.05	91.10

These piezometric levels are below existing ground surface elevation at these locations and are less than that of the underlying till unit, but are significantly higher than the standing water level in floodplain area at elevation 83.15 m.

Water level measurements in piezometers installed in the silty clay unit within the floodplain holes, with the exception of that installed in Borehole 95-11, gave values of between 85.53 m to 83.79 m on May 24, 1995 which are artesian with respect to the standing water elevation in the floodplain area of elevation 83.15 m.

Based on the foregoing data, it is concluded that pore water pressures within the silty clay unit are artesian with respect to the level of the floodplain area, and appear to be affected by the excess pressures in the underlying till unit. Based on the available data, the following piezometric profiles may be assumed for this site. It should be noted that the profiles presented below increase with depth at a rate greater than straight hydrostatic.

	Approximate Ground Surface Elevation (m)	Upper Piezometric Level (m)	Piezometric Level at Bedrock Surface (Elevation 40 m)
East Approach Area	88	86	91
West Approach Area	91	89	90
Floodplain Area	83.2	83.2	91

6.0 ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

6.1 General

As part of the proposed extension of Highway 417 from Highway 44 to Arnprior, two lane eastbound and westbound structures (EBL and WBL) are required to cross the Mississippi River and its associated floodplain. The currently proposed structures consist of four-span bridges supported on two abutments and three intermediate piers. The three central piers are proposed within the floodplain area, with both abutments proposed near the toe of the existing valley slopes. Approximate abutment locations and related span lengths are provided below.

Element	Westbound Structure		Eastbound Structure	
	Station (km + m)	Span Length (m)	Station (km + m)	Span Length (m)
West Abutment	17 + 878		17 + 853	
		85		84
Pier #1	17 + 963		17 + 937	
		65		65
Pier #2	18 + 028		18 + 002	
		65		65
Pier #3	18 + 093		18 + 067	
		65		65
East Abutment	18 + 158		18 + 132	

The longer span between the west abutment and Pier #1 location for both structures facilitates spanning the main river channel without the need for a support point within the channel. The remaining spans are used to traverse the east floodplain area. Currently proposed abutment locations are at the toe area of the existing valley slopes.

The proposed design grade at the west abutment location is elevation 95.0 for both EBL and WBL structures. The existing ground surface elevation at the crest of the west valley slope at this location varies from elevation 90 to 91 m. The ground surface elevation at the proposed abutment locations varies from elevation 84 to 85 m. The proposed design grade of the EBL structure at the east abutment is 93.5 m with a value of 94 m for the WBL structure. The reported ground surface elevation at the crest of the east valley slopes along both EBL and WBL alignments is

elevation 88 m. The ground elevation at the toe of the east valley slopes is in the order of elevation 84 m. The distance from the crest of both the east and west slopes to the toe of the slopes is in the order of 20 m.

The presence of deep deposits of weak, compressible silty clay at the proposed site, presents considerable difficulties related to support of the anticipated high bridge loads and accommodation of the expected ground settlements associated with the construction of the approach embankments. The most appropriate foundation support system for a structure of this size at this site is considered to be driven steel H piles end bearing on bedrock. However, large ground settlements due to the construction of the presently proposed earth fills of 10 m height behind the proposed abutment locations, present significant concerns with respect to the serviceability of the structure and negative skin friction forces on pile foundations at the abutments. Embankment stability of proposed retained fills behind the currently proposed abutment locations is also of concern.

A practical solution to alleviate these concerns is to extend the bridge and relocate the abutments on the flat tablelands. This will reduce the abutment fill heights. Furthermore, the abutment fills on the flat tablelands adjacent to the proposed relocated abutments, should be constructed well in advance of the installation of the relocated abutment foundation elements so that a significant part of the settlements under the abutment fills is completed prior to the installation of these elements. To accelerate the settlements, foundation drainage consisting of wick drains should be used in the abutment fill areas in conjunction with preloading. The report discusses these solutions in the following sections.

In addition to the above design concerns, the floodplain area east of the main river channel has been designated as a Class 1 Wetland by the MNR which will impose additional design and construction requirements to minimize both short and long term negative impacts. It is understood that consideration is being given to design of an incrementally launched bridge to reduce the extent of disturbance within the floodplain area. Accordingly, this proposed construction methodology will require detailed analysis of intermediate loading conditions on foundation elements during construction, that may be more critical than the long term condition.

A discussion on approach fill settlement and embankment stability aspects of the design is presented in Section 6.2, with a discussion on foundation related items presented in Section 6.3. General design and construction related items are presented in Section 6.4. A preliminary discussion of environmental related aspects of bridge construction within the designated Class 1 Wetland Floodplain area is presented in Section 6.5.

6.2 Approach Embankment Design

6.2.1 Overview

Approach embankment heights in the order of 4 to 5 m are proposed along the flat tableland areas behind the west abutments and 5 to 6 m behind the east abutments. However, for the currently proposed east and west abutment locations at the toe of the existing valley slopes, embankment heights increase over a distance of 20 m to achieve maximum heights in the order of 10 m immediately adjacent to the proposed abutments. Key design concerns related to this element of the project are considered to be the stability and settlement of the approach embankments and retained fills behind the abutments. The design rationale and results of analyses into these elements of the design are presented in the following sections.

6.2.2 Stability Analysis

6.2.2.1 Design Parameters

The interpreted design stratigraphy and porewater pressure distribution for the east and west approach areas are presented on the attached Figures 6 and 7, with interpreted undrained shear strength profiles used in the short term stability analysis presented on the attached Figures 8 and 9 respectively. The shear strength profiles were selected to envelope all of the recorded insitu vane shear data, and derived undrained shear strength estimates based on the relationship $C_u = 0.22 P_c'$ as suggested by Mesri (1975). The insitu vane shear strength data has not been modified based on plasticity as proposed by Bjerrum (1972), as most of the soils on-site have a plasticity index in the order of 20. The inclusion of the derived C_u values based on the relationship $C_u = 0.22 P_c'$, is considered to account for the slightly higher plasticity observed in the near surface portion of the clay layer.

Based on a review of data presented by Conlon (1966) and Bozozuk (1972), effective friction angles (ϕ') ranging from 23.5° to as high as 36° are reported for Champlain Clay deposits of this nature. $C' = 0$ is considered appropriate for long term analyses.

A unit weight of 19.5 kN/m³ and effective friction angle of 30° was assumed for the embankment fill in the analysis presented within.

6.2.2.2 Short Term Stability

For embankments constructed on normally to lightly over consolidated clays, the stability during fill construction (end of construction case) is expected to be the critical case. Once the fill is successfully completed and construction pore pressures dissipate with time, the safety factor against embankment failure is expected to increase.

The results of two-dimensional undrained stability analyses for variable fill heights and side slopes related to construction of the east approach and abutment embankments, are presented on Figure 11 with the results of similar analyses for the west approach area presented on Figure 12. Example plots of the most critical failure circles for various embankment heights are presented in Appendix E.

The data presented in Figures 11 and 12 indicates the following embankment safe heights for fills constructed using regular earth fill to achieve a minimum short term factor of safety of 1.3 which is considered applicable for the present site.

Location	Embankment Maximum Safe Height (m) for Factor of Safety = 1.3			
	Abutment Fills at toe of Valley Slopes		Approach Fills along Tableland Area	
	2H:1V	3H:1V	2H:1V	3H:1V
East Approach Area	5.6	6.2	9.6	10.0
West Approach Area	8.6	9.0	9.7	10.0

Based on the above data, the stability of the proposed heights of approach fills along the flat tableland areas behind both abutments is not a concern since the maximum estimated safe heights are greater than the maximum proposed embankment heights of 4 to 6 m. However, at the abutment locations, maximum allowable embankment safe heights at both locations are less than the proposed fill height of 10 m. Further stability analyses at these locations indicate that a factor of safety of 1.3 can be achieved for 2H:1V fill side slopes by the addition of 17 m and 10 m long, 5 m high stabilizing berms at the east and west abutment locations respectively.

Stability analyses summarized in Figures 11 and 12, a selection of which are presented in Appendix E, were carried out using the Bishop's Simplified Method and the slope stability computer software package GSLOPE distributed by MITRE Software Corporation, Edmonton, Alberta.

It should be noted that the abutment analysis assumes a plane strain condition which is probably not quite applicable due to the relatively short length of deep fills present behind the currently proposed abutment locations. However, the data is applicable for the assessment of the stability of embankments constructed at this location in advance of the abutment construction.

6.2.2.3 Long Term Stability

Effective stress analysis of 5 m high approach embankments constructed along the flat tableland areas indicates that a minimum effective friction angle of 28° (with $c' = 0$) is required to achieve a long term factor of safety of 1.5 for a 5 m high fill with side slopes of 2H:1V which is considered acceptable for the long term condition. This value is within the earlier quoted range of 23.5° to 36° for typical effective friction angle for these clay deposits, and accordingly, approach embankments constructed along the flat tableland area to a maximum height of 5 m are considered acceptable at side slopes of 2H:1V.

As previously mentioned, the short term analysis indicates that for 10 m high abutment fills as currently proposed, an end of construction safety factor of 1.3 can be achieved by addition of 17 m and 10 m long, 5 m high stabilizing berms at the east and west abutment locations respectively. For a long term safety factor of 1.5, a minimum effective friction angle of 28° ($c' = 0$) is required for the 10 m high fill with berms and this value is within the earlier quoted friction angle values of 23.5° to 36° . These analyses assume that the embankment fill will have an effective friction angle of 30° .

6.2.2.4 Discussion

The results of the analysis presented within indicate that stability of the proposed approach embankments along the flat tableland area is acceptable for the currently proposed maximum embankment heights of 5m. The design of these fills at side slopes no steeper than 2 horizontal to 1 vertical is recommended to satisfy both short and long term stability requirements. From a stability point of view only, no special construction procedures are considered necessary for embankments up to 5 m in height along the flat tableland areas.

Stability analysis for the proposed abutment fills constructed with side slopes of 2H:1V indicate that short term maximum safe height of 5.6 m at the east abutment area and 8.6 m at the west abutment area are both less than the currently proposed design fill heights of 10 m at these locations.

6.2.3 Settlement Analysis

6.2.3.1 Design Parameters

Settlements under embankments at this site are anticipated to consist of an initial elastic component, followed by settlements related to primary consolidation of the thick clay layer as construction porewater pressures dissipate. In addition, the Canadian Foundation Engineering Manual indicates that significant settlements related to secondary consolidation may occur if imposed loads approach or exceed the pre-consolidation pressures. Lo et. al. (1976) report that secondary compression contributed significantly to the settlements observed at the Gloucester Test Fill.

The analyses presented within has concentrated on the magnitude and rate of settlement related to primary consolidation of the upper silty clay layer, and a preliminary evaluation of secondary consolidation settlements. Settlements of the lower silty clay unit below approximate elevation 60 m, have not been included due to the anticipated low stress levels within this layer. Initial elastic settlements are anticipated to be in the order of 25 mm and will occur during construction.

Based on the data summarized in Table 5 and actual test results presented in Appendix D, the following consolidation data have been assumed for the settlement analyses.

	Loading Less than P_c'		Loading Beyond P_c'	
	C_R (Rebound Index)	C_v^* (cm ² /sec)	C_c (Compression index)	C_v (cm ² /sec)
Upper Silty Clay Crust	0.06	5×10^{-3}	n/a for current design loads	5×10^{-4}
Upper Silty Clay	0.06	5×10^{-3}	1.7	5×10^{-4}
Lower Silty Clay	Not Included in Analysis			

* C_v = Coefficient of Consolidation
 P_c' = Pre-Consolidation Pressure

The P_c' profiles with depth used in the settlement analyses for the east and west approach areas are presented in the attached Figure 10. The P_c' profiles in the upper regions of the upper silty clay layer were selected based on actual P_c' consolidation test data as summarized in Table 5.

However, for the remainder of this layer at depths below the lowest test data, the P_c' profiles were obtained using an OCR value of 1.5. As noted on the attached Figure 10, minimum P_c' values of 135 kPa at the west abutment and 110 kPa at the east abutment were adopted. All settlements within the crust areas has been assumed to occur at pressures less than P_c' .

For a final assessment of the P_c' design profile obtained as described above, Figure 10 also includes the P_c' profile with depth, back calculated using the relationship $C_u = 0.22 P_c'$ (Mesri 1975) and the design C_u profiles of Figures 8 and 9. The plots presented on Figure 10 indicates that there is generally good agreement between the two profiles in both east and west approach areas, although the P_c' profile within the east and west abutment locations is less than that predicted from the C_u profile. Based on this assessment, it is concluded that the settlement data presented within for the approach fills represents a satisfactory average condition, whereas the settlement estimate for the abutment fills represents a conservative condition.

The rate of secondary consolidation with time (C_α), defined as the percent change in layer thickness per log cycle of time, is dependent on the ratio of the stress increment applied to the layer to P_c' . Laboratory test results reported by Lo et al., (1976) from the Gloucester Test Fill site, indicate a C_α value in the order of 0.02 for the stress increases at shallow depth related to construction of a 2.4 m high fill (net height increase). However, test completed at low stress levels, indicate a C_α value of 0.003. Accordingly, for a preliminary assessment of secondary consolidation settlements, an average C_α of 0.01 has been assumed for the upper silty clay to clay layer above elevation 60 m. Secondary compression below this depth is not considered to be a concern.

6.2.3.2 Magnitude of Primary Settlement

The results of primary consolidation settlement analysis under the centre line of the proposed embankments, including estimates of initial and final effective stresses and assumed P_c' profiles, are presented in accordance with the following.

- Figure 13 - East Approach Embankments
- Figure 14 - East Abutment Embankments
- Figure 15 - West Approach Embankments
- Figure 16 - West Abutment Embankments
- Figure 17 - Abutment Embankment Settlements versus Fill Contact Pressure
- Figure 18 - Approach Embankment Settlements versus Fill Contact Pressure

The vertical stress increase profiles presented on Figure 13 to 17 were determined assuming plane strain conditions and using an elastic solution of a problem where a uniform vertical loading is applied to the surface of a semi-infinite mass. Excess pore water pressures immediately after construction, were estimated assuming undrained conditions, Poisson's ratio of 0.5, and values of 0.33 and 1.0 for Skempton's A and B parameters respectively.

Summary plots of the estimated distribution of vertical settlement with depth through the upper clay layer for variable fill heights are presented on Figures F1 to F4 in Appendix F for the above noted cases respectively.

The results of these analyses indicate the following:

- 1) For the currently proposed approach embankment heights of approximately 5 m along the tableland portions of the site, maximum primary consolidation settlements in the order of 200 mm at the West Approach area and 350 mm at the East Approach area are expected (refer to Figure 18) if traditional earth fill materials are utilized for embankment construction.
- 2) At stress levels applied by earth fills in excess of 5 m in height within both approach areas, settlement magnitudes increase considerably.
- 3) For the currently proposed maximum embankment heights in the order of 10 m adjacent to both abutments, the predicted increases in effective stresses with depth due to embankment construction using conventional earth fill are greater than the estimated P_c' profile and settlement estimates are excessive (3.0 m at the east abutment and 2.0 m at the west abutment, refer to Figure 17). As noted earlier, settlement estimates at these locations are considered to be conservative estimates.
- 4) The settlement results presented on Figure 18, indicate that settlements of the approach embankments can be reduced to about 100 to 115 mm, if contact stresses at the base of the fill do not exceed about 60 kPa.

The estimated settlements for the 5 m high east and west approach embankments along the flat tableland areas, are consistent with reported settlements of 270 mm under the centre-line of the Gloucester Test Fill with a net height of 2.4 m. The estimated settlements of between 2 and 3 m of the higher retained abutment fills of 10 m, is consistent with total

settlements of 2.2 m after 7 years reported by Bozozuk, 1972 for a 9 m high fill constructed on a deep deposit (70 m+) of marine silty clay at Berthierville, Quebec.

6.2.3.3 Rate of Settlement

Based on the consolidation parameters presented in Section 6.2.3.1, the estimated rate of excess porewater pressure dissipation (consolidation) and rate of development of primary consolidation settlements in the upper clay layer above elevation 60 m are presented on Figures 19, 20 and 21 for the approach and abutment fills respectively using the previously outlined design C_v parameters. The results presented on these figures include estimated time of consolidation assuming one way vertical drainage for the deposits above elevation 60 m to the surface only, and the improvement in dissipation time related to the installation of wick drains (or similar) at variable spacing (horizontal C_h values assumed equal to vertical C_v values).

The results indicate an estimated time to achieve 90% consolidation in the order of 28 years for the approach fills and over 200 years for the abutment fills. The greater time estimates for the abutment fill reflects the lower C_v values related to the greater amount of virgin consolidation related to higher fill heights. However, for an average C_v value of 3×10^{-3} cm/sec to account for the fact that all of the abutment loadings are not at stress levels in excess of P_o' , the estimated time for 90% consolidation reduces to 50 years (Figure 21). These estimated times for 90% consolidation are based on a uniform homogeneous clay layer with one-way drainage and do not reflect the presence of higher permeability silt lenses that are often present in these deposits which considerably increase the rate of consolidation. Although features of this type were not observed in the soil samples, published results from case histories indicate a large percentage of primary consolidation settlements are usually completed after about 10 to 15 years.

The summarized results presented on Figure 19, 20 and 21 indicate that the time required to achieve 90% consolidation under the 5 m high approach fills can be reduced to about 0.25 to 0.5 year if wick drains are installed to achieve an influence zone equivalent to a circle 2 to 4 m in diameter. It should be noted that the wick drain analysis does not include an allowance for potential clay smear related to their installation. Accordingly, based on experience from similar projects, wick spacings of 2 to 3 m is considered appropriate for this site.

6.2.3.4 Secondary Consolidation

Reported results for the Gloucester Test Fill (Lo et.al 1976) and other similar projects indicate that embankments constructed on similar materials experience significant secondary consolidation settlements when stress levels increase to greater than 80% of P_c' . As indicated in the attached Figures 13 to 16, stress levels under all proposed earth fills exceeding 3 m in height (average surface contact stress of approximately 60 kPa) contain significant depths where the final predicted stress levels exceed these recommendations and accordingly, settlements of this nature may be anticipated with time for all proposed embankments at this site.

Based on secondary consolidation index (C_α) of 0.01, secondary consolidation settlements with time are estimated to be in the order of 100 to 200 mm. These settlements will be in addition to those predicted for primary consolidation.

6.2.3.5 Discussion

Settlement estimates indicate that primary consolidation settlements under the currently proposed 10 m high abutment earth fills is 2 m to 3 m. The estimate also indicates 200 to 350 mm of settlement under the presently proposed 5m high approach earth fills in the west and east approach areas. These settlements are too excessive and cannot be accommodated in the bridge design.

The settlement assessment indicates that fill contact pressure under the approach embankments along the flat tableland areas should be limited to 60 kPa in order to limit primary settlements to about 100 mm. It is therefore recommended that the bridge be lengthened to reduce both east and west abutment fill heights so that these fills do not apply more than 60 kPa contact pressure on the ground. Even with fill contact stress of 60 kPa (fill height of about 3 m using regular earth fill), anticipated settlement of about 100 mm is too large to be accommodated in bridge design. Accordingly, the abutments fills should be constructed well ahead of foundation and bridge construction to facilitate a significant portion of the settlement to be completed. It is further recommended that the east and west abutment fills be preloaded and surcharged and provided with foundation drainage in the form of wick drains to accelerate settlements. Preload fill should be constructed of regular earth fill which should be removed after preloading is completed and the abutment fills constructed with light weight fill.

Details of the above recommendation are given in the following section.

6.2.4 Abutment and Road Approach Fill Construction

6.2.4.1 Abutment Fills

As indicated earlier, in order to limit settlements to an acceptable level, it is recommended that the bridge be lengthened to reduce the required abutment fill heights and that the embankment fill construction utilize lightweight polystyrene materials to limit the embankment contact pressure to 60 kPa. This implies that the abutments would be placed on the flat tablelands some distance back from the crest of the valley slopes and that no abutment or deep fills would be placed in the river valley thus reducing environmental concerns.

Even with the relocated abutments and limiting the abutment embankment contact pressure to 60 kPa the estimated primary settlements are too large to be accommodated in the bridge design and operation. Accordingly, the following is recommended to accelerate settlements and reduce post construction settlements to an acceptable level for bridge design and performance.

1. Construct the east and west abutment fills at least 1 year ahead of construction of the foundation support elements of the relocated abutments. Fill lengths extending from 15 m in front of the centre-line of the relocated abutment to a point behind the abutments where the proposed fill contact pressure are less than 50 kPa is recommended for advance construction.
2. Prior to embankment construction, strip all topsoil, organic and deleterious materials from the abutment fill areas.
3. Install wick drains from the prepared surface outlined under item 2, to elevation 60 m on a 2.5 m centre to centre triangular pattern. Wick drains should be installed under the entire footprint of the preload fill from 10 m in front to 20 m behind the centre-line of the relocated abutment. Since these drains will be covered by abutment fill, a 300mm layer of free draining granular material should be placed over the wick drains to act as a drainage layer. It is recommended that at least 2 pneumatic piezometers be installed in each abutment area (project total of 8) to monitor wick drain performance.

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4. After the wicks are installed, preload and surcharge with regular earth fill. Preload plus surcharge load contact pressures should be at least 40 kPa greater than the final design contact pressure up to a maximum allowable of 100 kPa.
 5. The settlement of the preload fills should be monitored using settlement plates. It is recommended that at least four settlement plates be installed in each abutment area (project total of 16).
 6. It is expected that with the wick drains and the surcharged preload, a significant portion of the settlement should be completed in 1 year.
 7. Subsequent to the period of preloading, the preload earth fill should be removed for a distance of 20 m behind the rear face of the abutment and the embankment reconstructed in this area using lightweight polystyrene fill to achieve a maximum fill contact stress of 60 kPa. The 300 mm granular drainage layer over the wick drains should not be removed.
 8. Design details of the light weight polystyrene fill are outlined in the MTO's non-standard special provision for polystyrene embankment which is available from MTO's pavement and foundation section.
 9. Polystyrene fill should be covered with at least 2 m of regular earth fill for frost protection.
 10. The light weight polystyrene fill beyond a distance of 20 m from the back of the abutment should be feathered into the approach earth fill at a 5:1 slope.

The above recommended scheme is expected to reduce the post construction settlements at the abutments; however, there may still be a residual settlement of up to 50 mm at the abutments that will occur with time. The bridge should be designed to accommodate up to 50 mm of settlements at the abutments.

6.2.4.2 Road Approach Fills

The settlements under the east and west approach fills may be estimated from Figure 18. The estimates indicate that settlements approaching 100 mm or more may be expected for fill contact pressure of 60 kPa. It is therefore recommended that all fills which will apply a contact pressure of greater than 50 kPa should be preloaded and surcharged one year in advance to complete a significant portion of the settlement prior to paving.

The above recommendations apply between Station 17+800 to 18+225 for the westbound lanes and 17+775 to 18+200 for the eastbound lane which was our scope of work. It should be noted that similar settlements may occur under similar heights of road approach fill beyond the scope of this investigation.

6.3 Foundation Design Recommendations

6.3.1 Overview

The currently proposed EBL and WBL bridge structures will be of four span construction, supported on two abutments and three central piers. As outlined in Section 6.1, the most westerly span over the main channel of the Mississippi River is in the order of 85 m with the remaining spans through the floodplain area at 65 metres. For these span lengths, prestressed, continuous concrete deck structures, with heavy foundation loads are anticipated. In addition, to help minimize disturbances to the Class 1 floodplain area, consideration is being given to construction of an incrementally launched structure to avoid the necessity for extensive temporary false work that would be required within the floodplain area for a cast-in-place structure. However, if an incrementally launched design is adopted, careful evaluation of intermediate foundation loading conditions during construction that may be more critical than the end of construction case will have to be completed.

In order to accommodate the anticipated magnitude of loads, the most appropriate foundation support system for the currently proposed bridge configuration is considered to be driven steel H piles, bearing on bedrock. Other potential foundation support systems, such as friction piles or end bearing caissons are not considered suitable for the current bridge configuration due to the limited allowable loads to limit the settlement of friction piles, and the feasibility of end bearing

caisson construction through the great depths of overburden present at this site.

Large long term settlements of the currently proposed approach embankments are expected to occur which may not only result in some differential settlements at the bridge abutments but will also result in downdrag forces in the abutment piles and consequent reduction of the abutment pile load carrying capacity. As discussed earlier, this problem can be alleviated by adding outer spans to the bridge which will reduce the height of the approach embankments.

6.3.2 End Bearing Steel H Piles

6.3.2.1 End Bearing Capacity

Steel H piles driven to practical refusal on the underlying bedrock may be designed for the following factored geotechnical resistances at the ULS condition.

Pile Type	Size	ULS Factored Geotechnical Resistance (kN)
Steel H	HP310x79	1150
Steel H	HP310X110	1600

For estimation purposes, pile lengths based on bedrock elevation data summarized in Table 2 may be assumed. Settlements at the SLS condition may be assumed equal to the elastic compression of the pile.

The pile capacities should not exceed the structural capacity of each pile.

6.3.2.2 Negative Skin Friction

At the presently proposed abutment locations involving up to 10 m of fill, the settlement estimates are excessive (> 2 m) which will cause significant downdrag forces on the abutment piles. However, if the recommendations provided in Section 6.2.4 are implemented (prior to foundation installation) including lengthening the bridge, reducing abutment fill height and limiting fill contact pressure to 60 kPa, preloading the abutment area and providing wick drainage to

accelerate and reduce post construction settlements below abutment fills, then downdrag forces in the abutment piles are not a concern.

Negative skin friction forces at the proposed central piers is not considered to be a concern and may be neglected in the design, provided any fills associated with access roads constructed through the floodplain area are removed after foundation and pier construction is complete.

6.3.3 Installation

Pile driving operations through the overlying silty clay layer should be relatively straightforward, although set-up of partially installed long piles may be a problem if pile driving operations are suspended. The use of pile driving shoes are recommended to help protect the pile during seating onto the bedrock surface. They will also protect against boulders that may be present in the underlying glacial till layer.

Pile driving operations should be closely monitored at all times, and a continuous pile driving record for each pile should be kept. Upon obtaining refusal on bedrock, the elevation of the pile should be accurately surveyed and checked after installation of adjacent piles for any evidence of pile heave. Piles that are determined to have heaved, should be re-driven.

It is anticipated that pile refusal will be obtained at or below the bedrock elevations shown on the attached borehole logs of Appendix A and summarized in Table 2.

There is some concern that piles driven to bedrock may communicate artesian water pressures to surface, possibly causing piping of fines from depth. Our experience in drilling at this site suggests that there is a time lag of several days between the time the till unit overlying bedrock (or sand seams deep in the Leda Clay) are penetrated and artesian flow reaches the surface. Flow volumes are also relatively low. However, the potential for localized areas of higher permeability cannot be ruled out. The artesian flow was also noted to contain dissolved salts. Discharge of these artesian water into the river may present an environmental concern.

It is anticipated that the silty clay materials will be remoulded over a sufficient length during driving operations, and will close in around the piles to create an effective seal against potential artesian flows. However, as an additional precaution, it is recommended that the upper 3 m of the piles through the silty clay layer be installed through a pre-augered hole and backfilled with a bentonite/cement grout after pile driving operations are complete. The sealing approach is preferred over providing a granular filter under the pile cap because there is a potential of adding water discharge containing salts to the river which may create an environmental concern. This approach will require the use of casing for piles installed through the floodplain deposits.

6.3.4 Lateral Pile Resistance

Spring constants for the evaluation of long term lateral pile movement of a 310 x 79 steel H pile subjected to horizontal loading and spaced not closer than 5 pile diameters, may be calculated by multiplying the following coefficient of horizontal subgrade reaction (k_H) values by the spring influence length.

Steel H piles placed at toe area of existing Valley Slopes				
Material Type	East Abutment		West Abutment	
	Depth Limits	K_H MN/m ³	Depth Limits	K_H MN/m ³
Upper Clay	Elevation 81 to 76	33	Elevation 83 to 75	33
Upper Clay	Elevation 76 to 70	50	Elevation 75 to 69	67
Upper Clay	Elevation 70 to 60	67	Elevation 69 to 60	83
Lower Clay	Below Elevation 60	83	Below Elevation 60	83

Steel H piles at least 5 m back from the crest of the existing Valley Slopes				
Material Type	East Approach		West Approach	
	Depth Limits	K_H MN/m ³	Depth Limits	K_H MN/m ³
Upper Crust	Base of Pile Cap to Elevation 84	157	Base of Pile Cap to Elevation 86	157
Upper Clay	Elevation 84 to 80	73	Elevation 86 to 82	57
Upper Clay	Elevation 80 to 74	33	Elevation 87 to 75	40
Upper Clay	Elevation 74 to 70	57	Elevation 75 to 69	67
Upper Clay	Elevation 70 to 60	67	Elevation 69 to 60	83
Lower Clay	Below Elevation 60	83	Below Elevation 60	83

The determination of spring constants for use in the numerical analysis of lateral pile movement may be obtained using the following relationship:

Spring Constant (K) = $K_H \times B \times L$, where

K_H = coefficient of horizontal subgrade reaction (MN/m³)

B = width of 310 x 79 pile (0.3 m)

L = influence length of spring element

It should be noted, that the assessment of lateral pile and resulting pile cap and abutment movements should include the anticipated lateral loads related to the vertical stress increase under the proposed approach embankments, unless approach embankments are constructed well in advance of construction of the foundation elements.

An assessed horizontal passive resistance for a 310 x 79 steel H pile may be taken as 120 kN at the ULS condition and 60 kN at SLS condition. If the settlements at the abutments are largely completed by advance fill construction, preloading and provision of foundation wick drains, then the lateral pile capacity may be supplemented by using inclined (batter) piles. The use of batter piles are not recommended if the abutment embankments are not constructed in advance and the maximum contact pressure at the base of the retained fill does not exceed 60 kPa to limit secondary compression.

We have assumed that the river engineering aspects of the bridge design including assessment of potential of shifting of the river channel and associated erosion will be carried out by others. The scour potential of the river around the proposed foundation elements adjacent to the main channel should also be assessed by a suitably qualified river engineering expert.

It is anticipated that these potential concerns can be mitigated by either placing the pile caps below the maximum expected depth of scour or the piles exposed above scour depth should be designed for lateral stability. These concerns could also be mitigated by the installation of suitably designed erosion protection measures such as rip-rap or possibly steel sheet piles along the river bank areas adjacent to the west abutment and Pier 1 locations.

6.3.5 Abutment Backfill

To provide for free drainage of water behind the retained abutments, the use of free draining granular material, such as that meeting the gradation requirements of OPSS Granular A or OPSS Granular B limited to a maximum fines content of 8% passing the No 200 Sieve, is recommended behind the proposed bridge abutments to the dimensions outlined in Ontario Provincial Standard Drawing OPSS-3501.00. Weep holes to facilitate drainage through the abutment should also be provided. Construction specification for compaction should meet Ontario Provincial Standard Specification OPSS-501.

Section 6-7 of the Ontario Highway Bridge Design Code should be used for estimating lateral earth pressure against the abutment walls. Following are the soil parameters which should be used in the earth pressure estimation:

$$\begin{aligned}\gamma &= \text{unit weight of compacted granular fill} = 21.2 \text{ kN/m}^3 \\ \phi' &= \text{angle of internal friction: } 35^\circ \text{ for Granular A} \\ &\quad 30^\circ \text{ for Granular B}\end{aligned}$$

Heavy vibratory compaction equipment should not be allowed within a wedge obtained by drawing a line at 1H:1V from the toe of the abutment.

6.4 General Construction Recommendations

6.4.1 Concrete Sulphate Requirements

Based on the soluble sulphate testing presented in the attached Table 4, the reported sulphate content values are less than 0.1% at all locations with the exception of the test result on a peat sample obtained from within Borehole 95-5. Accordingly, based on this data, it is concluded that the soils in the central floodplain area of the site have a moderate risk of sulphate attack.

6.4.2 Temporary Excavations

Temporary excavations for pile cap construction are anticipated to be required through the overlying clay deposits at the east and west abutment locations and through the floodplain deposits at the central pier locations.

Excavations through the upper clay deposits at the east and west abutment locations are anticipated to remain stable if excavated not steeper than 1 horizontal to 1 vertical to the maximum expected depth of 2 m. If excavations are required beyond this depth, their stability should be carefully checked by a suitably qualified geotechnical engineer using the undrained shear strength profiles presented on Figures 8 and 9 and/or other factual information presented in this report as appropriate. Groundwater seepage from the native soils into excavations at these locations is expected to be minimal and may be handled by sumps and pumps.

Temporary open cut excavations through the floodplain deposits to permit construction of the pier pile caps are not anticipated to be feasible from a technical or environmental perspective. Accordingly the use of a suitably designed cofferdam consisting of either a large diameter caisson liner or a continuous steel sheet pile enclosure installed through the upper flood plain and peat deposits and terminated in the underlying low permeability silty clay is recommended. After initial drainage of water inside the sheet pile enclosure and if flow through permeable layers (granular layers, peat) has been cut off by the piles, dewatering within the cofferdam may be achieved using a conventional sump and pump arrangement. In light of the poor soil conditions anticipated at the base level of the pile cap excavation at the central pier locations, it is recommended that a 0.3 m thick layer of clear crushed stone be placed immediately upon excavation to provide a better working surface.

The cofferdam should be designed by a Registered Professional Engineer experienced in designing cofferdams. For estimation of the active and passive earth pressures for design of the cofferdam the following soil parameters should be used.

γ = unit weight of soil = 16 kN/m³

K_A = coefficient of Active Earth Pressure: 0.5

K_p = coefficient of Passive Earth Pressure: 2.0

The cofferdam should be designed for the prevailing water level in the river. Passive resistance of the peat or alluvial materials above the level of the underlying clay should be ignored.

All excavation work should be conducted in accordance with the Ontario Occupational Health and Safety Act.

6.4.3 Road Approach Fill Construction

The subgrade areas under all proposed road approach fills should be stripped of all topsoil, organics and other deleterious material to expose the top of the underlying silty clay. As summarized in Table 2, topsoil thicknesses in the order of 150 to 300 mm are expected, although locally thicker deposits of organic material may be encountered at the toe area of the existing slopes. The exposed subgrade should be proof rolled, and any obvious soft spots subexcavated.

Embankment fill material may consist of any locally available inorganic materials as approved by the engineer. Fill should be placed in 150 mm lifts at placement water content of -2 to +2% of its optimum water content and compacted to achieve at least 95% of its Standard Proctor Maximum Dry Density throughout, increasing to 98% in the upper 1 m below the future pavement. The fill should be overbuilt to accommodate anticipated settlements.

6.4.4 Construction Inspection and Monitoring

The proposed works outlined within consist of the construction of load bearing elements in direct contact with the native soils. Accordingly, all foundation works should be completed as per MTO Standards. Subgrade inspection and compaction control of the proposed approach embankments is also important.

6.4.5 Frost Depth

The estimated maximum depth of frost penetration at the site is 1.9 m. Accordingly, the base of all proposed pile caps should be provided with a minimum of 1.9 m of earth cover or equivalent insulation.

6.4.6 Floodplain Access Road Construction

Construction of access roads through the central floodplain area to permit construction at the proposed pier locations is anticipated. Given the high water table in this area, the use of coarse granular material placed on top of a non-woven geotextile such as Terrafix 360 R (or similar) is recommended for the access road construction. However, settlement and compression of the underlying peat and alluvium deposits should be anticipated, and regular upgrading of these roads will be required. Road performance through this area will be greatly enhanced by the incorporation of a reinforcing geogrid.

In light of the environmentally sensitive nature of the wetland, it is recommended that the access roads be constructed during the winter by placing the construction materials on top of the frozen surface of the floodplain which will settle into place during spring break-up. This approach will offset potential development of mud waves in front of and to the side of the access roads if conventional end dumping procedures are employed.

6.4.7 Seismic Design

The estimated zonal velocity ratio (v) from the Ontario Building Code for Arnprior, Ontario is 0.10. Accordingly, for the proposed multi-span bridge at this location, seismic design forces related to the above zonal velocity ratio should be included. It is anticipated that this can be completed using the equivalent static load method with an appropriate response coefficient (C) for a v value of 0.10 as shown on Figure 2-4.8.2 of the Ontario Highway Bridge Design Code. However, in the determination of the parameter "P" in the equivalent static analysis, lateral pile resistances provided earlier should be factored by 2 to account for the short term nature of earthquake loads.

Based on discussion with the Geological Survey of Canada, the Arnprior area is considered to be tectonically stable, and additional seismic analysis related to the reported fault that passes close to the proposed site is not considered necessary.

6.5 Environmental Considerations

It is understood from discussions with MNR personnel, that the east floodplain area of the site has been designated as a Class 1 wetland primarily because it acts as spawning habitat for northern Pike in the springtime. The Mississippi River is a popular fishing location and accordingly, the MNR are very concerned about any potential degradation to the fish stock, on either a temporary and/or a long term basis. In light of this requirement, careful consideration must be given to the development of construction procedures to achieve this objective. During design, a Non Standard Special Provision (NPSS) should be developed in conjunction with MTO's Regional Environmental Unit to satisfactorily address the MNR's concerns.

Conventional cast in place construction techniques requiring extensive temporary formwork would most likely result in disturbance to most of the floodplain along both rights of way. For this reason, it is understood that an incrementally launched bridge design is being considered. However, this proposed approach will still require temporary access to the floodplain area to permit construction of the piers, most likely in the form of a temporary access road. Given the extent of the works required at the piers, and average climatic conditions in the area, restricting floodplain construction to winter only would not appear to be feasible.

As discussed in Section 6.4.6, winter access road construction by placing of materials on the frozen surface of the floodplain is anticipated to create significantly less disturbance to the floodplain than traditional end dumped construction. However, compression of the floodplain deposits immediately beneath the footprint area of the road will occur. The suitability of this area as spawning habitat after access road removal should be assessed by a suitably qualified aquatic biologist.

One major disadvantage likely to be perceived by the MNR regarding construction of an access road across the entire width of the floodplain, is the interruption of water flow and physical interruption to movement of fish and other wildlife. For this reason, consideration should be given to mobilizing all equipment and materials to the pier locations adjacent to the main river channel (Pier No. 1) using a barge, with an access road to the location of Pier Nos. 2 and 3. This approach will reduce the length and footprint area of access roads.

However, these and other potential solutions should be further pursued with the MNR, including the potential for stocking of the river to mitigate temporary loss of spawning habitat. Over the long term, provided the access road is removed and all construction works are completed in a careful manner, it is anticipated that the floodplain area should recover to its present state.

It should be noted that one of the major causes for loss of fish habitat is related to downstream siltation of spawning areas as opposed to physical disturbance. For this project, the amount of earthworks within the floodplain area will be very limited and downstream siltation should not be a major concern. However, appropriate siltation control procedures must be included in the design.

7.0 CONCLUSIONS AND RECOMMENDATIONS

- 1) Subsurface investigations at the site of the proposed Highway 417/Mississippi River Bridges confirm the presence of deep deposits of weak sensitive silty clay from just below ground surface to the underlying bedrock some 45 m below the level of the floodplain area at approximate elevation 40 m.
- 2) Groundwater conditions at the site consist of standing water in the floodplain area, artesian water pressure close to bedrock, and pore water pressures within the silty clay deposits that are greater than hydrostatic with respect to the standing water level in the floodplain area.
- 3) The currently proposed bridge configuration involves placing of up to 10 m high abutment fills in the river valley. Placing 10 m high fills in the river valley will cause environmental and stability concerns. Furthermore, settlement estimates indicate that unacceptably high settlements (greater than 2 m) would result from such fill heights and this level of settlement can not be accommodated in bridge design. The report therefore recommends lengthening the bridge so that abutments are relocated to the flat tableland area avoiding the need to place high fills in the river valley. The bridge should be lengthened and lightweight fill materials utilized for embankment construction to an extent which will limit abutment fill contact pressures to not greater than 60 kPa.

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- 4) Even with the lengthened bridge and reduced fill height, it is essential to limit abutment fill settlements to no more than 50 mm. Accordingly, in order to accelerate a significant part of the settlements prior to abutment foundation construction, it is recommended that the abutment fills be constructed at least one year in advance of foundation construction and that the abutment fill area be preloaded and surcharged and provided with wick drains to accelerate the dissipation of foundation pore pressures. The foundation soil settlement at the abutments should be monitored during preloading to confirm that a significant part of the settlement is completed prior to foundation installation. Once preloading is completed, the preload fill should be removed and the final abutment fill should be constructed with light weight polystyrene blocks with an adequate earth cover for frost protection. These recommendations are expected to reduce the future settlement potential at the abutments to less than 50 mm.
 - 5) All road approach fills that apply greater than 50 kPa contact pressure should be preloaded and surcharged with at least 40 kPa of additional stress and constructed at least one year in advance of paving. No foundation wick drains are required for road approach fills.
 - 6) All abutment and approach fills should be built at side slopes no steeper than 2H:1V.
 - 7) All bridge piers and abutments should be founded on Steel H Piles driven to bedrock. If the abutment fills are constructed one year in advance following the recommendations given above to reduce the amount of post construction settlement, then negative skin friction or downdrag forces on the abutment piles are not of concern.
 - 8) Construction of temporary access roads through the floodplain area during the winter is recommended to reduce disturbance of the floodplain deposits. During design, Non Standard Special Provision should be developed in conjunction with MTO's Regional Environmental Unit.

8.0 REFERENCES

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TABLE 1
HIGHWAY 417 / MISSISSIPPI RIVER BRIDGES
Arnprior, Ontario
BOREHOLE SUMMARY

BOREHOLE NUMBER	NORTHING (m)	EASTING (m)	GROUND ELEVATION (m)	DEPTH (m)	PIEZOMETER TIP ELEVATIONS	
					SHALLOW (m)	DEEP (m)
GEOTECHNICAL BOREHOLES						
95-1	5026872.5	323474.5	91.45	16.46		78.3
95-2	5026844.1	323500.1	86.92	49.10		40.7
95-3	5026768.0	323560.5	84.30	47.52	82.8	38.6
95-4	5026727.7	323612.9	82.93	47.52	82.0	64.0
95-5	5026671.4	323647.5	83.02	49.05	82.0	71.2
95-6	5026631.1	323699.9	83.69	49.91		36.9
95-7	5026603.7	323708.3	88.06	16.46		73.2
95-8	5026836.3	323460.5	91.10	16.46		75.8
95-9	5026810.4	323489.6	87.16	49.17		40.7
95-10	5026745.3	323542.8	84.42	45.80	82.0	40.1
95-11	5026689.0	323577.4	83.10	45.55	82.0	61.4
95-12	5026648.7	323629.8	83.04	47.35	82.0	68.3
95-13	5026597.4	323663.9	83.45	48.92		38.1
95-14	5026580.1	323683.5	88.35	16.46		73.4
.STEEL DRIVE POINT PIEZOMETERS						
Top of PipeLength of Pipe						
P-1	5026815	323661	83.9		1.98	
P-2	5026791	323683	83.8		1.98	
P-3	5026742	323726	83.8		1.98	
P-4	5026626	323508	83.9		1.98	
P-5	5026578	323551	83.7		1.98	
P-6	5026536	323588	83.8		1.98	

TABLE 2
HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
Arnprior, Ontario
BOREHOLE STRATIGRAPHIC SUMMARY

Borehole #	Total Depth (m)	Topsoil Thickness (mm)	Stratigraphic Upper Elevations (m) (Layer Thickness in Brackets)					
			Floodplain	Silty Clay Deposits			Glacial Till	Bedrock
				Upper Crust	Firm to Stiff Upper Clay	Stiff to Very Stiff Lower Clay		
95-1	16.5	150	np	91.3 (4.5)	87.0 [12.0]	ne	ne	ne
95-2	49.1	150	np	86.7 (4.1)	82.6 (20.1)	62.5 (21.7)	ne	40.8 [3.0]
95-3	47.5	np	84.3 (4.3)	np	80.0 (20.1)	59.9 (20.9)	ne	39.0 [2.2]
95-4	47.5	np	83.0 (3.8)	np	79.2 (21.6)	57.6 (18.9)	ne	38.7 [2.7]
95-5	49.1	np	83.0 (3.1)	np	79.9 (20.9)	59.0 (21.6)	ne	37.4 [3.0]
95-6	49.9	225	np	83.2 (1.8)	81.4 (21.9)	59.3 (21.0)	38.3 (1.5)	36.8 [3.0]
95-7	16.5	150	np	88.0 (5.0)	83.0 [11.4]	ne	ne	ne
95-8	16.5	150	np	91.0 (4.0)	87.0 [12.4]	ne	ne	ne
95-9	49.2	150	np	87.0 (3.4)	83.6 (23.8)	59.8 (19.3)	ne	40.5 [2.5]
95-10	45.8	np	83.4 (4.9)	np	78.5 (19.4)	59.1 (19.1)	ne	40.0 [2.3]
95-11	45.5	np	83.1 (3.6)	np	79.5 (21.9)	57.6 (17.4)	ne	40.2 [2.2]
95-12	47.4	np	83.1 (3.8)	np	79.3 (17.3)	62.0 (23.0)	ne	39.0 [2.9]
95-13	48.9	150	np	83.4 (1.7)	81.7 (22.3)	59.4 (20.5)	38.9 (1.0)	37.9 [3.0]
95-14	16.5	300	np	88.1 (4.7)	83.4 [11.4]	ne	ne	ne

Notes:

- 1) [] - square bracket indicates layer not fully penetrated
- 2) ne - layer not encountered
- 3) np - layer not present
- 4) All elevations are Geodetic.

TABLE 3
HIGHWAY 417 / MISSISSIPPI RIVER BRIDGES
Arnprior, Ontario
SUMMARY OF WATER LEVEL READINGS

BOREHOLE NUMBER	GROUND ELEVATION (m)	TIP ELEVATION (m)	WATER ELEVATION				
			DATE				
			Mar.14/95	Mar.18/95	Apr.12/95	Apr.13/95	May.24/95
WESTERN APPROACH AND ABUTMENTS							
95-1	91.45	78.3		88.40		89.61	89.47
95-2	86.92	40.7		88.00		91.10	90.11
95-8	91.10	75.8		82.40		89.04	89.05
95-9	87.16	40.7		88.69		88.86	88.78
EASTERN APPROACH AND ABUTMENTS							
95-6	83.69	36.9	>86.69			92.57	92.65
95-7	88.06	73.2	78.46			85.72	85.72
95-13	83.45	38.1	>86.45			90.83	90.94
95-14	88.35	73.4				85.80	85.79
CENTRAL PIERS							
95-3	84.30	38.6	>87.3		91.93		92.68
		82.8	83.83		83.11		83.21
95-4	82.93	64.0	85.06		84.95		85.53
		82.0	83.20		83.05		83.17
95-5	83.02	71.2	83.38		83.09		84.00
		82.0	80.83		83.71		83.09
95-10	84.42	40.1	85.54		85.73		85.65
		82.0	83.42		83.38		83.26
95-11	83.10	61.4	80.41		81.54		81.10
		82.0	83.08		83.28		83.15
95-12	83.04	68.3	79.76		83.11		83.79
		82.0	83.05		83.04		83.09
STEEL DRIVE POINT PIEZOMETERS							
Top of Pipe		Ice Thickness					
P-1	83.9		0.45			83.0	DRY
P-2	83.8		0.53		82.9		83.0
P-3	83.8		0.51		82.9		83.2
P-4	83.9		0.48		82.7		83.0
P-5	83.7		0.41		82.9		83.1
P-6	83.8		0.41			82.9	83.1

TABLE 4
HIGHWAY 417 / MISSISSIPPI RIVER BRIDGES
Arnprior, Ontario
SUMMARY OF SULPHATE CONTENT DETERMINATIONS

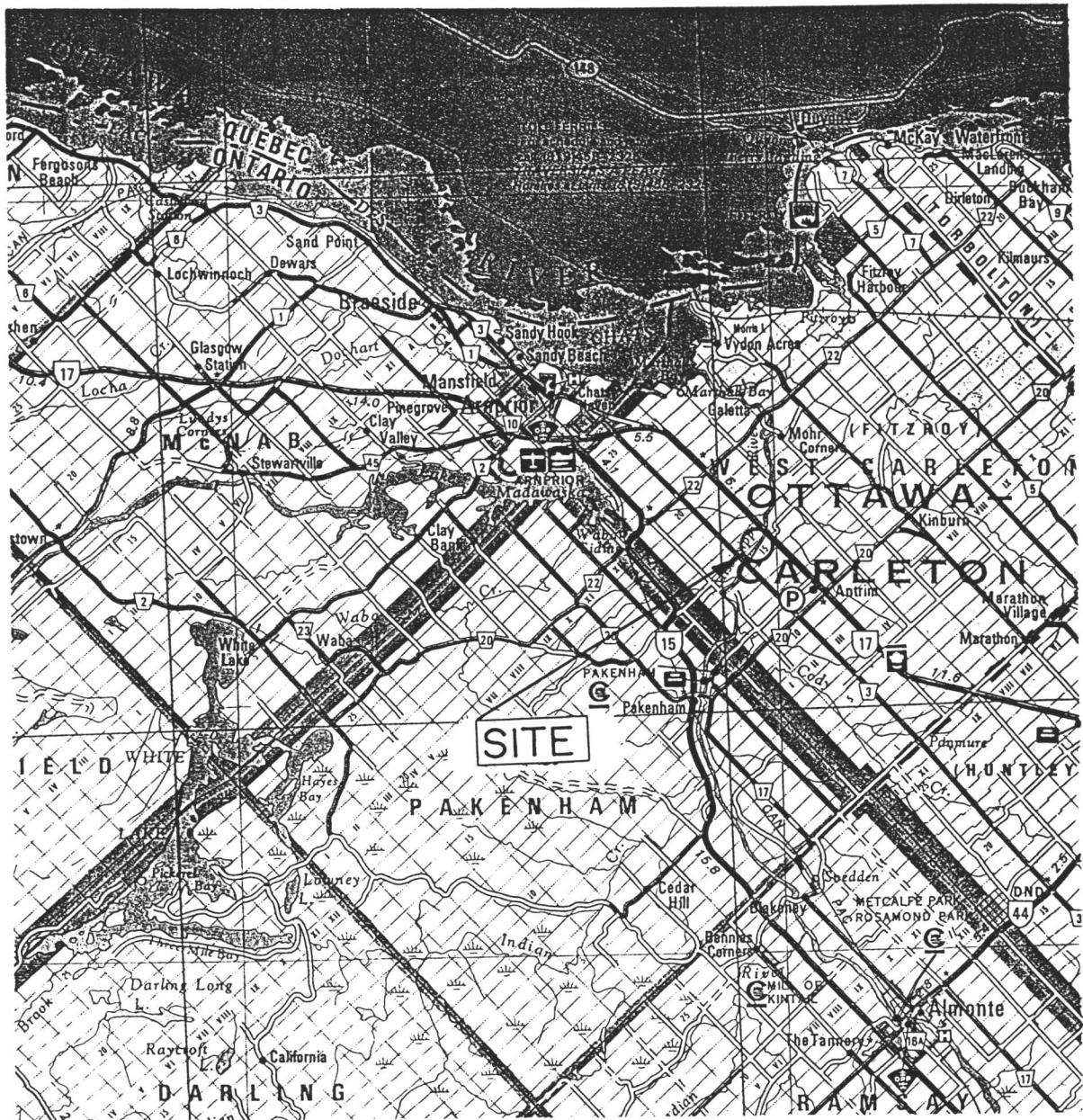
BOREHOLE NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (m)	SAMPLE ELEVATION (m)	SAMPLE TYPE	SULPHATE CONTENT (%)
95-2	2	1.52	85.40	Upper Clay Crust	0.060
95-3	3	2.29	82.01	Upper Alluvium	0.019
95-4	3	3.05	79.88	Upper Silty Clay	0.060
95-5	2	1.52	81.48	Peat	0.170
95-7	4	4.57	83.49	Upper Clay Crust	0.019
95-8	5	6.10	85.00	Upper Silty Clay	0.019
95-10	2	1.52	82.90	Upper Alluvium	0.019
95-11	3	3.05	80.05	Peat	0.096
95-12	3	3.05	79.99	Lower Alluvium	0.040
95-13	1	0.76	82.69	Upper Clay Crust	0.039

TABLE 5
HIGHWAY 417 / MISSISSIPPI RIVER BIDGES
Arnprior, Ontario
SUMMARY OF CONSOLIDATION TEST DATA

Test No.	Location Details				Index Properties					Pre-Consolidation Pressure (kPa)			Compression Data	
	West Approach Area													
	BH	SA #	Sample Depth (m) (Elev.)	P _o (kPa)	Nat'l W.C. %	P.I	% Silt	% Clay	Initial Void Ratio	Casagrande	Becker	OCR	Cr	Cc
1	95-8	4	4.8 (86.3)	55	68.9	33	32	68	1.829	180	186	3.3	0.053	1.18
2	95-1	7	9.4 (82.2)	87	77.3	36	23	77	2.099	140	135	1.5	0.070	1.71
3	95-1	9	12.4 (79.1)	106	65.6	34	26	74	1.817	180	159	1.5	0.055	1.72
	East Approach Area													
4	95-7	3	3.3 (84.8)	45	47.5	34	32	68	1.276	500	500	11.1	0.050	0.56
5	95-7	6	7.9 (80.2)	76	75.0	30	31	69	2.030	180	166	2.2	0.074	1.49
6	95-14	9	12.4 (76.0)	103	55.0	20	43	57	1.486	110	107	1.0	0.021	0.60

P'o - Existing effective overburden pressure

OCR - Over-consolidation ratio using Becker P'c construction



SCALE 1:250,000 Échelle

kilometres 5 0 5 kilomètres

DESIGNED	SP
DRAWN	SP
DATE	MAY, 1995
APPROVED	IC
SCALE	AS SHOWN

MINISTRY OF TRANSPORTATION, ONTARIO

MISSISSIPPI RIVER BRIDGES
GEOTECHNICAL INVESTIGATION
SITE LOCATION PLAN

Arnprior, Ontario

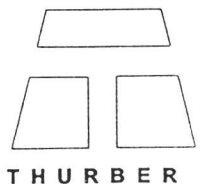
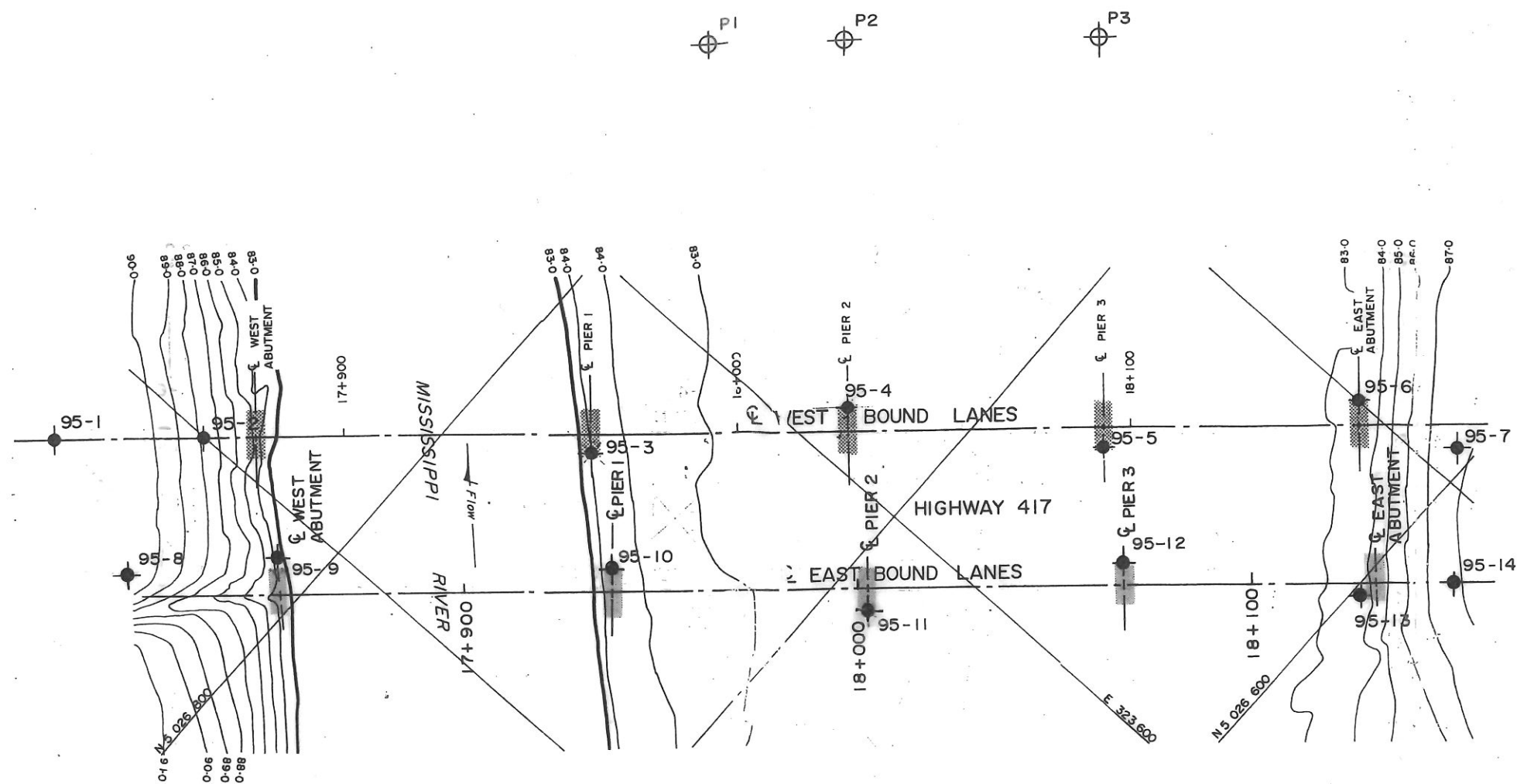


FIGURE I



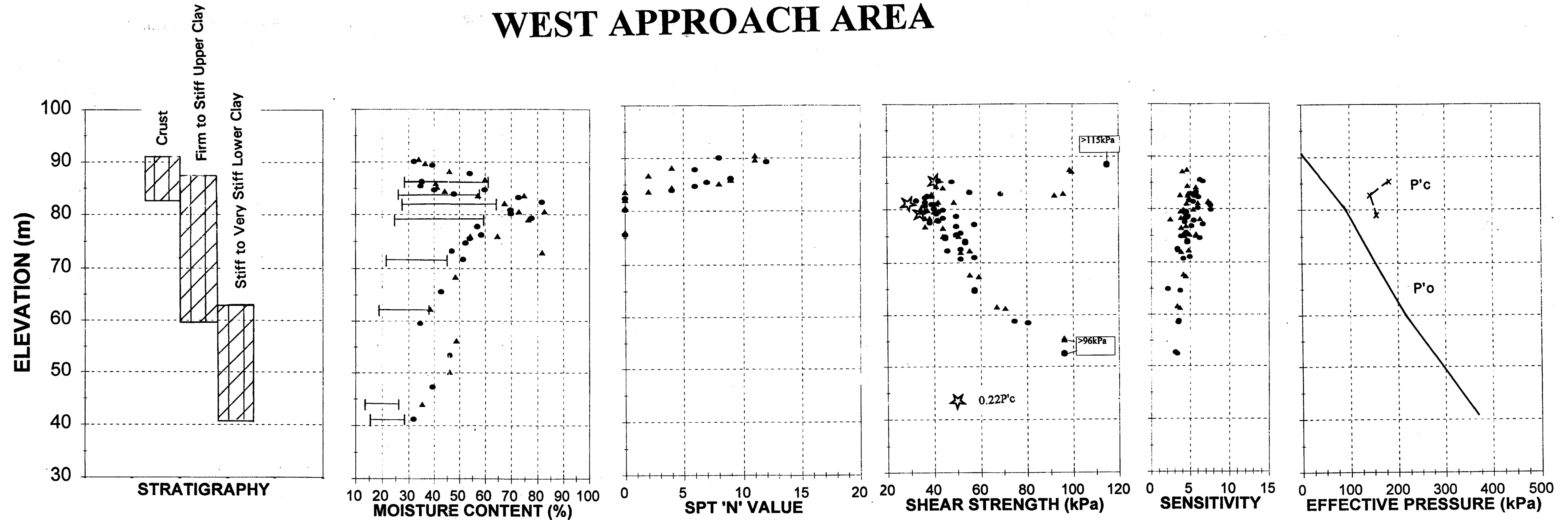
LEGEND

- BOREHOLE
- ⊕ DRIVE-POINT PIEZOMETER


SCALE
10 5 0 10 20m

DESIGNED	CB	MINISTRY OF TRANSPORTATION OF ONTARIO	 THURBER
DRAWN	DW	GEOTECHNICAL INVESTIGATION HIGHWAY 417, MISSISSIPPI RIVER BRIDGE	
DATE	MAY 1995		
APPROVED	IC	BOREHOLE LOCATION PLAN	
SCALE	AS SHOWN	Arnprior, Ontario	
			FIGURE 2

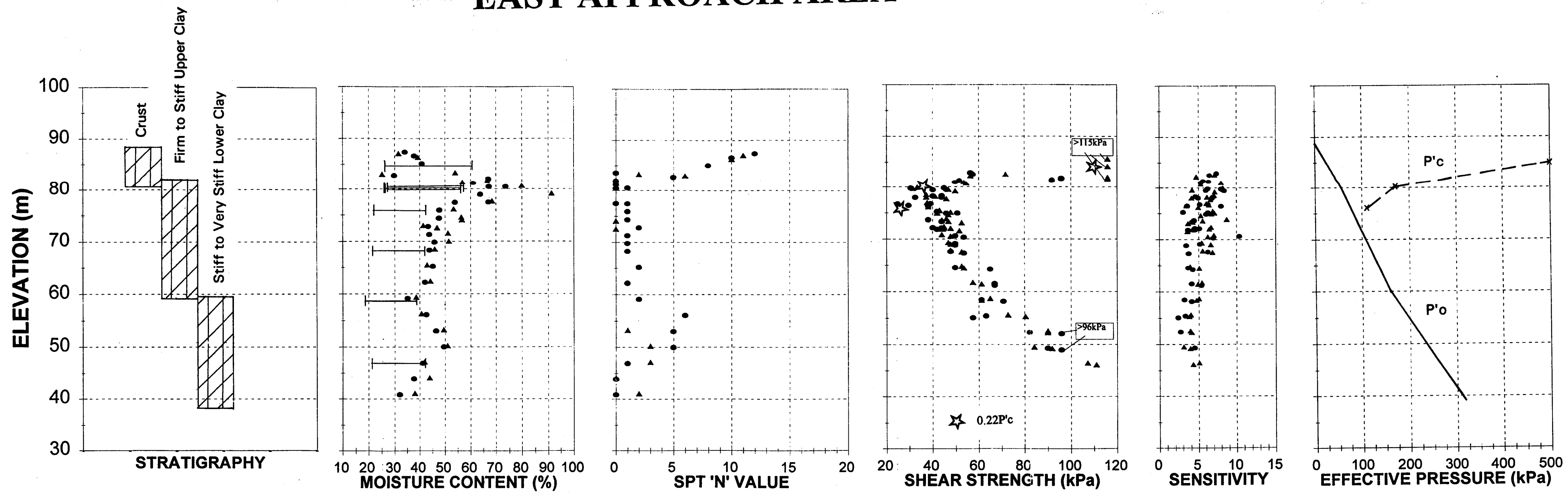
WEST APPROACH AREA




▲ WEST BOUND LANES • EAST BOUND LANES

DESIGNED	CB	MINISTRY OF TRANSPORTATION OF ONTARIO	 THURBER
DRAWN	CB	GEOTECHNICAL INVESTIGATION HIGHWAY 417, MISSISSIPPI RIVER BRIDGE	
DATE	MAY 1995		
APPROVED	IC	SUMMARY PLOT OF ENGINEERING PROPERTIES OF SILTY CLAY LAYER - WEST APPROACH AREA	
SCALE	AS SHOWN		
			FIGURE 3

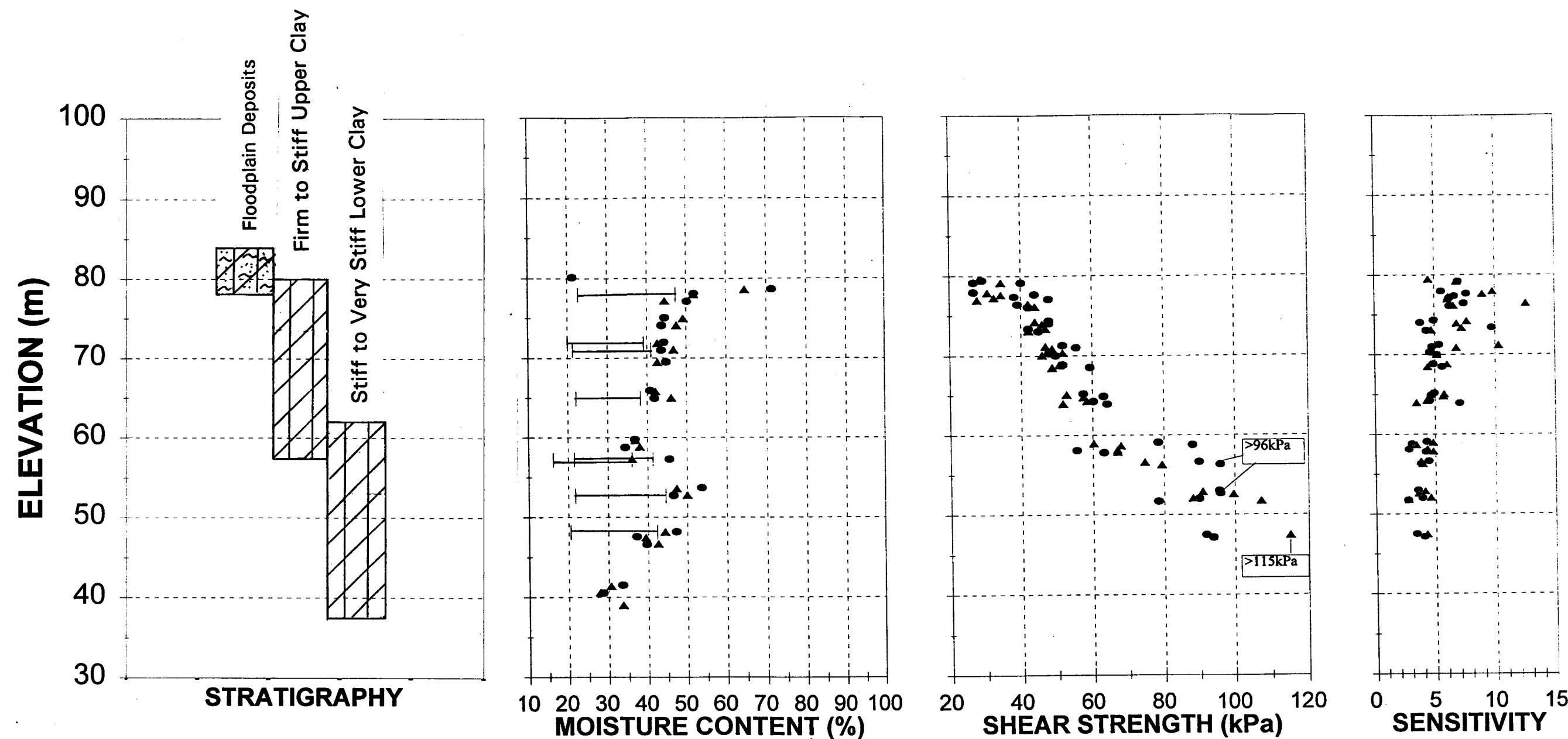
EAST APPROACH AREA



▲ WEST BOUND LANES • EAST BOUND LANES

DESIGNED	CB	MINISTRY OF TRANSPORTATION OF ONTARIO GEOTECHNICAL INVESTIGATION HIGHWAY 417, MISSISSIPPI RIVER BRIDGE SUMMARY PLOT OF ENGINEERING PROPERTIES OF SILTY CLAY LAYER - EAST APPROACH AREA	 THURBER
DRAWN	CB		
DATE	MAY 1995		
APPROVED	IC		
SCALE	AS SHOWN		
		FIGURE	4

CENTRAL FLOODPLAIN AREA



▲ WEST BOUND LANES • EAST BOUND LANES

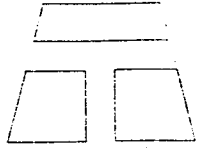
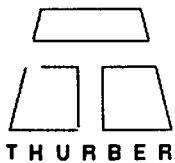
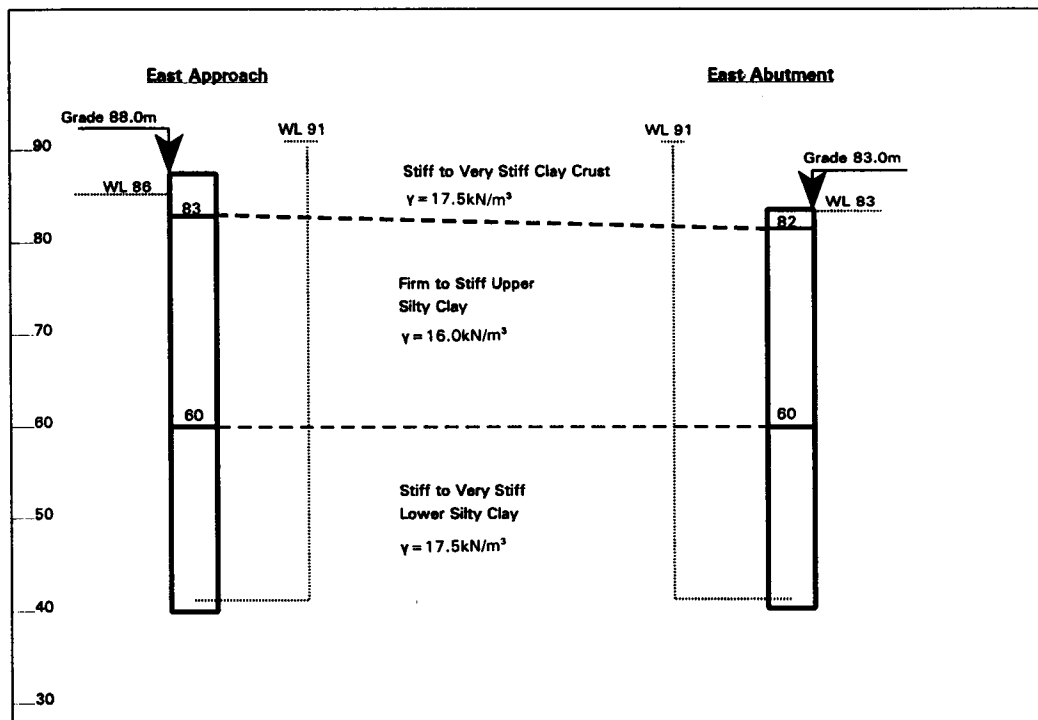
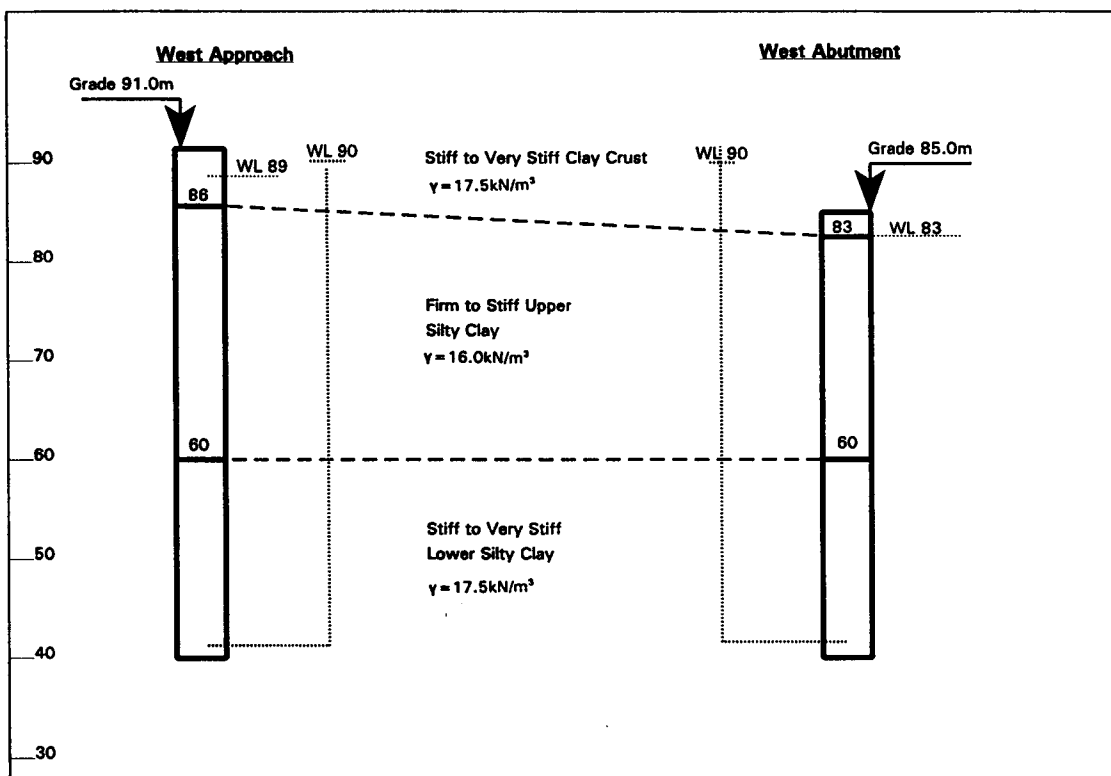
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DRAWN	CB		
DATE	MAY 1995		
APPROVED	IC		
SCALE	AS SHOWN		

FIGURE 5



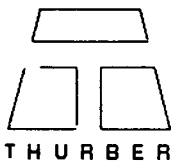
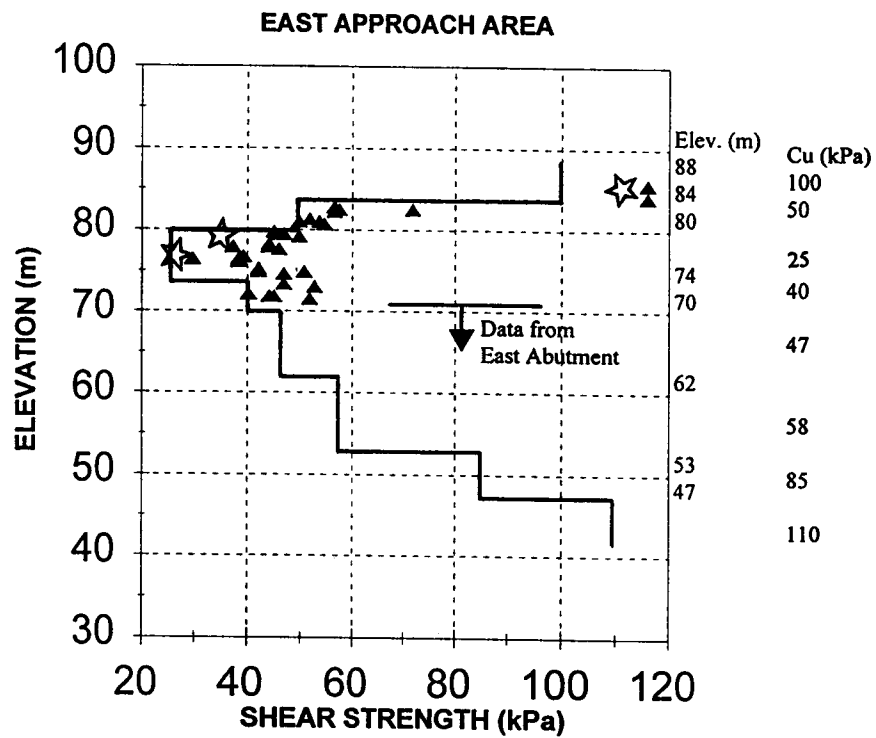
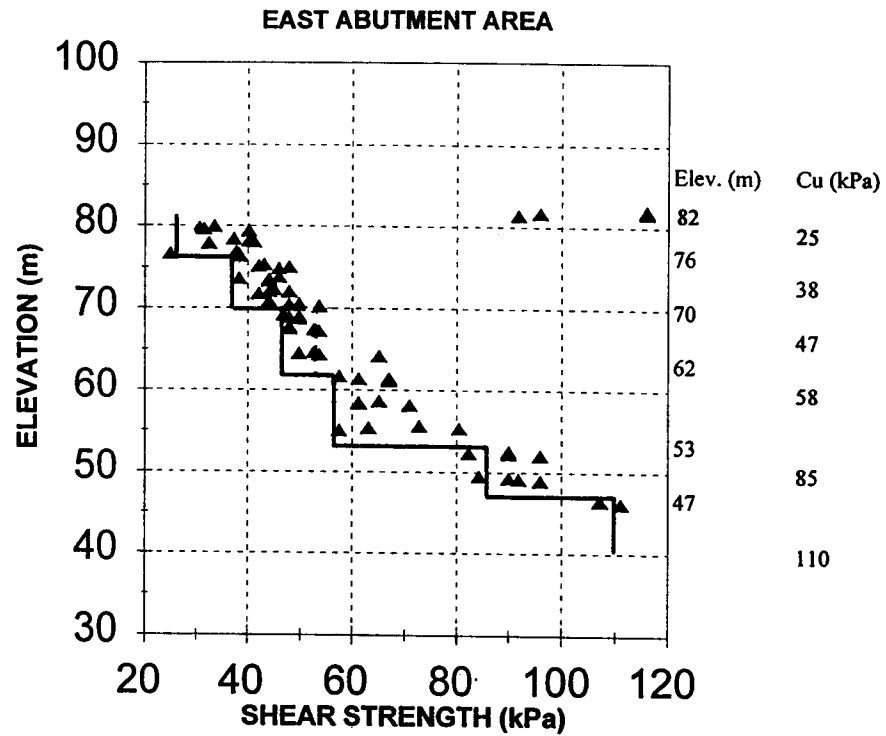
HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
SUMMARIZED DESIGN STRATIGRAPHY
EAST APPROACH AREA

FIGURE 6



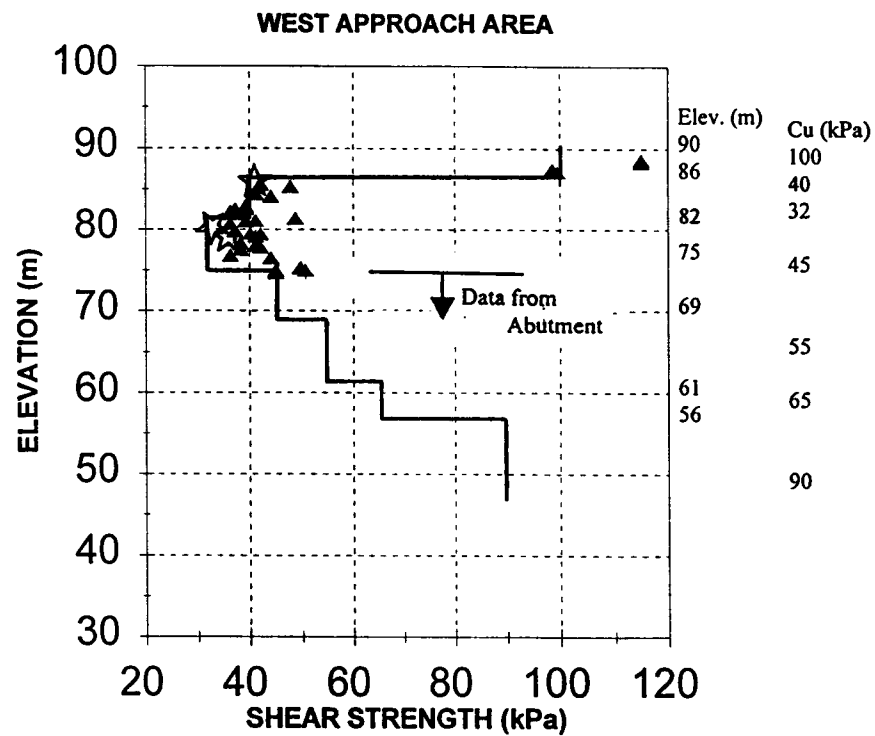
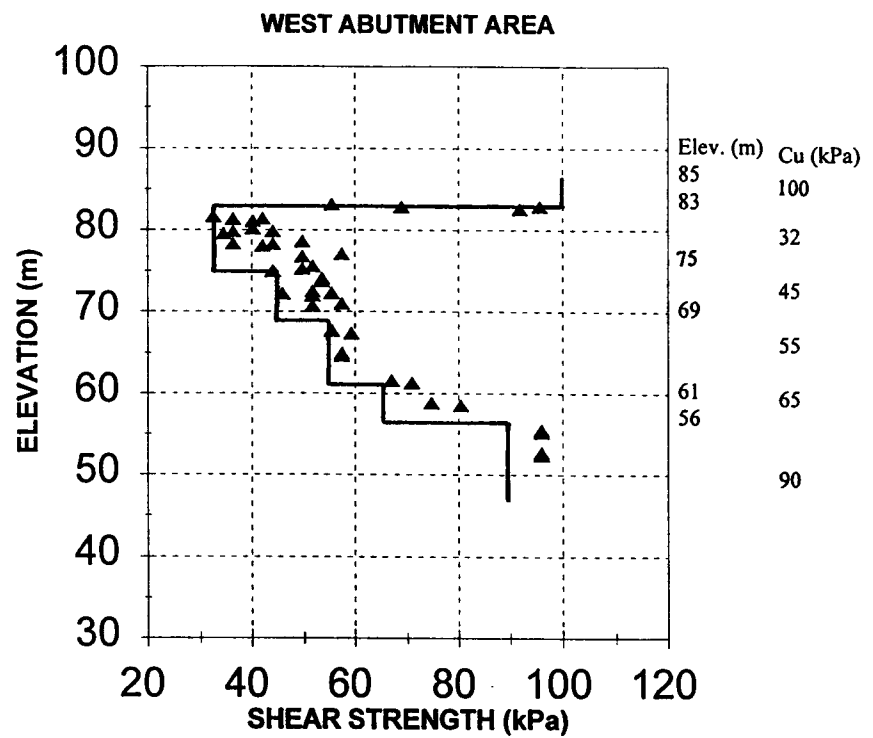
HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
SUMMARIZED DESIGN STRATIGRAPHY
WEST APPROACH AREA

FIGURE 7



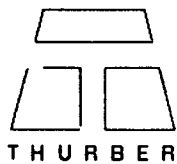
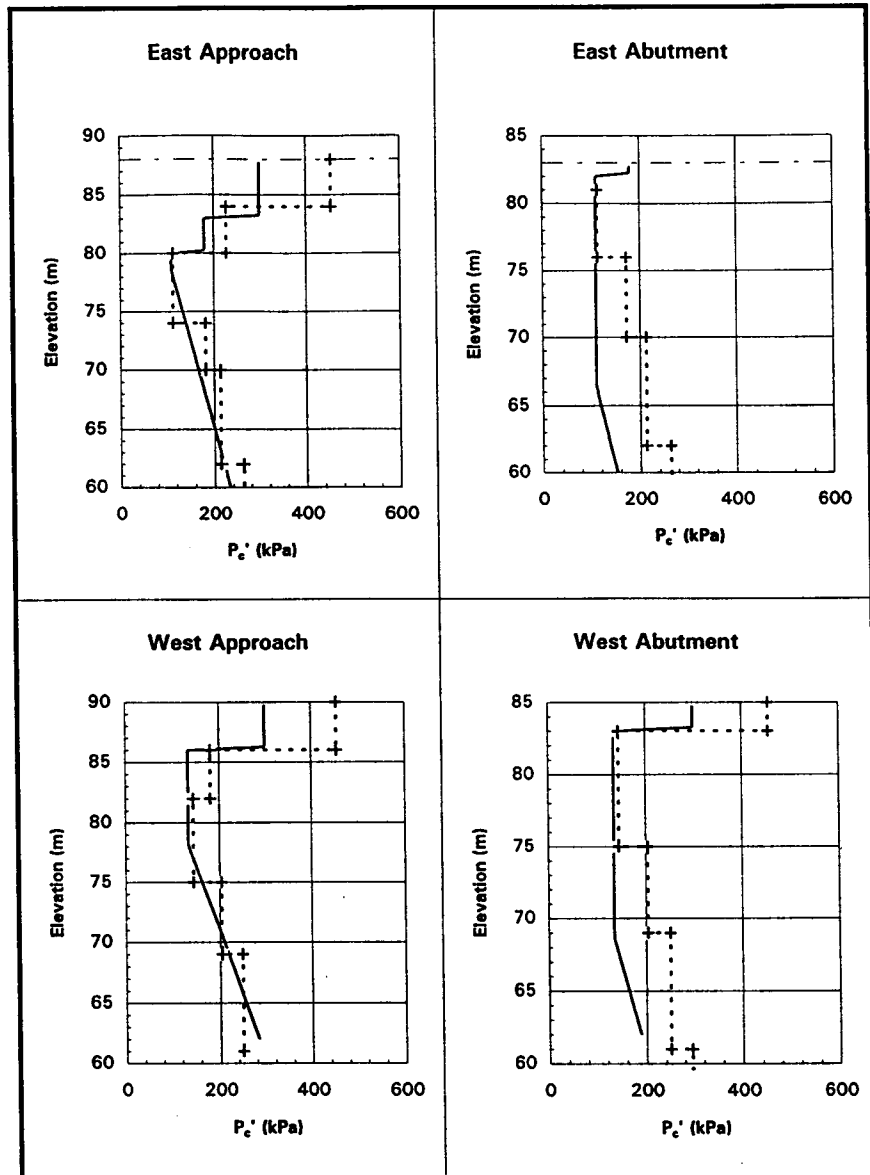
HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
DESIGN SHEAR STRENGTH ENVELOPES
EAST APPROACH AREA

FIGURE 8



HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
DESIGN SHEAR STRENGTH ENVELOPES
WEST APPROACH AREA

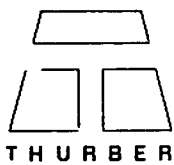
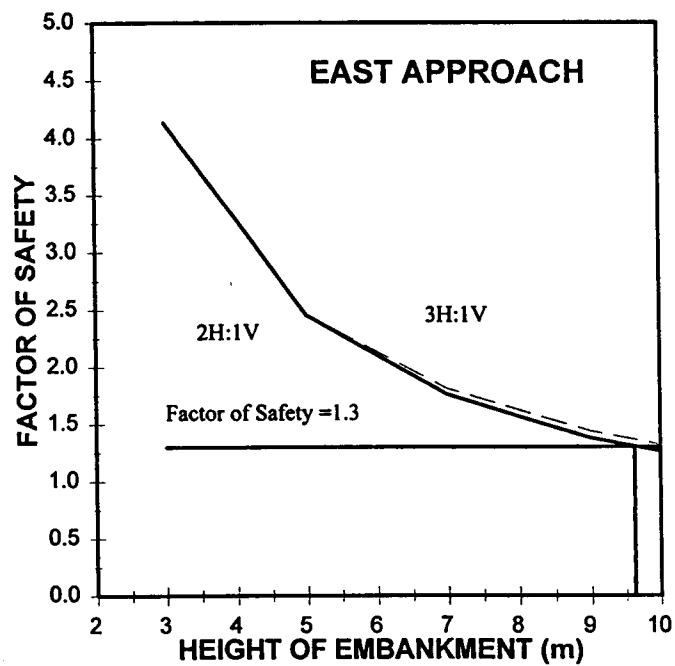
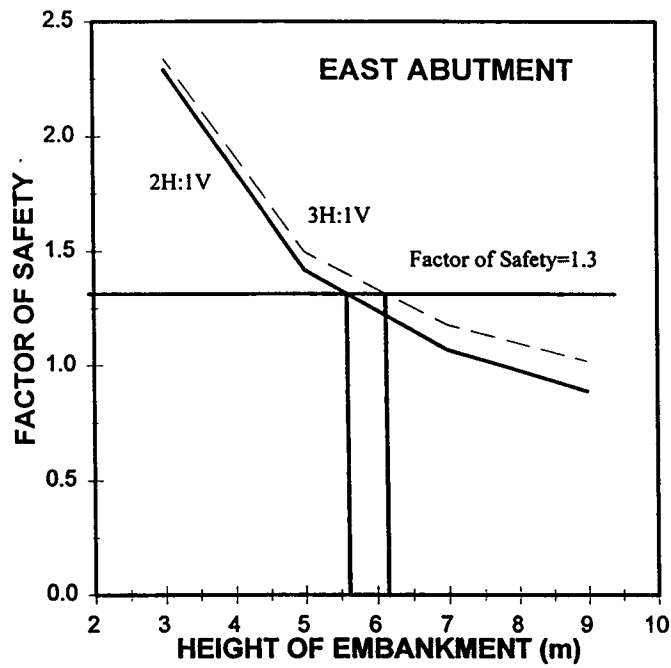
FIGURE 9



HIGHWAY 417/MISSISSIPPI RIVER BRIDGES

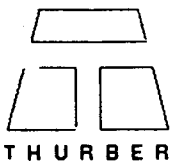
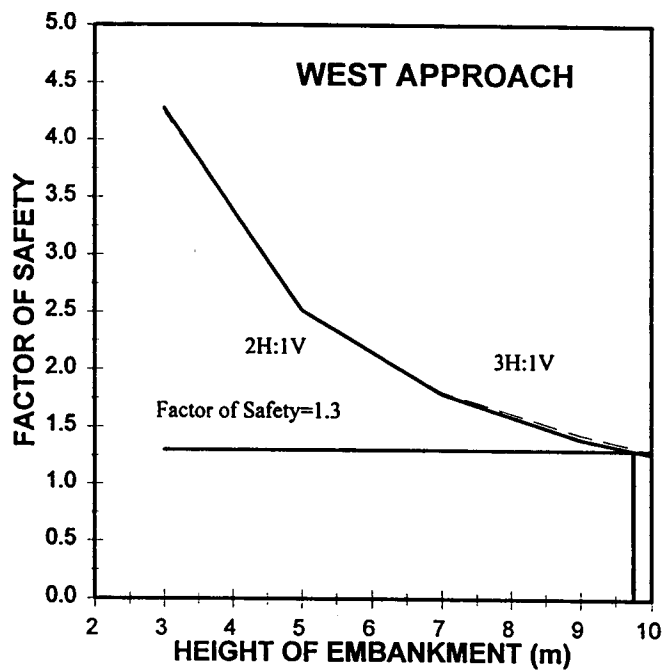
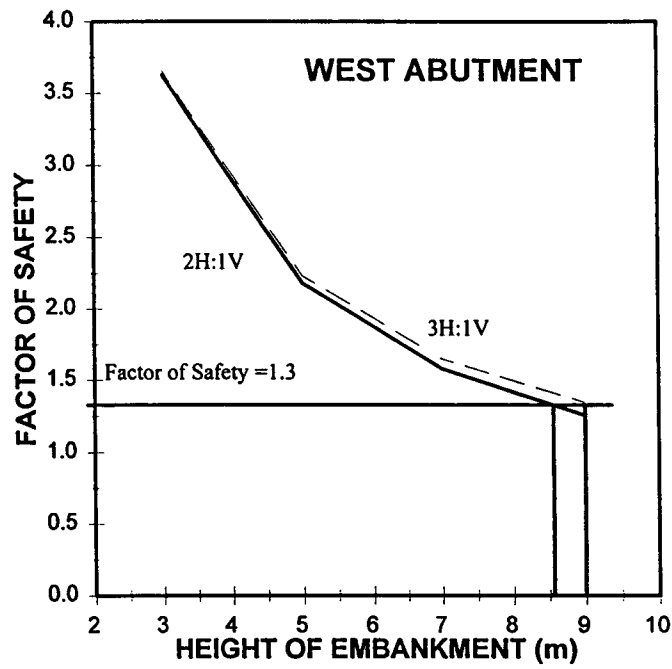
PRE-CONSOLIDATION DESIGN PROFILES

FIGURE 10



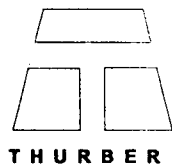
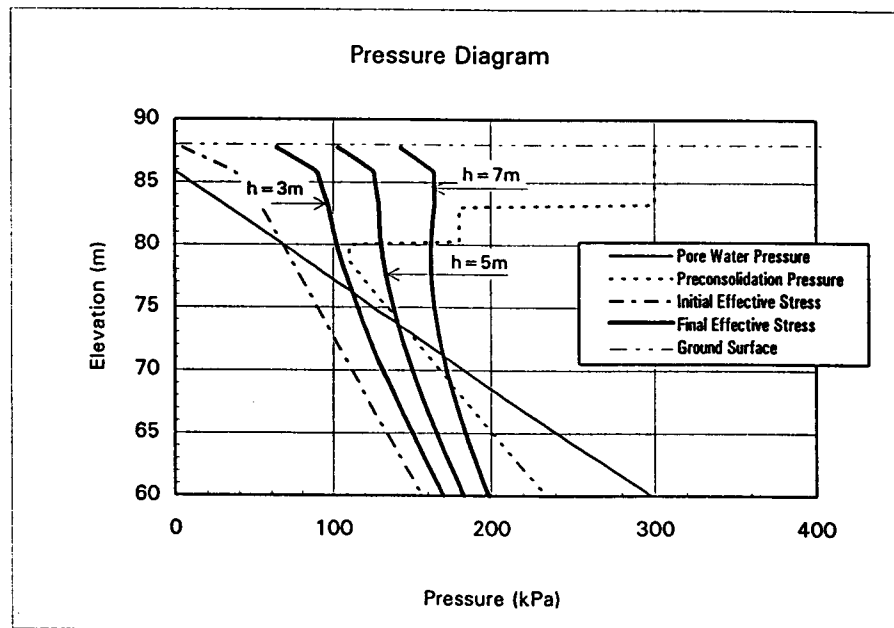
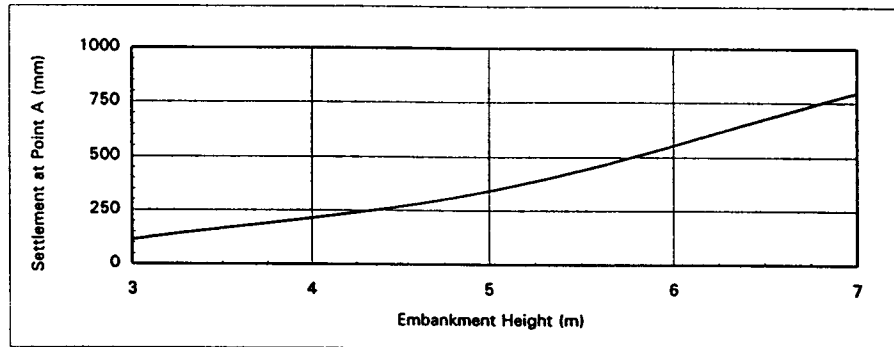
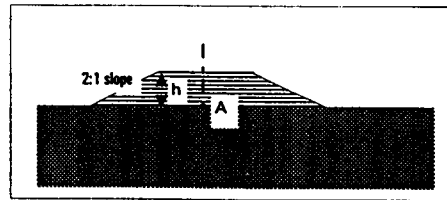
HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
SUMMARIZED STABILITY CALCULATIONS
EAST APPROACH AREA

FIGURE 11



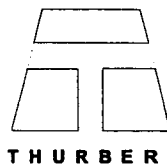
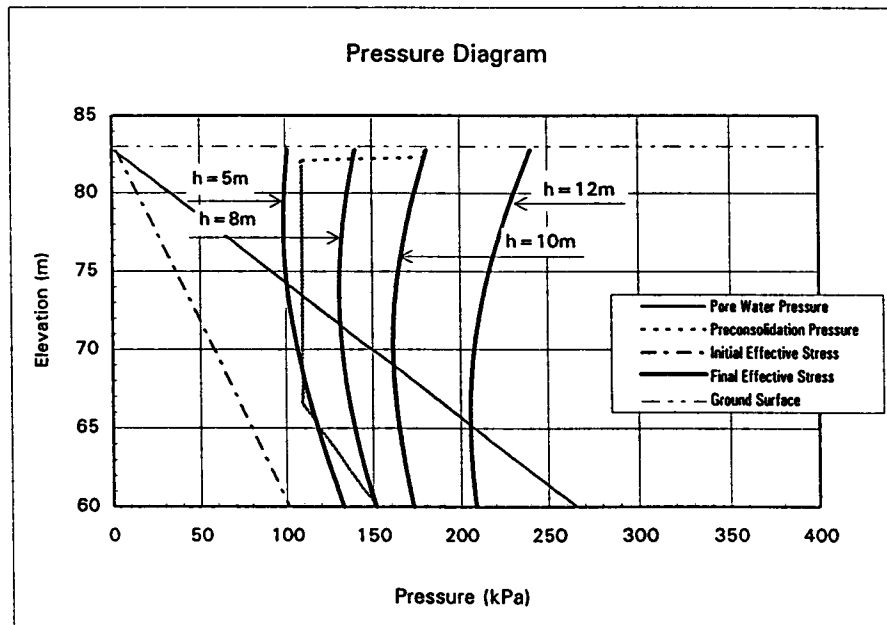
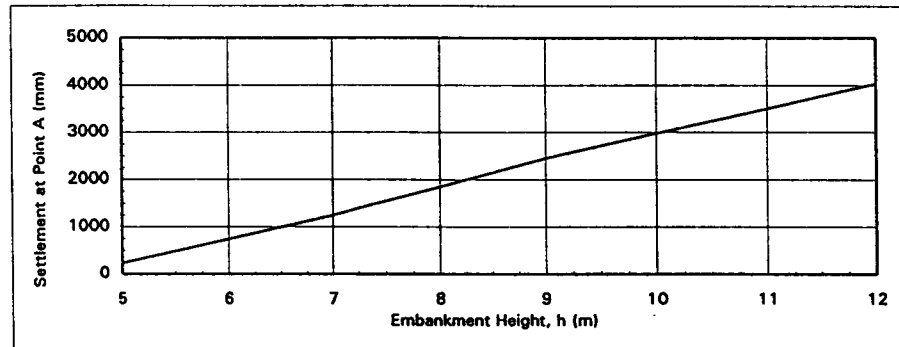
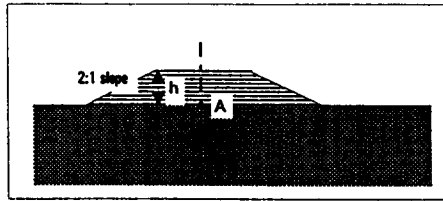
HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
SUMMARIZED STABILITY CALCULATIONS
WEST APPROACH AREA

FIGURE 12



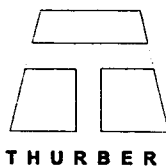
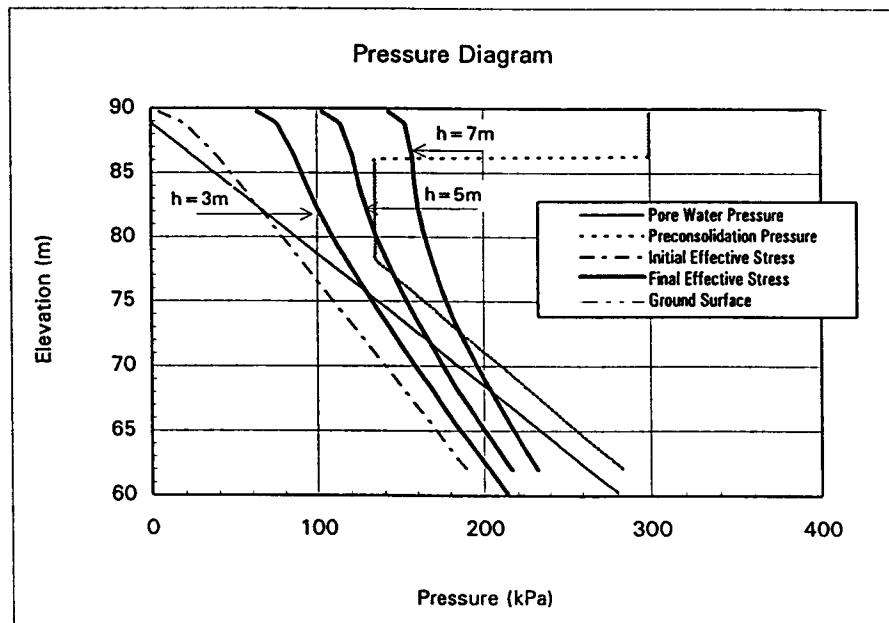
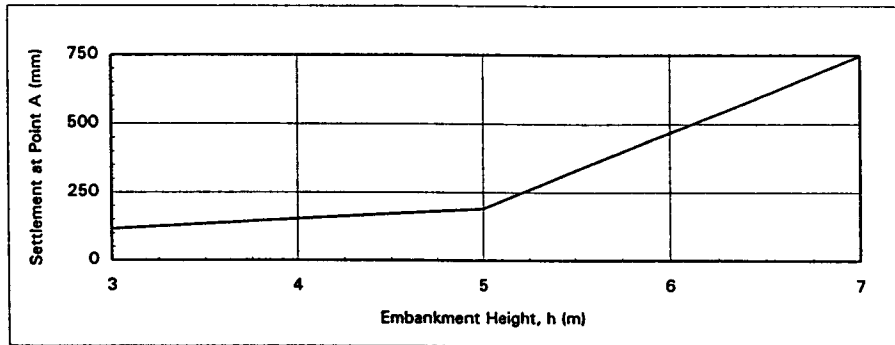
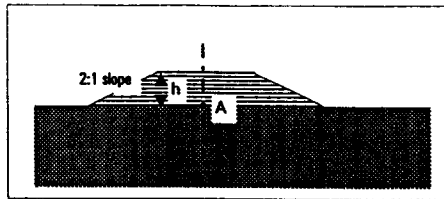
HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
SUMMARIZED SETTLEMENT CALCULATIONS
EAST APPROACH EMBANKMENT

FIGURE 13



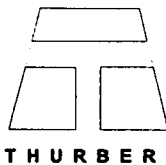
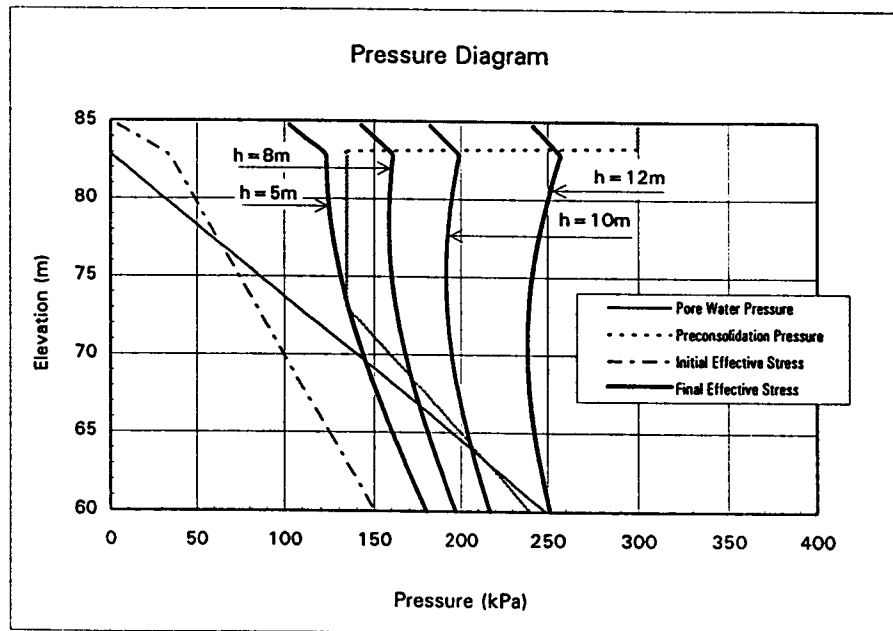
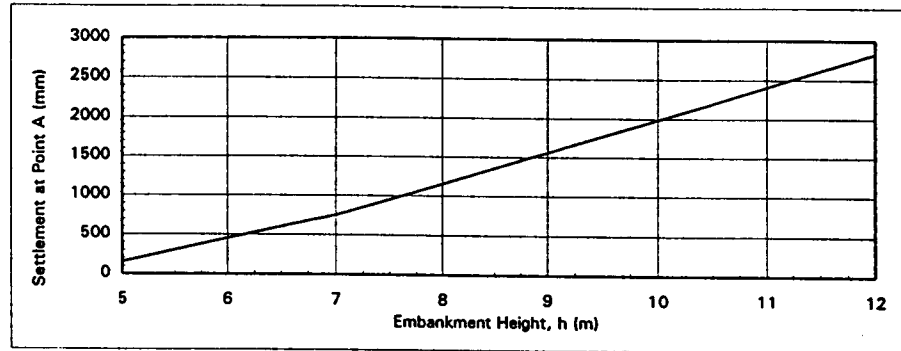
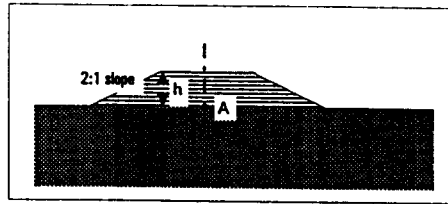
**HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
SUMMARIZED SETTLEMENT CALCULATIONS
EAST ABUTMENT EMBANKMENT**

FIGURE 14



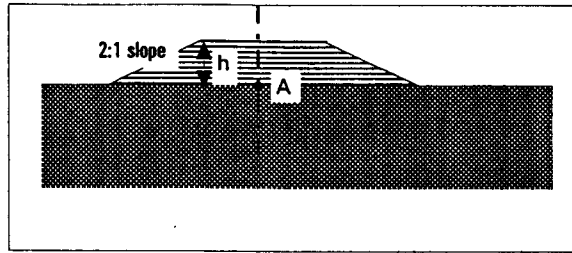
HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
SUMMARIZED SETTLEMENT CALCULATIONS
WEST APPROACH EMBANKMENT

FIGURE 15

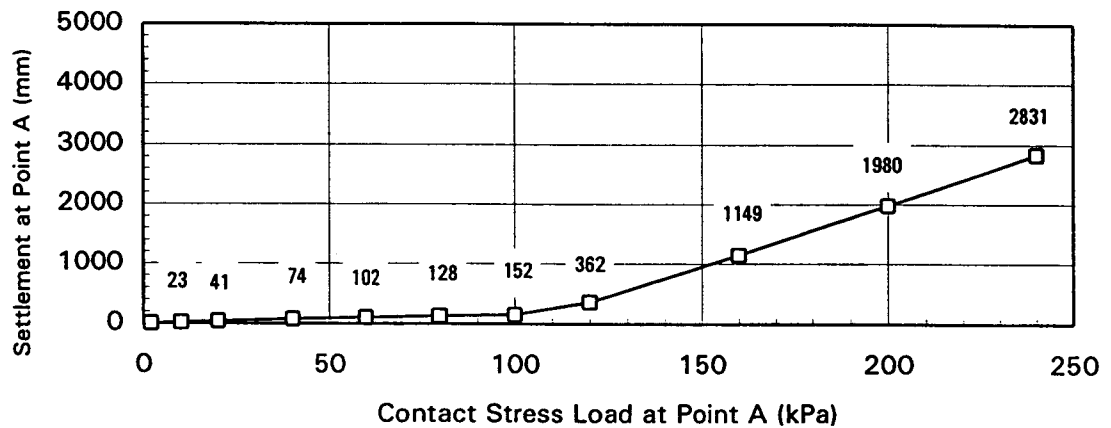


HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
SUMMARIZED SETTLEMENT CALCULATIONS
WEST ABUTMENT EMBANKMENT

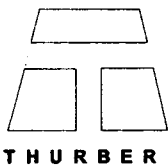
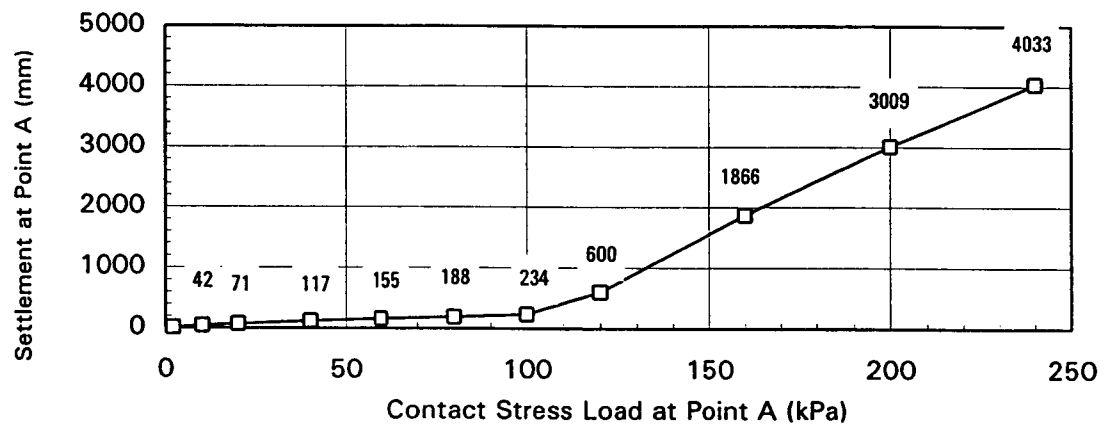
FIGURE 16



West Abutment Embankment

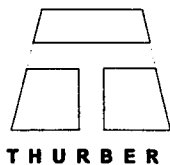
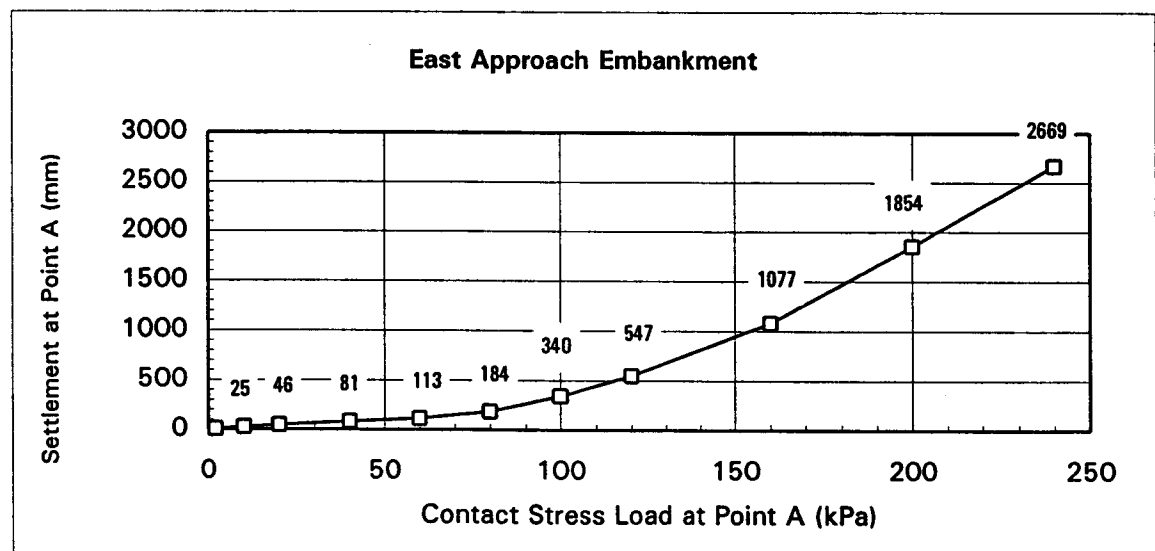
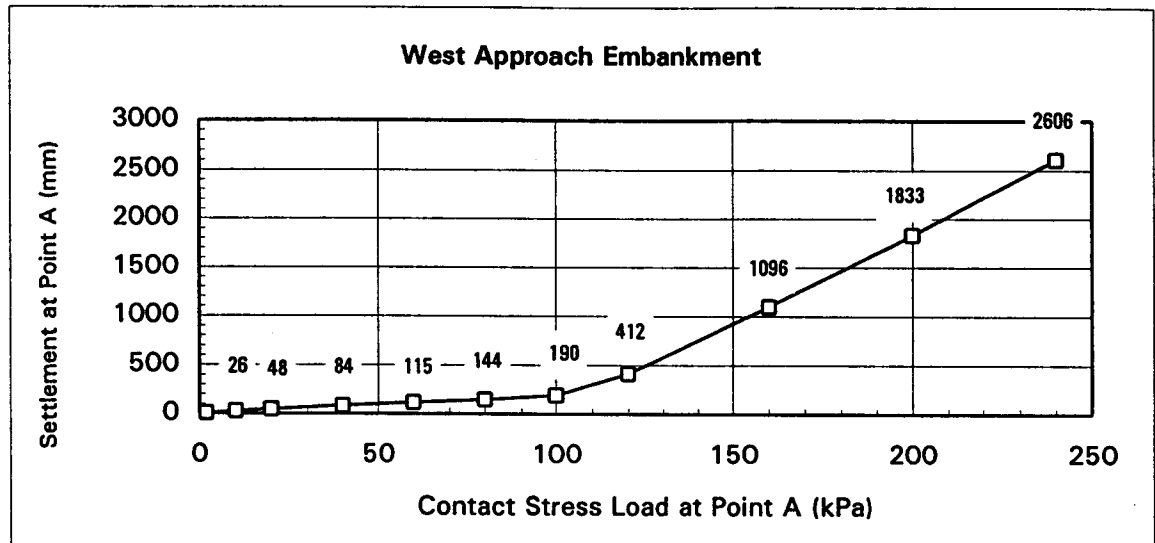
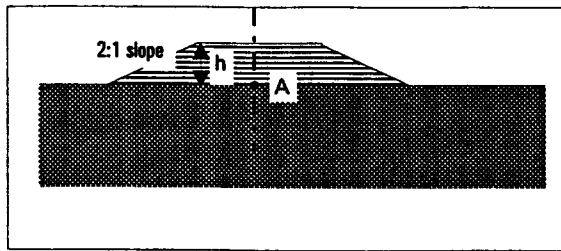


East Abutment Embankment



HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
ABUTMENT EMBANKMENT SETTLEMENT
Versus
FILL CONTACT PRESSURE

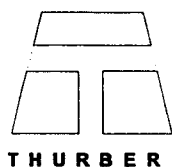
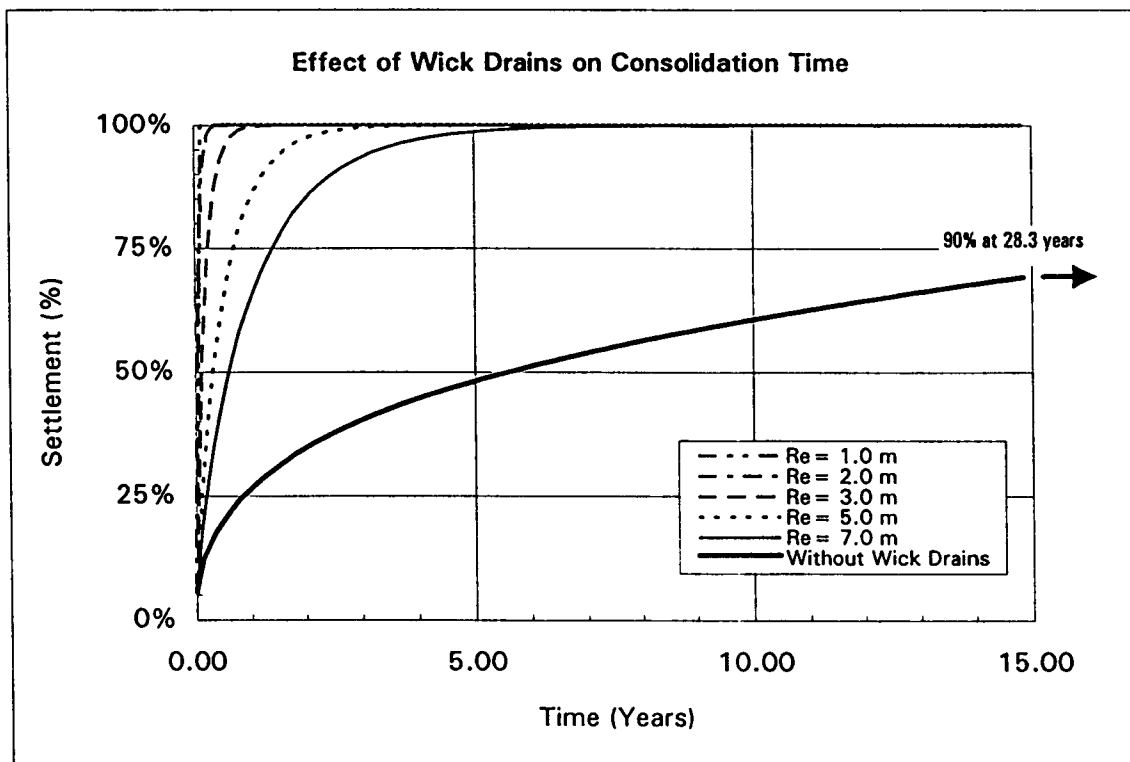
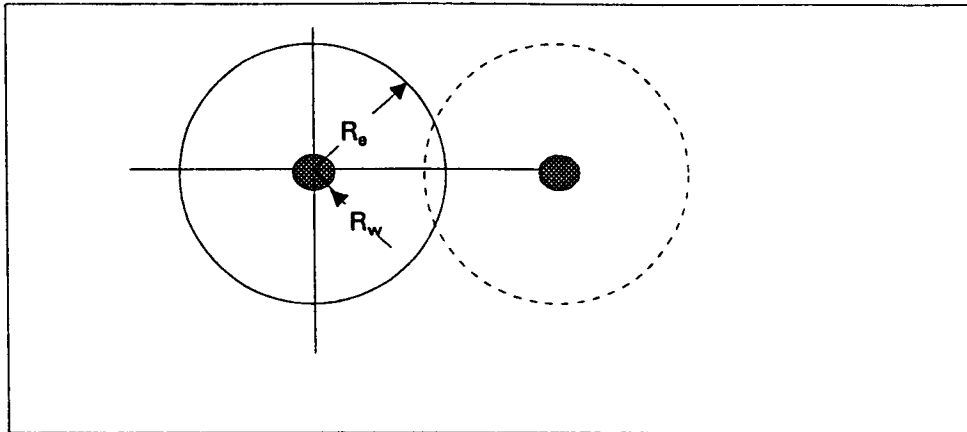
FIGURE 17



HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
APPROACH EMBANKMENT SETTLEMENT
Versus
FILL CONTACT PRESSURE

FIGURE 18

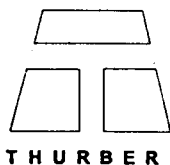
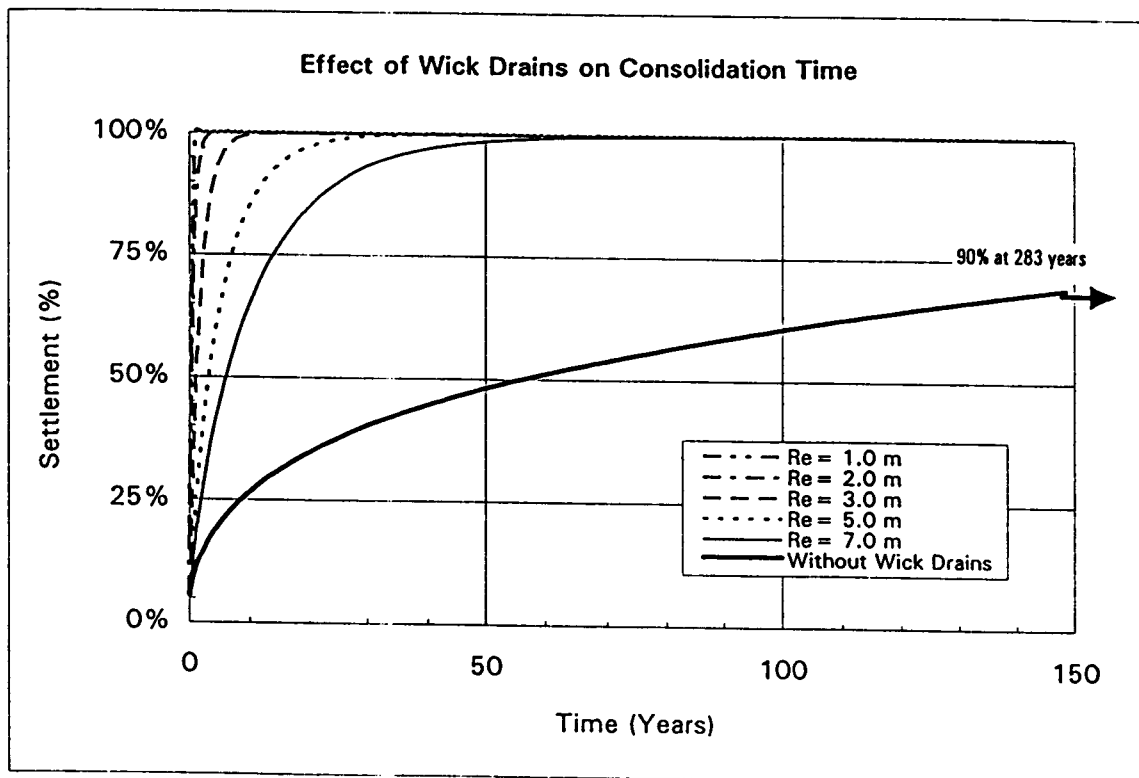
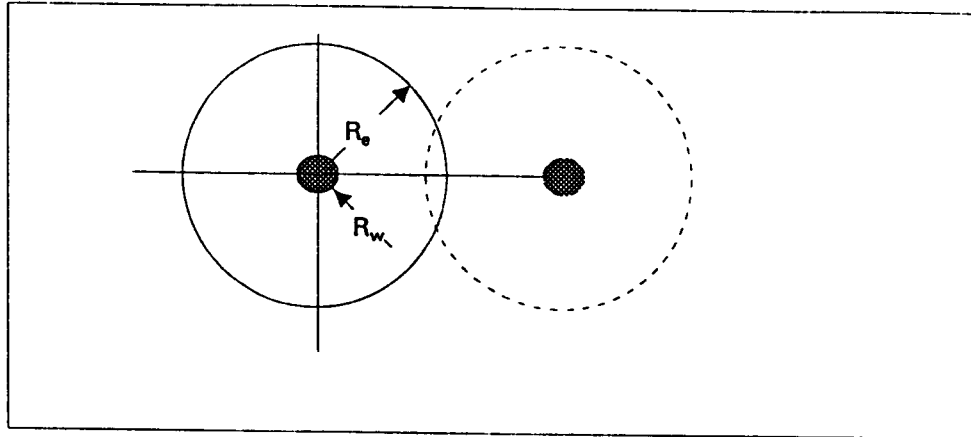
$C_{\text{radial}} = C_v$	5.00E-03 cm ² /sec
Wick Radius (R_w)	0.1 m
Wick Bottom Elevation	60.0 m



HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
RATE OF SETTLEMENT ANALYSIS
APPROACH EMBANKMENT FILLS
 $(C_v = 5 \cdot 10^{-3} \text{ cm}^2/\text{s})$

FIGURE 19

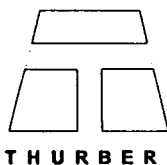
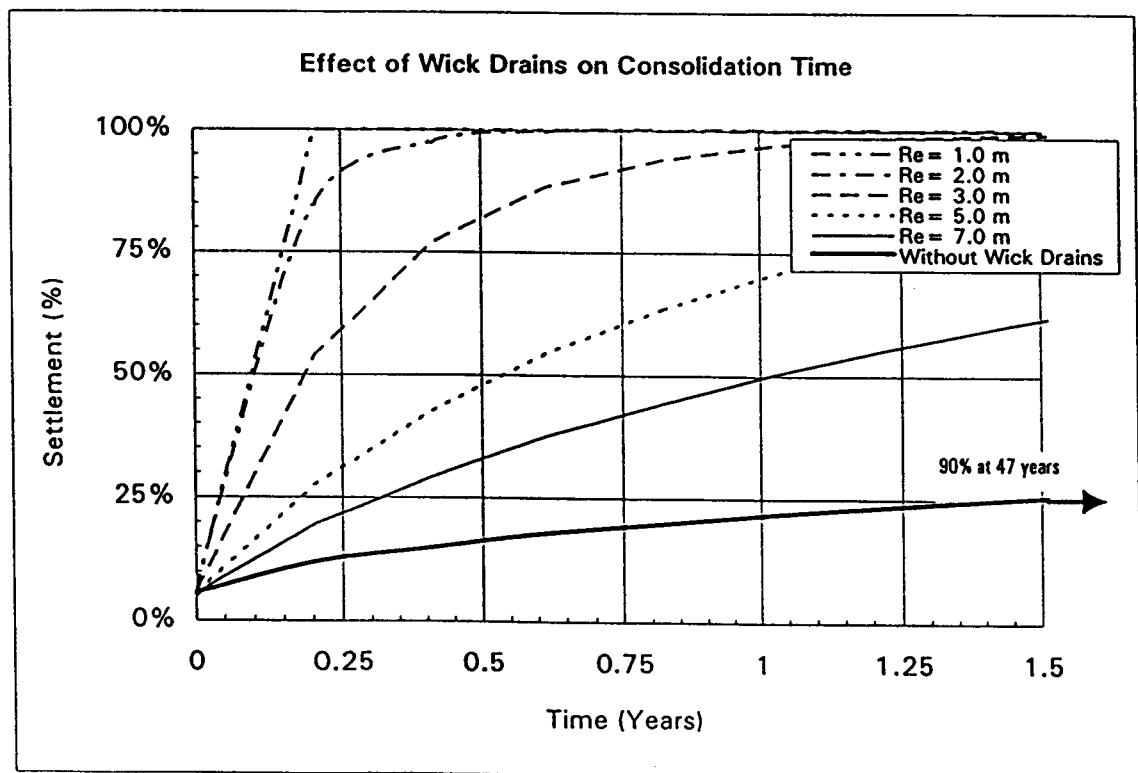
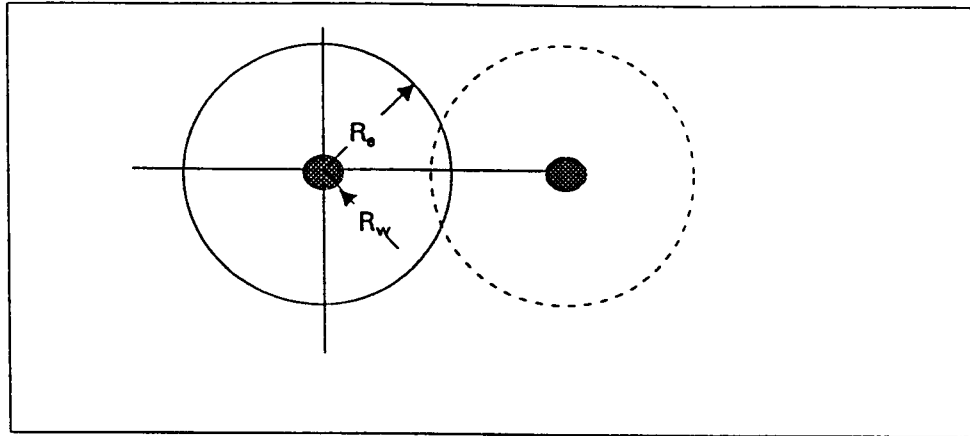
$C_{\text{radial}} = C_v$	5.00E-04 cm ² /sec
Wick Radius (R_w)	0.1 m
Wick Bottom Elevation	60.0 m



HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
RATE OF SETTLEMENT ANALYSIS
ABUTMENT EMBANKMENT FILLS
 $(C_v = 5 \cdot 10^{-4} \text{ cm}^2/\text{s})$

FIGURE 20

$C_{\text{radial}} = C_v$	3.00E-03 cm ² /sec
Wick Radius (R_w)	0.1 m
Wick Bottom Elevation	60.0 m



HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
RATE OF SETTLEMENT ANALYSIS
ABUTMENT EMBANKMENT FILLS
 $(C_v = 3 \times 10^{-3} \text{ cm}^2/\text{s})$

FIGURE 21

APPENDIX A

HIGHWAY 417, MISSISSIPPI RIVER BRIDGE
BOREHOLE LOGS

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ *N* VALUE
Very Soft	Less than 10	Less than 2
Soft	10 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	greater than 200	Greater than 30







NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT *N* VALUE
Very Loose	less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR TEST HOLE LOGS

SYMBOLS FOR SAMPLE TYPE		
	Shelby Tube	 A - Casing
	SPT	 Grab/Auger sample
	No Recovery	 Core

- MC - Moisture Content (% by Weight) as determined by sample]

 Water Level

C_{vane} Shear Strength Determination by Field Insitu Vane

C_{pen} Shear Strength Determination by Pocket Penetrometer

C_{lab} Shear Strength Determination using a Laboratory Vane Apparatus

C_u Undrained Shear Strength determined by Unconfined Compression Test

- (1) SPT Standard Penetration Test - refers to the number the blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			



BOREHOLE GRAPHIC SYMBOLS

SOILS



FILL

ORGANICS

CLAY

SILT

SAND

GRAVEL

COBBLES



SILTY CLAY

CLAYEY SILT

SILTY SAND

SAND & GRAVEL

CLAYEY SILT TILL

SILTY CLAY TILL

SANDY SILT TILL

ROCK



SHALE

LIMESTONE



SILTSTONE

GRANITE

OTHER



CEMENT GROUT

BENTONITE GROUT

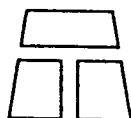


CONCRETE

WATER



BENTONITE SEAL



THURBER

RECORD OF BOREHOLE No 95-1

1 OF 1

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 872.5 E 323 474.5 ORIGINATED BY CB
DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
DATUM Geodetic DATE 95.03.15 & 95.03.15 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
91.5								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%) 20 40 60			GR SA SI CL
91.3	TOPSOIL (150mm)						91					
0.1	Silty CLAY to CLAY Very stiff Brown to stiff		1	SS	11		90					
			2	SS	11		89					
	-becoming grey		3	SS	4		88					
87.0							87	+ 4				
4.5	Silty CLAY to CLAY Firm Grey		4	SS	2		86					
			5	TW	PH		85					
			6	SS	WH		84	5 6				
			7	TW	PH		83	6 5				
			8	SS	WH		82	7				15.1
			9	TW	PH		81	5 4				
			10	ST	PH		80	4 4				
			11	SS	WH		79	4 2				15.6
75.0							78	4 4				
16.5	END OF BOREHOLE AT 16.5m. Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip. WATER LEVEL READINGS DATE DEPTH ELEVATION (m) (m) 95/03/18 3.05 88.40 95/04/13 1.84 89.61 95/05/24 1.98 89.47						77	5 4				
							76					
							75					

+ 3, × 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-2

2 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 844.1 E 323 500.1 ORIGINATED BY GA
DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
DATUM Geodetic DATE 95.03.15 & 95.03.15 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
			12	SS	WH		56					
							55					
							54					
							53					
							52					
							51					
			13	SS	WH		50					
							49					
							48					
							47					
							46					
							45					
			14	SS	WH		44					
							43					
							42					
40.9							41					
46.1	Fine grained, grey, sound, bioclastic LIMESTONE with occasional thin wavy shale partings, occasional sub-vertical fractures typically calcite infilled		15	NQ	RQD 80		40					
37.8			16	NQ	RQD 97		39					
49.1	END OF BOREHOLE AT 49.1m. Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip WATER LEVEL READINGS DATE DEPTH ELEVATION (m) (m) 95/03/18 + 1.10 88.00 95/04/13 + 4.18 91.10 95/05/24 + 3.19 90.11 + indicates water level above ground surface						38					

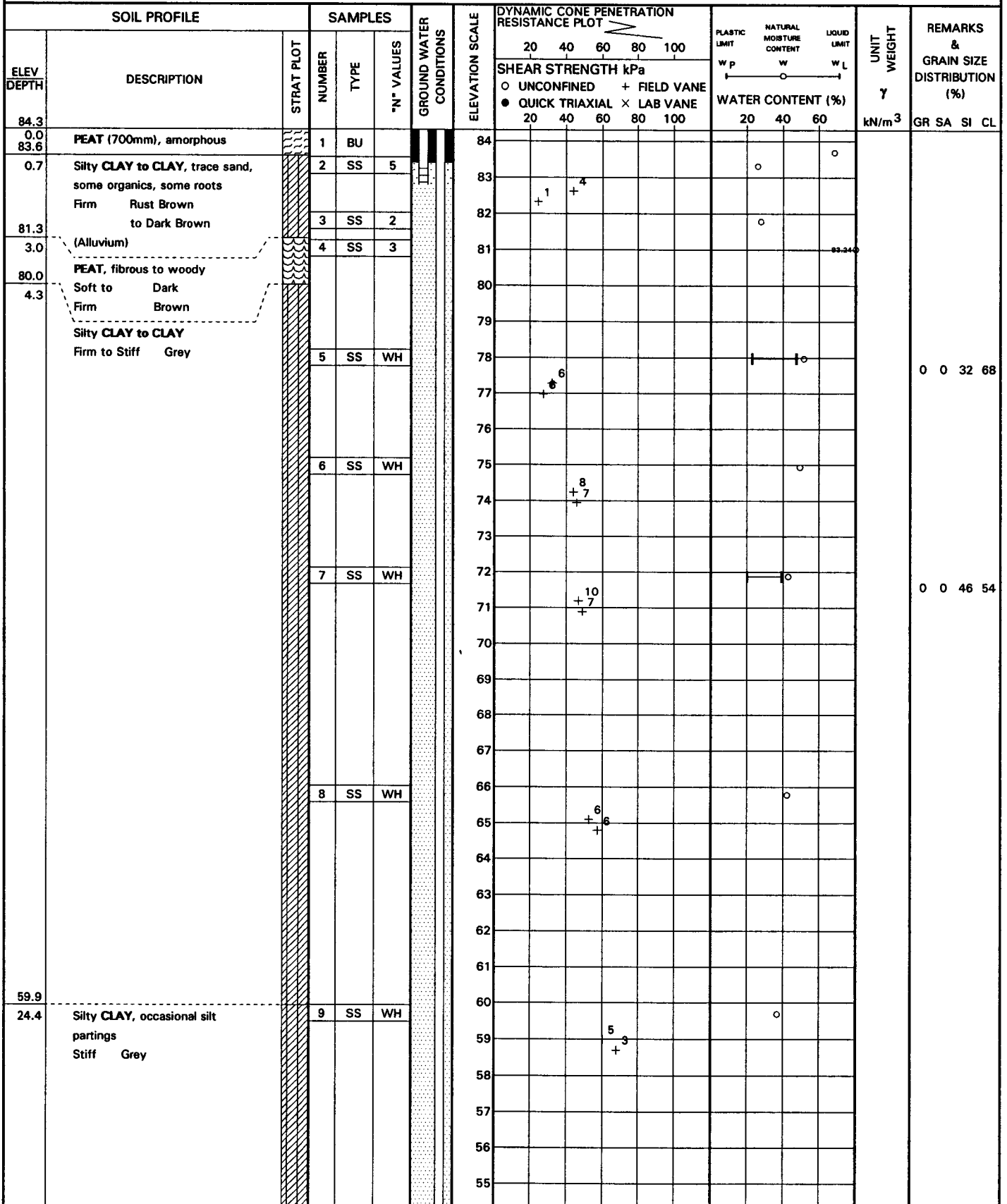
+ 3, x 3: Numbers refer to 20
Sensitivity 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-3

1 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 768.0 E 323 560.5 ORIGINATED BY CB
DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AGUERS COMPILED BY SP
DATUM Geodetic DATE 95.03.08 & 95.03.10 CHECKED BY IC



Continued Next Page

+ 3, × 3: Numbers refer to
Sensitivity 20
15 10 6 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-3

2 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 768.0 E 323 560.5 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AGUERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.08 & 95.03.10 CHECKED BY IC

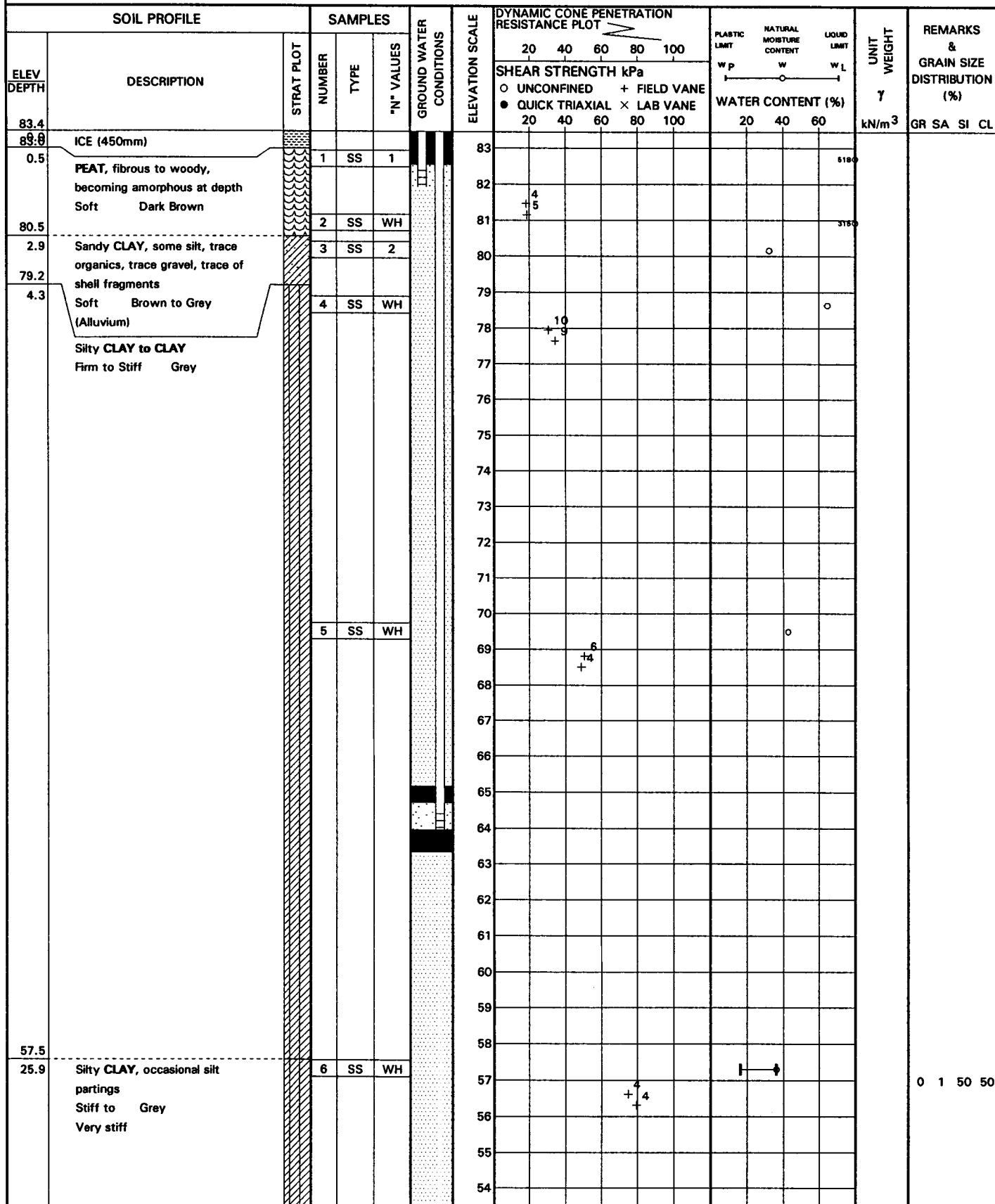
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
			10	SS	WH		54							
							53	4	4					
							52							
							51							
							50							
							49							
			11	SS	WH		48							
							47							
							46							
							45							
							44							
							43							
			12	SS	WH		42							
							41							
							40							
39.0							39							
45.3	fine-grained, grey, sound, bioclastic LIMESTONE with occasional thin wavy shale		13	NQR	QD = 100		38							
36.8	partings, occasional sub-vertical fractures, typically calcite infilled		14	NQ	RQD 100		37							
47.5	END OF BOREHOLE AT 47.5m. Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip. WATER LEVEL READINGS UPPER PIEZOMETER DATE DEPTH ELEVATION (m) (m) 95/03/14 0.47 83.83 95/04/12 1.19 83.11 95/05/24 1.09 83.21 LOWER PIEZOMETER DATE DEPTH ELEVATION (m) (m) 95/03/14 > +3.00 > 87.30 95/04/12 +7.63 91.93 95/05/24 +8.38 92.68 + Indicates water level above ground surface RQD = Rock Quality Designation													

RECORD OF BOREHOLE No 95-4

1 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 727.7 E 323 612.9 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.11 & 95.03.12 CHECKED BY IC



Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-4

2 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 727.7 E 323 612.9 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.11 & 95.03.12 CHECKED BY IC

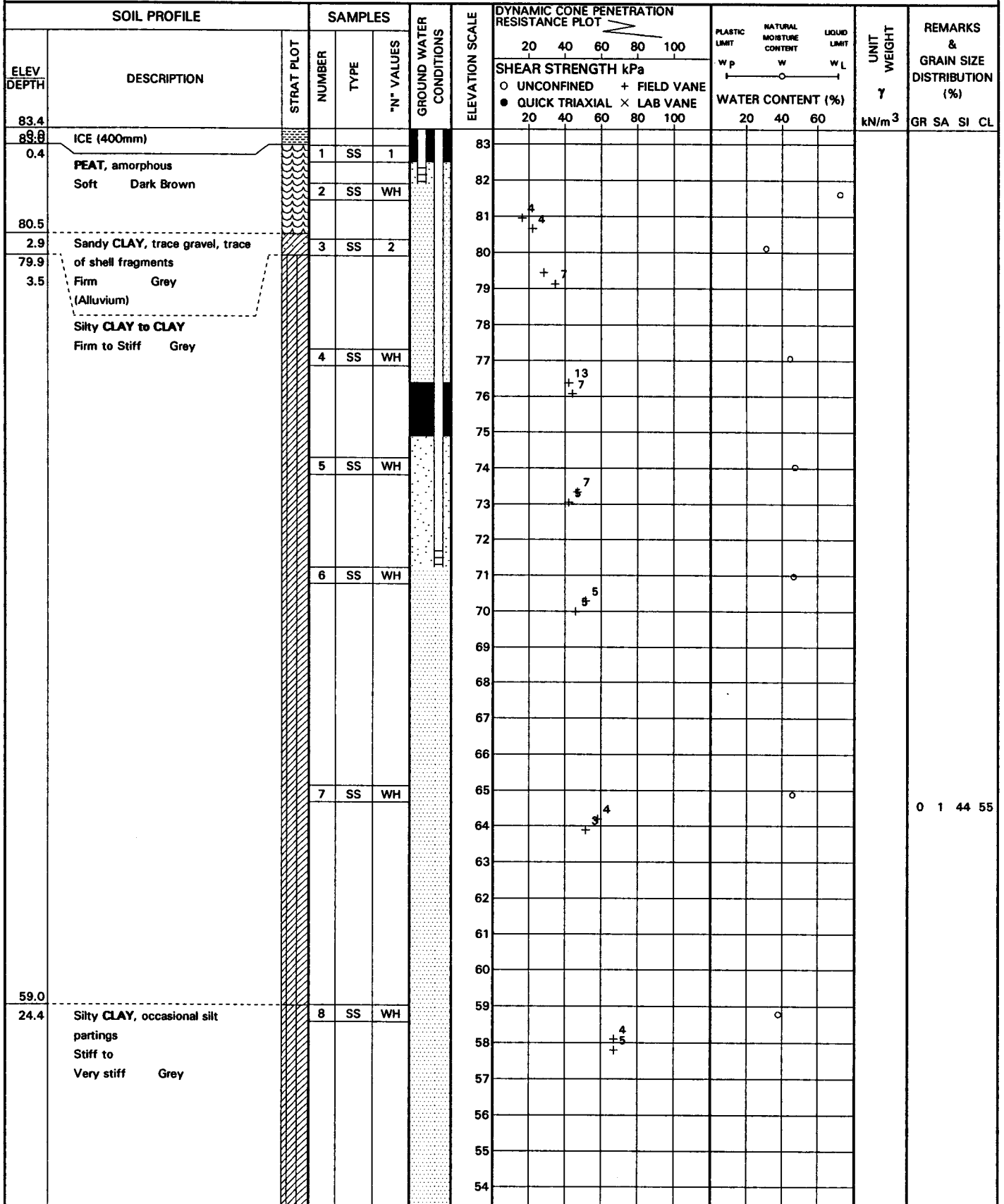
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				20 40 60						
							53									
							52									
							51									
							50									
							49									
			7	SS	WH		48									
							47									
							46									
							45									
							44									
							43									
							42									
							41									
							40									
38.7			8	SS	WH		39									
44.8	-some sand seams		9	NQ	RQD 92		38									
	Fine-grained, grey, sound, bioclastic LIMESTONE with occasional thin wavy shale partings, occasional sub-vertical fractures, typically calcite infilled		10	NQ	RQD 100		37									
35.9							36									
47.5	END OF BOREHOLE AT 47.5m. Piezometer installations consist of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip.															
	WATER LEVEL READINGS															
	UPPER PIEZOMETER															
	DATE DEPTH ELEVATION															
	(m) (m)															
	95/03/14 +0.27 83.20															
	95/04/12 +0.12 83.05															
	95/05/24 +0.24 83.17															
	LOWER PIEZOMETER															
	DATE DEPTH ELEVATION															
	(m) (m)															
	95/03/14 +2.13 85.06															
	95/04/12 +2.02 84.95															
	95/05/24 +2.60 85.53															
	+ indicates water level above ground surface															

RECORD OF BOREHOLE No 95-5

1 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 671.4 E 323 647.5 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.06 & 95.03.07 CHECKED BY IC



Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

METRIC[illegible]

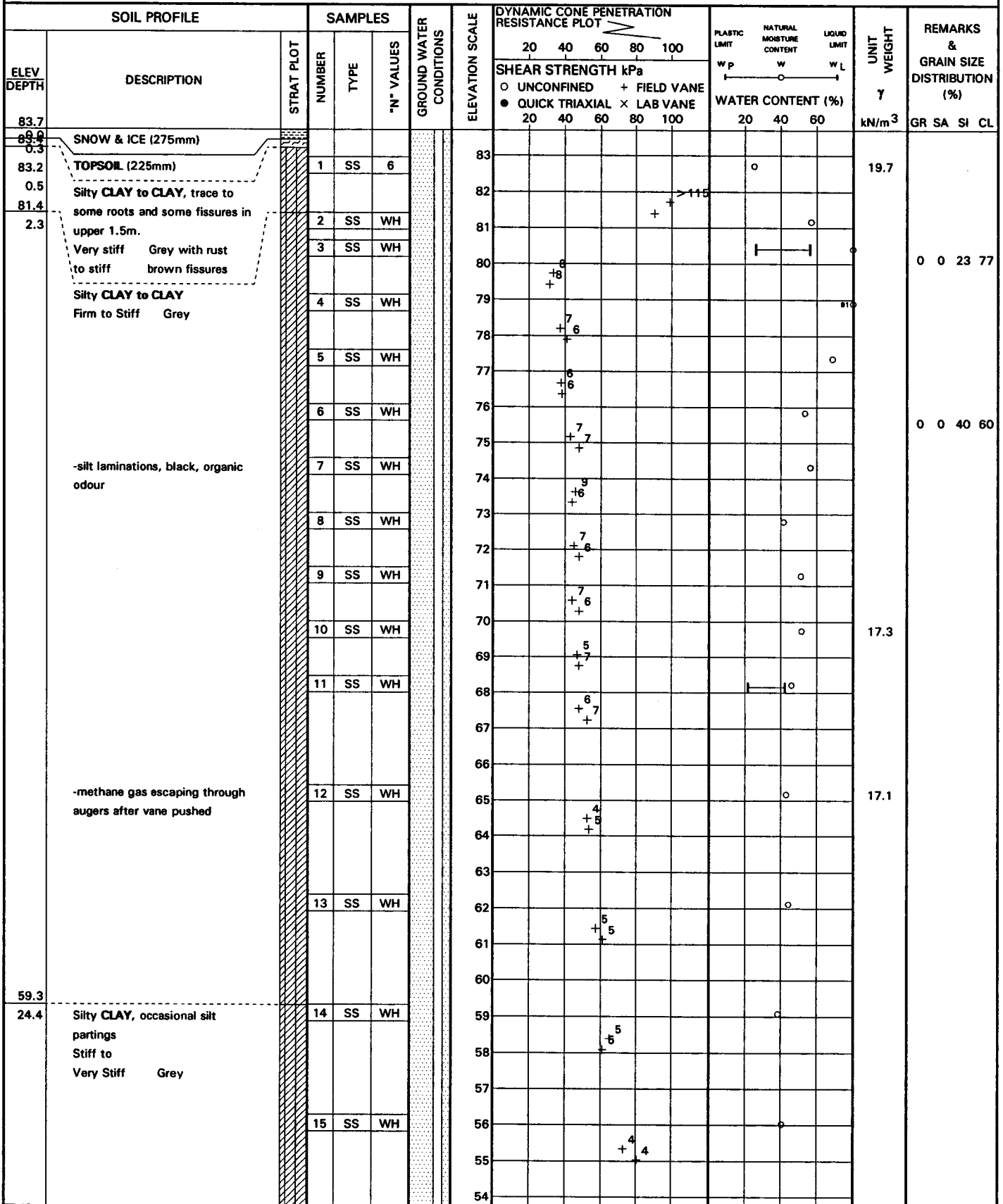
+ ³, × ³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 95-6

1 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 631.1 E 323 699.9 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.01 & 95.03.04 CHECKED BY IC



Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-6

2 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 631.1 E 323 699.9 ORIGINATED BY CB
DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
DATUM Geodetic DATE 95.03.01 & 95.03.04 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%) 20 40 60			GR SA SI CL
			16	SS	WH		53					
							52	4 +4				
			17	SS	WH		50					
							49	3 +4				
			18	SS	WH		47					
							46	5 +4				
			19	SS	WH		44					
							43					
			20	SS	WH		41					
38.3							40					
45.4	Silty SAND, some gravel, some clay, (TILL)		21	SS	WH		38					
36.8	Very Loose Grey						37					11 45 27 16
46.9			22	NQ	RQD		36					
				RC	80		35					
			23	NQ	RQD		34					
					83							
33.8	Fine-grained, grey, sound, bioclastic LIMESTONE with occasional thin wavy shale partings, occasional sub-vertical fractures typically calcite infilled		24	NQ	RQD							
					70							
49.9	END OF BOREHOLE AT 49.9m. Piezometer installation consists of 19mm diameter schedule 40 PVC with a 0.45m slotted tip. WATER LEVEL READINGS DATE DEPTH ELEVATION (m) (m) 95/03/14 > +3.00 > 86.69 95/04/13 +8.88 92.57 95/05/24 +8.96 92.65 + indicates water level above ground surface											

+ 3, × 3: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-7

1 OF 1

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 603.7 E 323 708.3 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm SOLID STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.13 & 95.03.14 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					PLASTIC LIMIT w _p NATURAL MOISTURE CONTENT w LIQUID LIMIT w _L
88.1								20 40 60 80 100					
87.9	TOPSOIL (150mm)												
0.1	Silty CLAY to CLAY, some fissures Very Stiff Brown to stiff		1	SS	11								
			2	SS	10								
			3	SS	PH								
			4	SS	2								
83.0													
5.1	Silty CLAY to CLAY Firm to Stiff Grey												
			5	SS	WH								
			6	TW	PH								
			7	TW	PH								
			8	TW	PH								
			9	TW	PH								
			10	SS	WH								
			11	SS	WH								
71.6													
16.5	END OF BOREHOLE AT 16.46m. Piezometer installations consists of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip. WATER LEVEL READINGS DATE DEPTH ELEVATION (m) (m) 95/03/14 9.60 78.46 95/04/13 2.34 85.72 95/05/24 2.34 85.72												

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-8

1 OF 1

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 836.3 E 323 460.5 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.14 & 95.03.15 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
91.1								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%) 20 40 60			GR SA SI CL
91.1 0.1	TOPSOIL (150mm) Silty CLAY to CLAY, some rust coloured fissures in upper 1.5m. Very Stiff Brown becoming Grey		1	SS	10		91					15.7 0 0 32 68
			2	SS	12		90					
							89					
			3	SS	6		88					
87.0							87					
4.1	Silty CLAY to CLAY Firm Grey		4	TW	PH		86					
			5	SS	WH		85					
			6	SS	WH		84					
			7	TW	PH		83					
			8	SS	WH		82					
			9	TW	PH		81					
							80					
							79					
							78					
							77					
							76					
							75					
74.6												
16.5	END OF BOREHOLE AT 16.5m. Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip. WATER LEVEL READINGS DATE DEPTH ELEVATION (m) (m) 95/03/18 8.70 82.40 95/04/13 2.06 89.04 95/05/24 2.05 89.05											

+³, ×³: Numbers refer to
Sensitivity

20
15
10

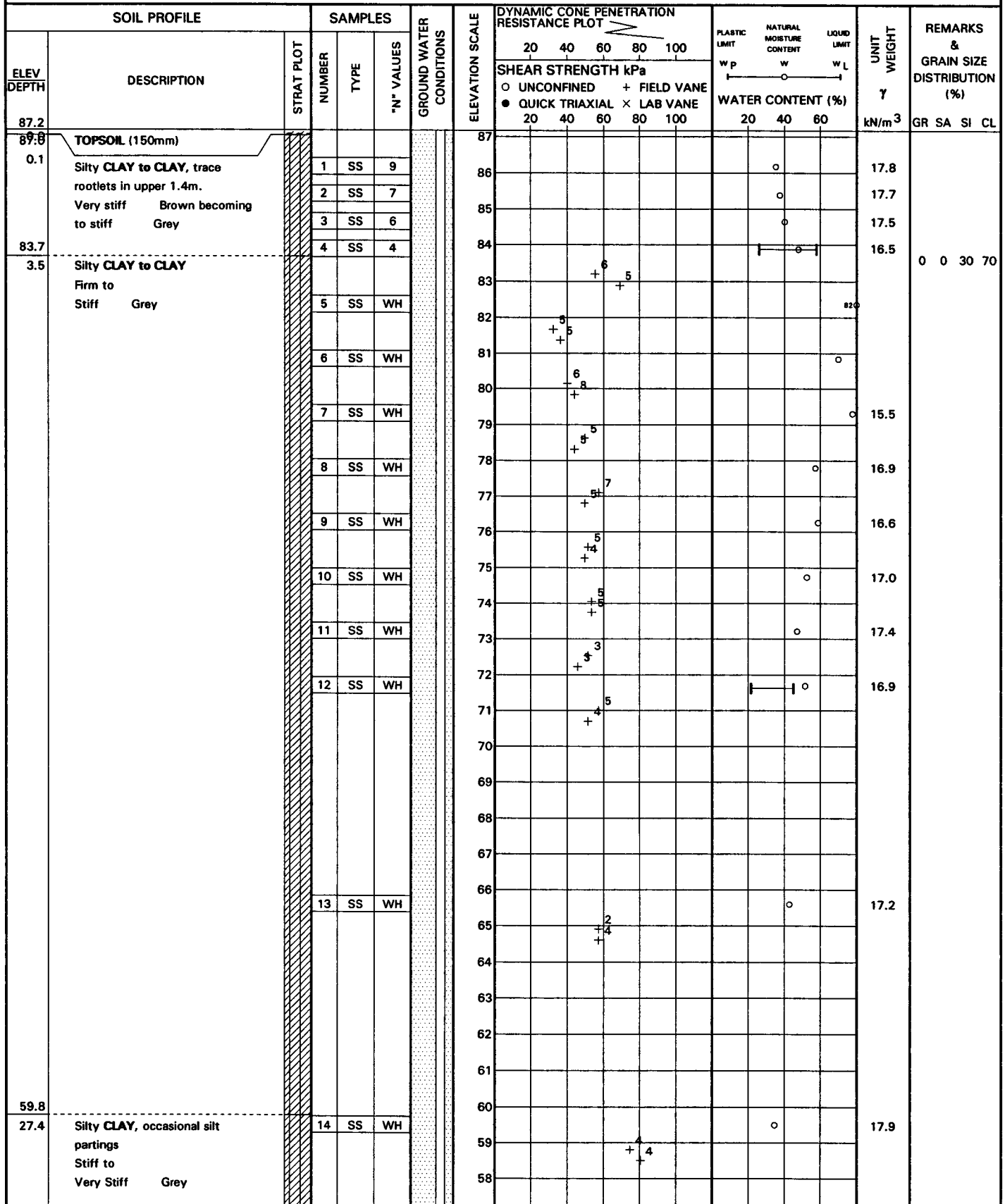
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-9

1 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 810.4 E 323 489.6 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.14 & 95.03.16 CHECKED BY IC



Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-9

2 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 810.4 E 323 489.6 ORIGINATED BY CB
DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
DATUM Geodetic DATE 95.03.14 & 95.03.16 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60						80	100	SHEAR STRENGTH kPa			WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE															
							57																
							56																
							55																
							54																
			15	SS	WH		53										17.6						
							52																
							51																
							50																
							49																
			16	SS	WH		48										17.9						
							47																
							46																
							45																
							44																
							43																
							42																
							41																
40.4	-50mm sand layer		17	SS	WH		40										0 10 49 41						
46.7	Fine-grained, grey, sound, bioclastic LIMESTONE, with occasional thin wavy shale partings, occasional sub-vertical fractures typically calcite infilled		18	NQ	RQD 100		39																
38.0			19	NQ	RQD 100		38																
49.2	END OF BOREHOLE AT 49.2m. Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip. WATER LEVEL READINGS DATE DEPTH ELEVATION (m) (m) 95/03/18 +1.50 88.69 95/04/13 +1.67 88.86 95/05/24 +1.62 88.78 + indicates water level above ground surface																						

+³, ×³: Numbers refer to Sensitivity

METRIC

+ ³, × ³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 95-10

2 OF 2

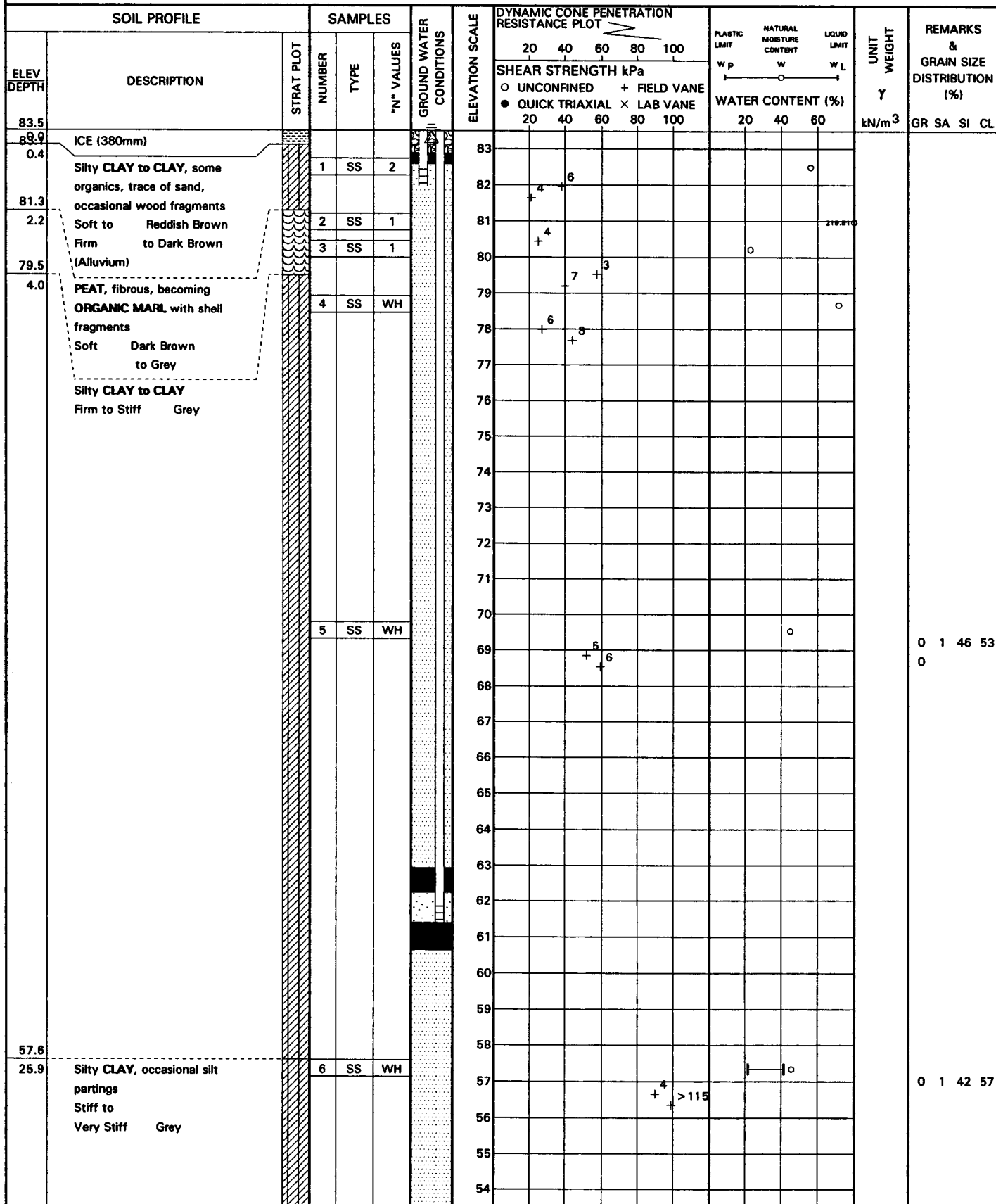
METRIC

W.P. 451-90-03/04 LOCATION N 5 026 745.3 E 323 542.8 ORIGINATED BY GA
DIST 42 HWY 417 BOREHOLE TYPE 210mm SOLID STEM AUGERS COMPILED BY SP
DATUM Geodetic DATE 95.03.08 & 95.03.10 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
						20	40	60	80	100	20	40	60		
			9	SS	WH										
40.0	-some sand layers		11	SS	WH										
43.5	Fine-grained, grey, sound, bioclastic LIMESTONE with occasional thin wavy shale partings, occasional sub-vertical fractures typically calcite infilled		12	NQ	RQD										
37.7			13	NQ	RQD										
45.8	END OF BOREHOLE AT 45.8m. Piezometer installations consist of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip.														
	WATER LEVEL READINGS														
	UPPER PIEZOMETER														
	DATE DEPTH ELEVATION														
	(m) (m)														
	95/03/14 1.00 83.42														
	95/04/12 1.04 83.38														
	95/05/24 1.16 83.26														
	LOWER PIEZOMETER														
	DATE DEPTH ELEVATION														
	(m) (m)														
	95/03/14 +1.12 85.54														
	95/04/12 +1.31 85.73														
	95/05/24 +1.23 85.65														
	+ indicates water level above ground surface														

+ 3 × 3: Numbers refer to 20
Sensitivity 15 5 10 (%) STRAIN AT FAILURE

METRIC

[illegible]

+ ³, × ³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 95-11

2 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 689.0 E 323 577.4 ORIGINATED BY GA
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.11 & 95.03.12 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
			7	SS	WH									
40.2														
43.3	Fine-grained, grey, sound, bioclastic LIMESTONE with occasional thin wavy shale partings, occasional sub-vertical fractures typically calcite infilled		8	NQ	RQD 100									
37.9			9	NQ	RQD 100									
45.5	END OF BOREHOLE AT 45.5m. Piezometer installations consist of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip. WATER LEVEL READINGS UPPER PIEZOMETER DATE DEPTH ELEVATION (m) (m) 95/03/14 0.02 83.08 95/04/12 +0.18 83.28 95/05/24 +0.05 83.15 LOWER PIEZOMETER DATE DEPTH ELEVATION (m) (m) 95/03/14 2.69 80.41 95/04/12 1.56 81.54 95/05/24 2.00 81.10 + indicates water level above ground surface													

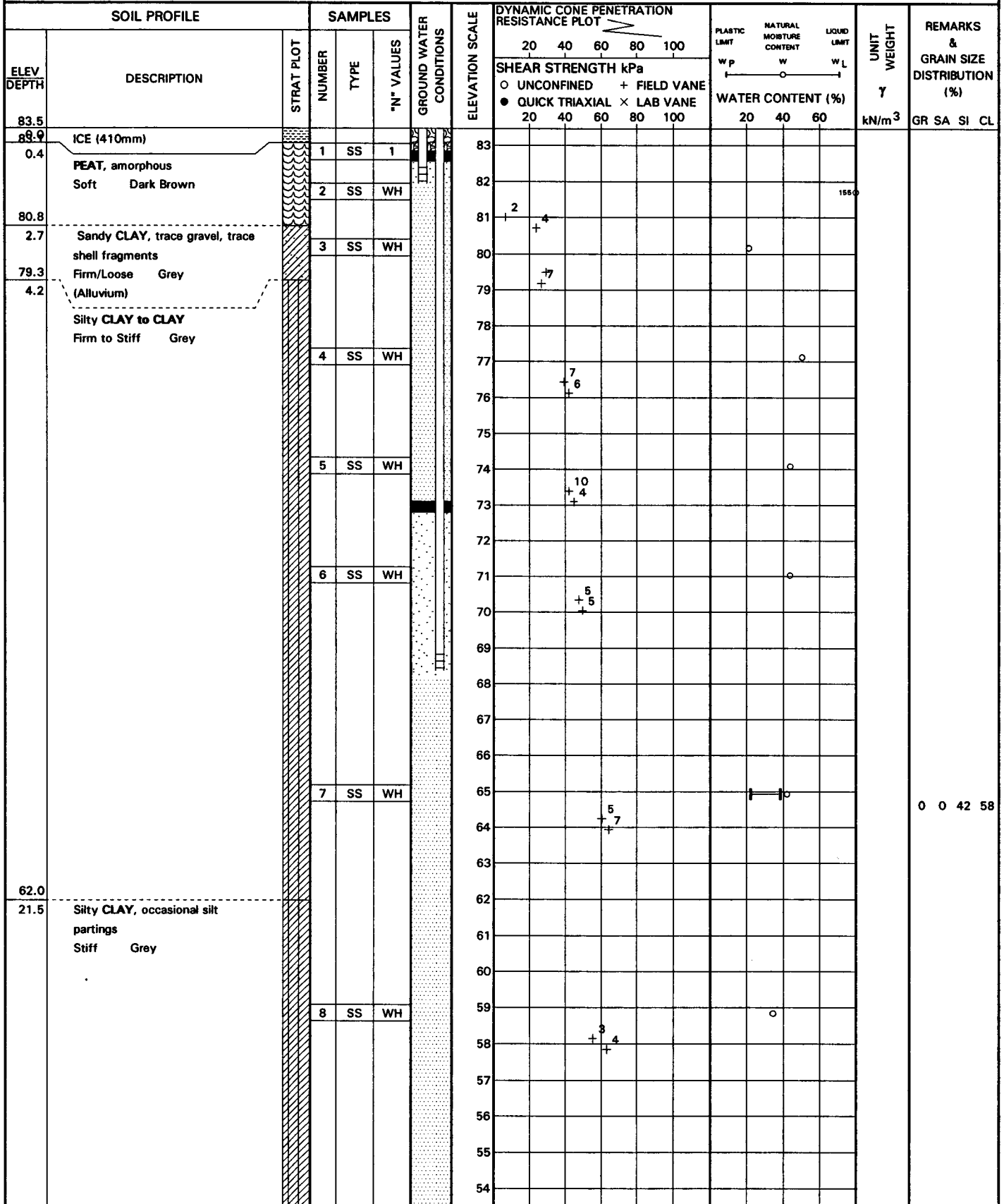
+ 3, x 3; Numbers refer to 20
Sensitivity 15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-12

1 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 648.7 E 323 629.8 ORIGINATED BY CB
DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
DATUM Geodetic DATE 95.03.06 & 95.03.08 CHECKED BY IC



Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity 20
15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-12

2 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 648.7 E 323 629.8 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.06 & 95.03.08 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
			9	SS	WH		53					0 1 36 63
							52					
							51					
							50					
							49					
							48					
							47					
			10	SS	WH		46					
							45					
							44					
							43					
							42					
							41					
			11	SS	WH		40					
							39					
39.0							38					
44.5	Fine-grained, grey, sound, bioclastic LIMESTONE , occasional thin wavy shale partings, occasional sub-vertical fractures typically calcite infilled		12	NQ	RQD 96		37					
36.1			13	NQ	RQD 100							
47.4	END OF BOREHOLE AT 47.4m. Piezometer installations consist of 19mm diameter schedule 40 PVC pipe with a 0.45m slotted tip. WATER LEVEL READINGS UPPER PIEZOMETER DATE DEPTH ELEVATION (m) (m) 95/03/04 +0.01 83.05 95/04/12 0.00 83.04 95/05/24 +0.05 83.09 LOWER PIEZOMETER DATE DEPTH ELEVATION (m) (m) 95/03/14 3.28 79.76 95/04/12 +0.07 83.11 95/05/24 +0.75 83.79 + indicates water level above ground surface											

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

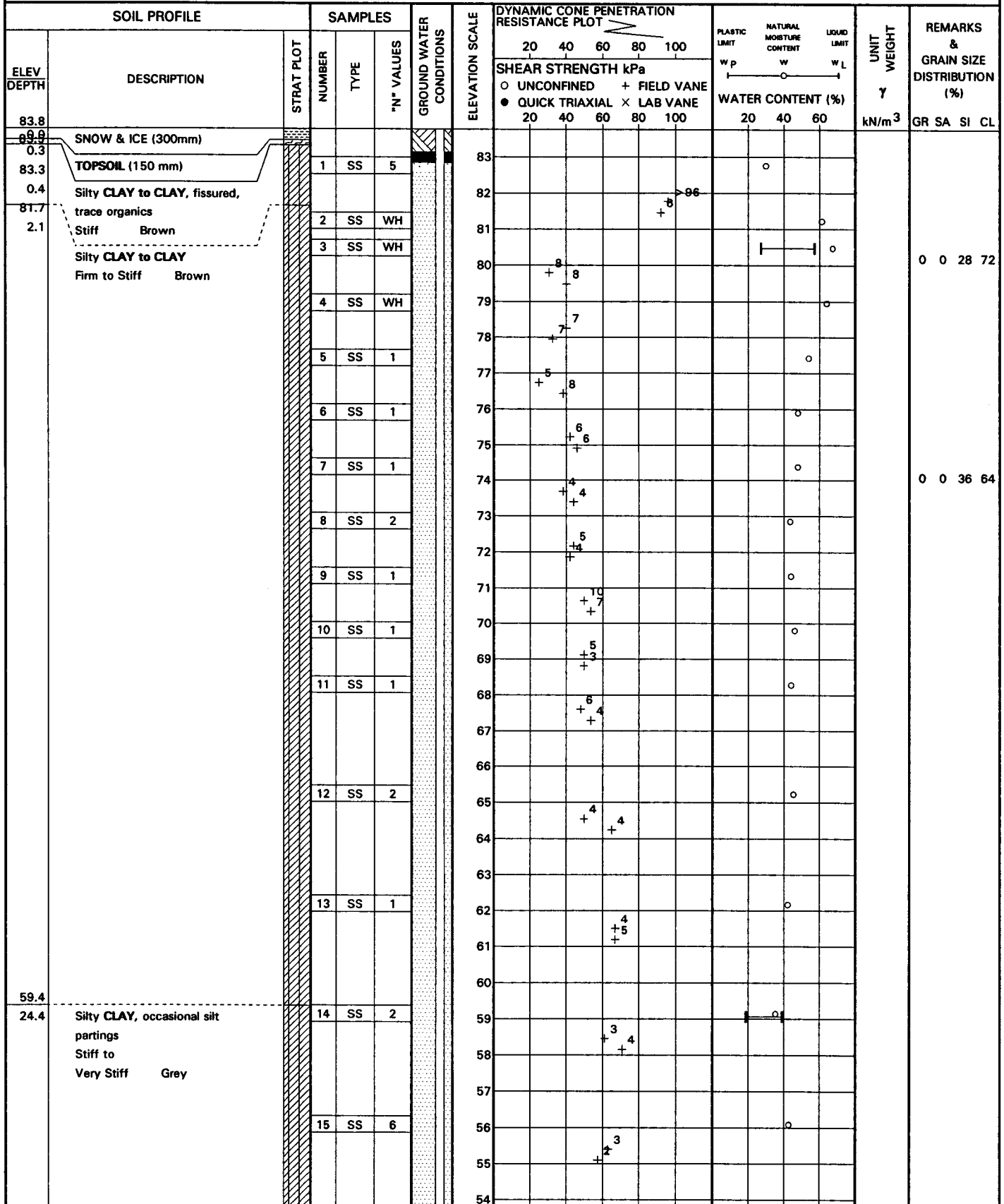
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-13

1 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 597.4 E 323 663.9 ORIGINATED BY GA
 DIST 42 HWY 417 BOREHOLE TYPE 210mm SOLID STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.01 & 95.03.04 CHECKED BY IC



Continued Next Page

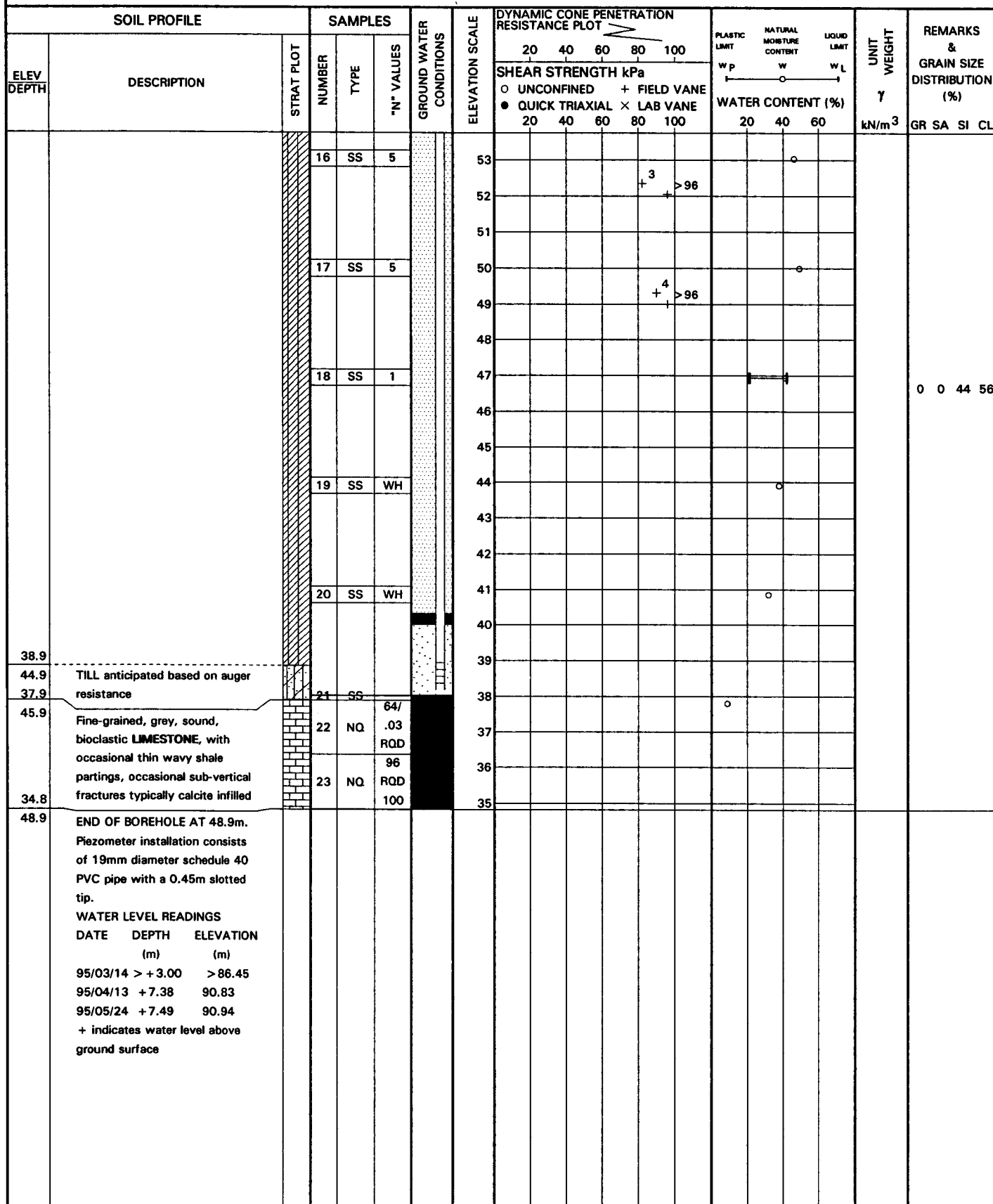
+ 3, x 3: Numbers refer to
Sensitivity 20
15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-13

2 OF 2

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 597.4 E 323 663.9 ORIGINATED BY GA
DIST 42 HWY 417 BOREHOLE TYPE 210mm SOLID STEM AUGERS COMPILED BY SP
DATUM Geodetic DATE 95.03.01 & 95.03.04 CHECKED BY IC



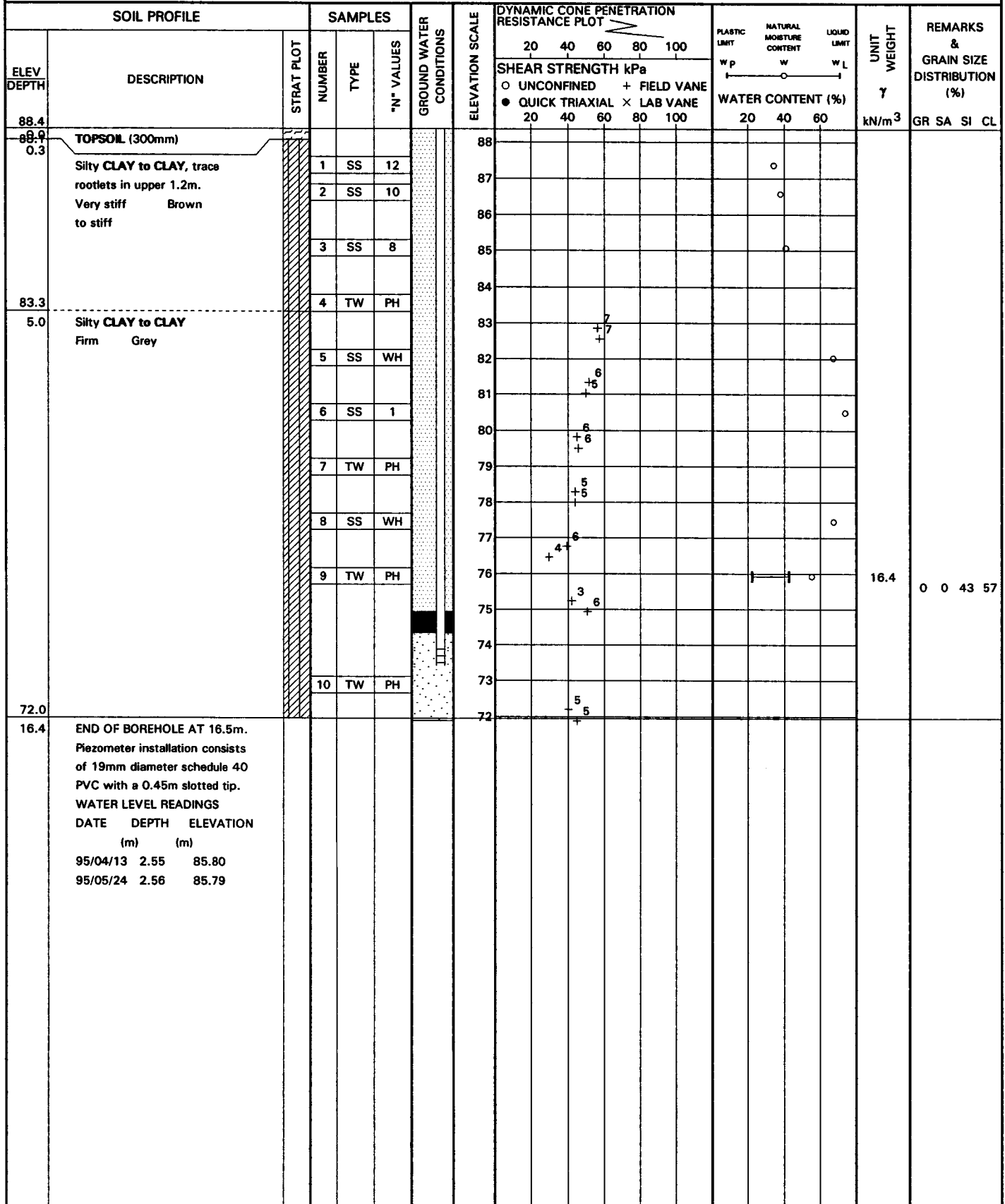
+³, ×³: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 95-14

1 OF 1

METRIC

W.P. 451-90-03/04 LOCATION N 5 026 580.1 E 323 683.5 ORIGINATED BY CB
 DIST 42 HWY 417 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY SP
 DATUM Geodetic DATE 95.03.14 & 95.03.14 CHECKED BY IC



+ 3, × 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

APPENDIX B

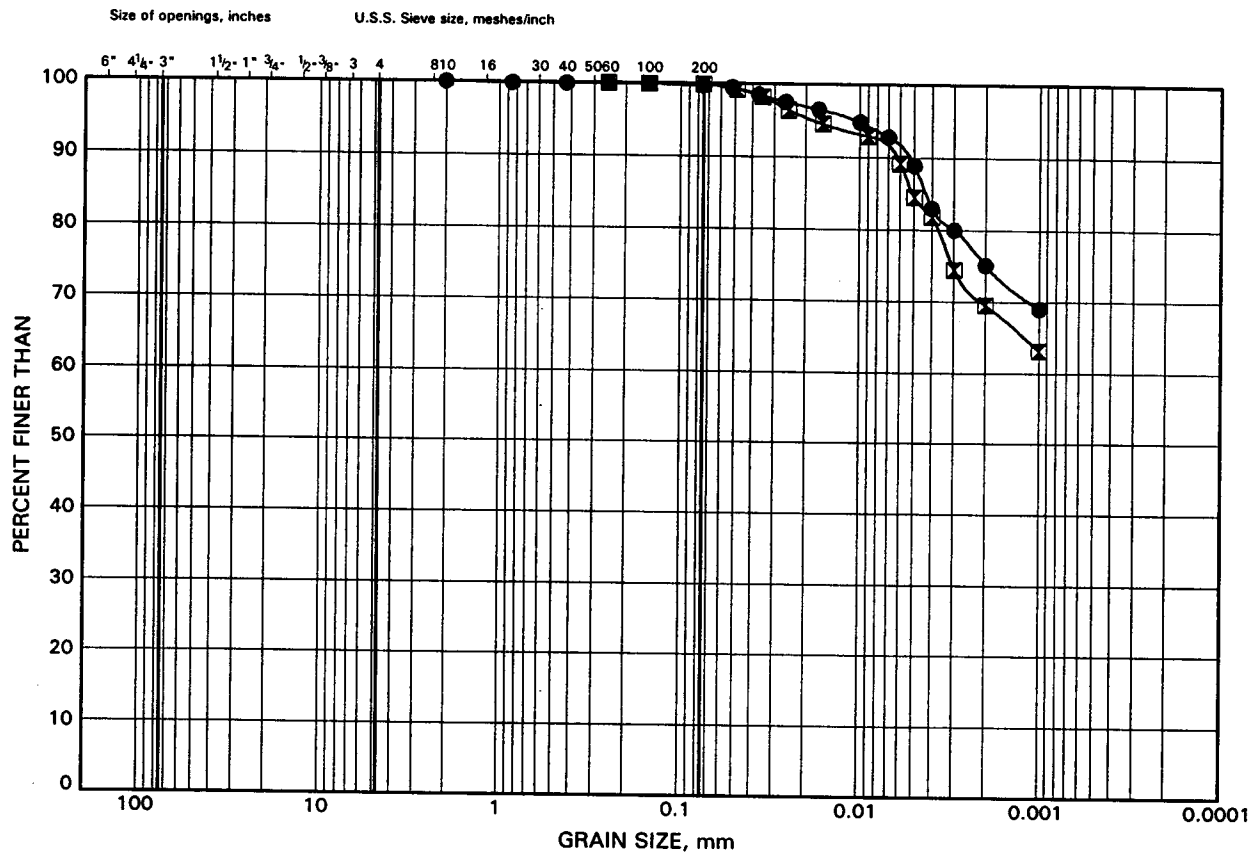
**HIGHWAY 417, MISSISSIPPI RIVER BRIDGE
LABORATORY TEST RESULTS**

Grain Size Analyses

MISSISSIPPI RIVER BRIDGE INVESTIGATION GRAIN SIZE DISTRIBUTION

FIGURE B1

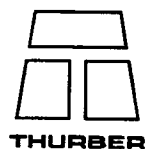
Dessicated Crust-Upper Silty Clay West Approach



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-2	3.28	83.64
⊠	95-9	3.28	83.88

Date May 1995
Project 451-90-03/04



Prep'd WM
Chkd. IC

FIGURE B2

Size of openings, inches

U.S.S. Sieve size, meshes/inch

6" 4 1/4" 3" 1 1/2" 1" 3/4" 1/2" 3/8" 3/16" 4 8 10 16 30 40 50 60 100 200

100 10 1 0.1 0.01 0.001 0.0001


PERCENT FINER THAN

GRAIN SIZE, mm

Grain Size (mm)	Percent Finer Than (%)
100	100
10	100
1	100
0.1	100
0.075	100
0.06	98
0.05	97
0.04	95
0.03	93
0.025	91
0.02	87
0.015	83
0.0125	78
0.01	73
0.0075	67
0.006	60

COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-7	3.28	84.78

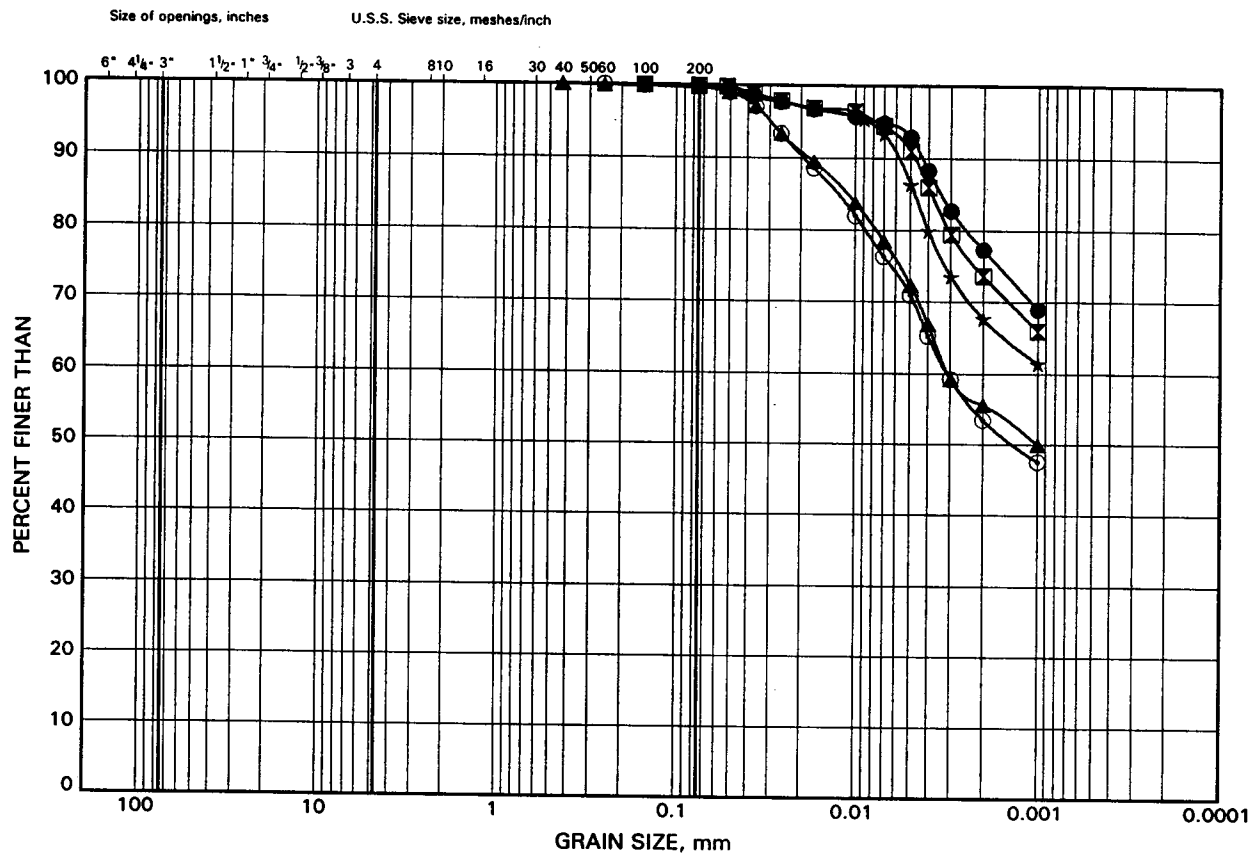


Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION GRAIN SIZE DISTRIBUTION

FIGURE B3

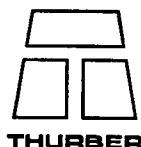
Upper Silty Clay - WBL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-1	9.37	82.08
◻	95-1	12.42	79.03
▲	95-2	18.52	68.40
★	95-3	6.33	77.97
○	95-3	12.42	71.88

Date May 1995
Project 451-90-03/04

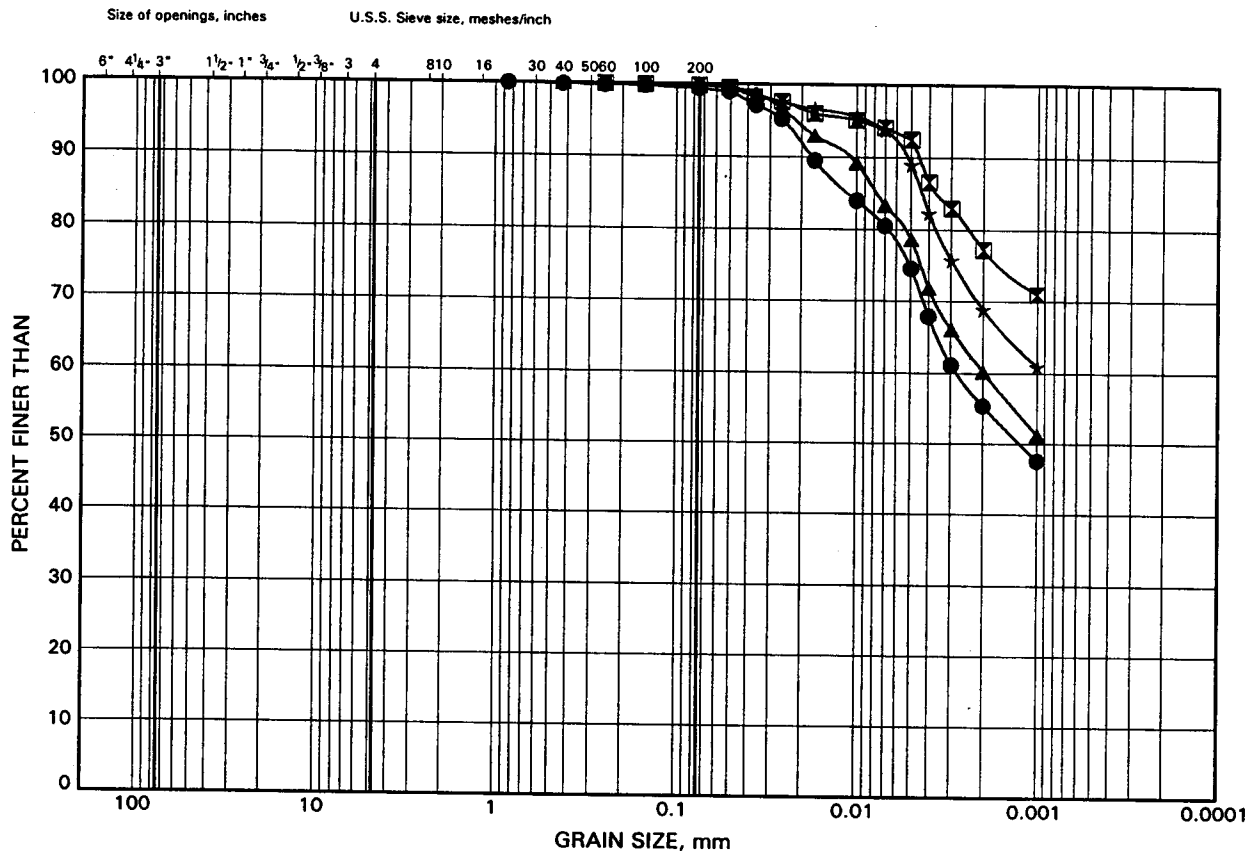


Prep'd WM
Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION GRAIN SIZE DISTRIBUTION

FIGURE B4

Upper Silty Clay - WBL

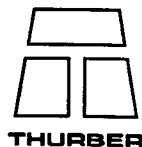


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-5	18.52	64.88
◻	95-6	3.28	80.41
▲	95-6	7.85	75.84
★	95-7	7.85	80.21

Date May 1995

Project 451-90-03/04



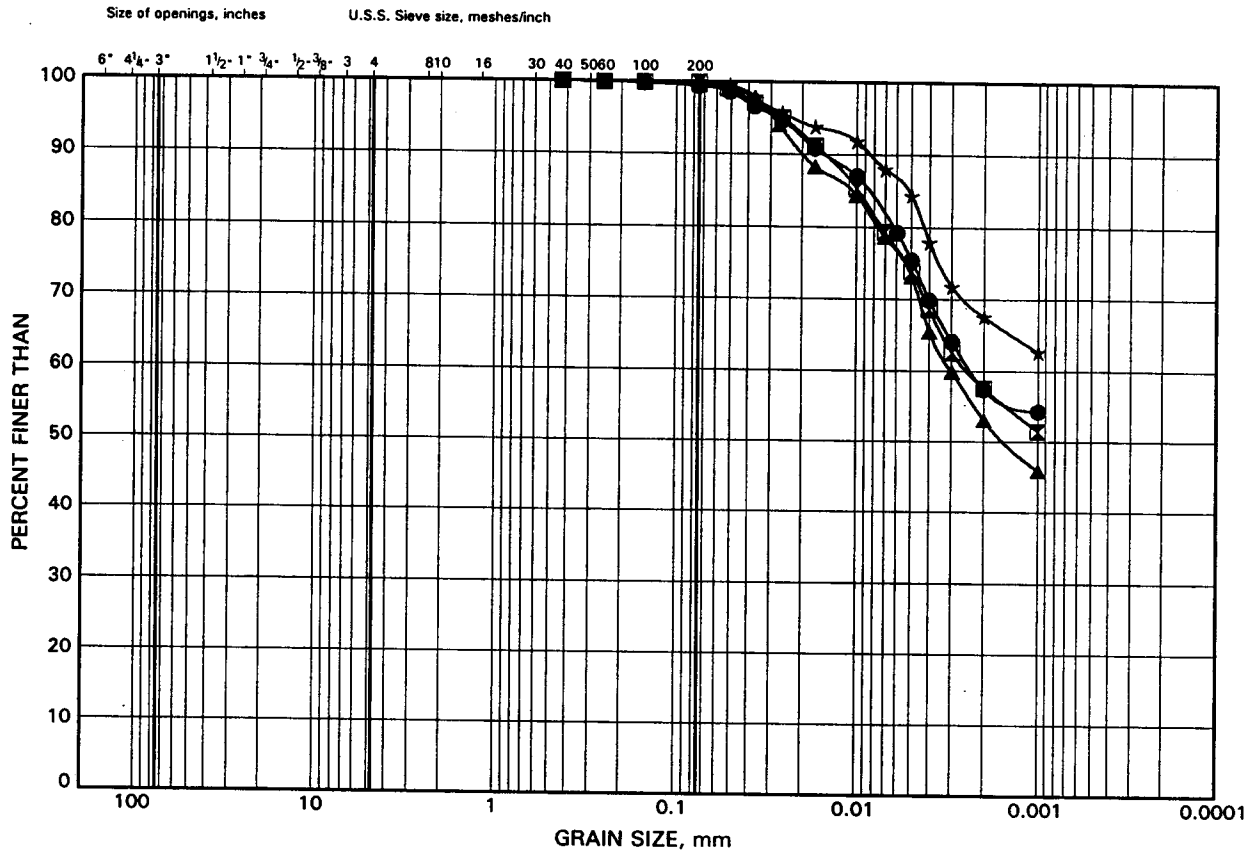
Prep'd WM

Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION GRAIN SIZE DISTRIBUTION

FIGURE B5

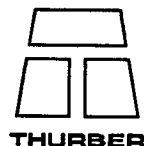
Upper Silty Clay - EBL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-10	6.33	77.15
⊠	95-10	12.42	71.06
▲	95-11	13.95	69.53
★	95-8	4.80	86.30

Date May 1995
Project 451-90-03/04

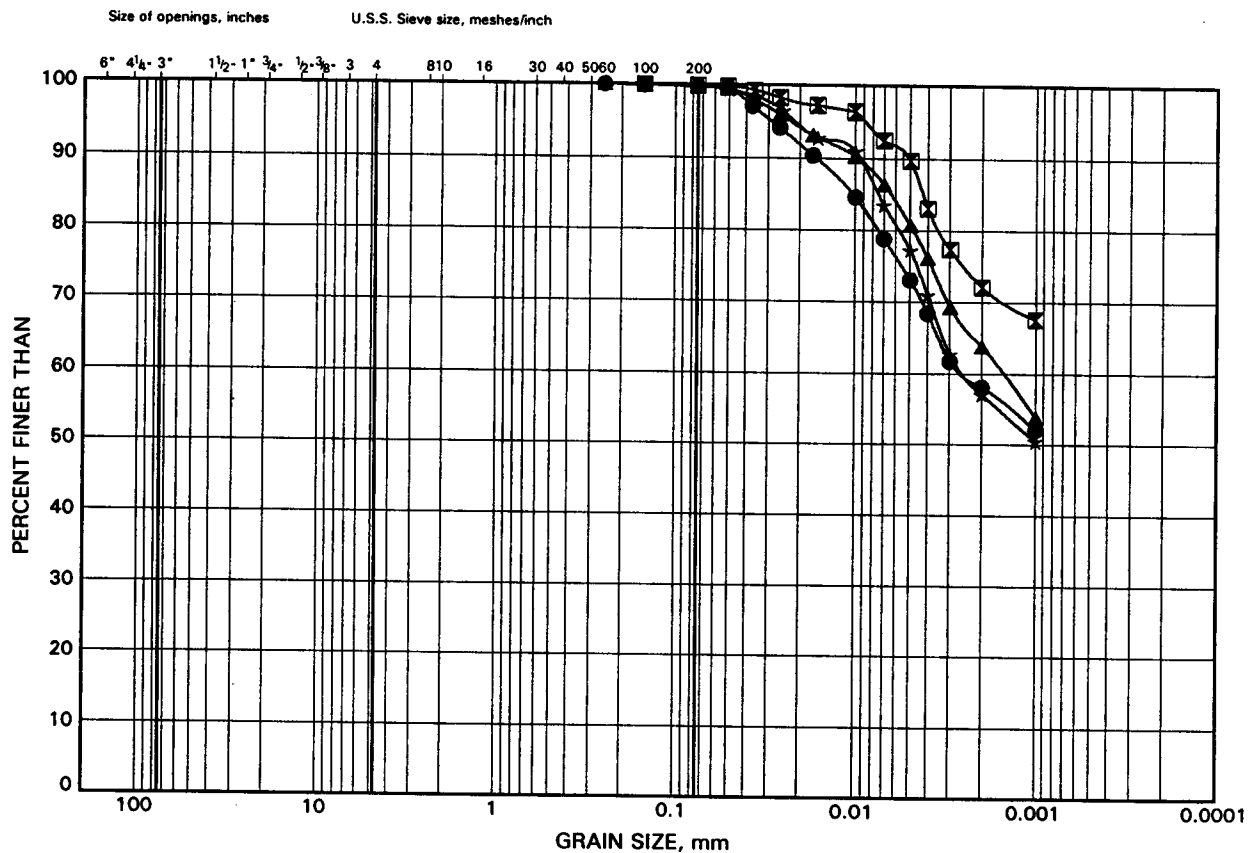


Prep'd WM
Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION GRAIN SIZE DISTRIBUTION

FIGURE B6

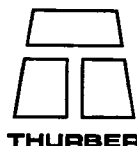
Upper Silty Clay - EBL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-12	18.52	64.93
☒	95-13	3.28	80.47
▲	95-13	9.37	74.38
★	95-14	12.42	75.93

Date May 1995
Project 451-90-03/04

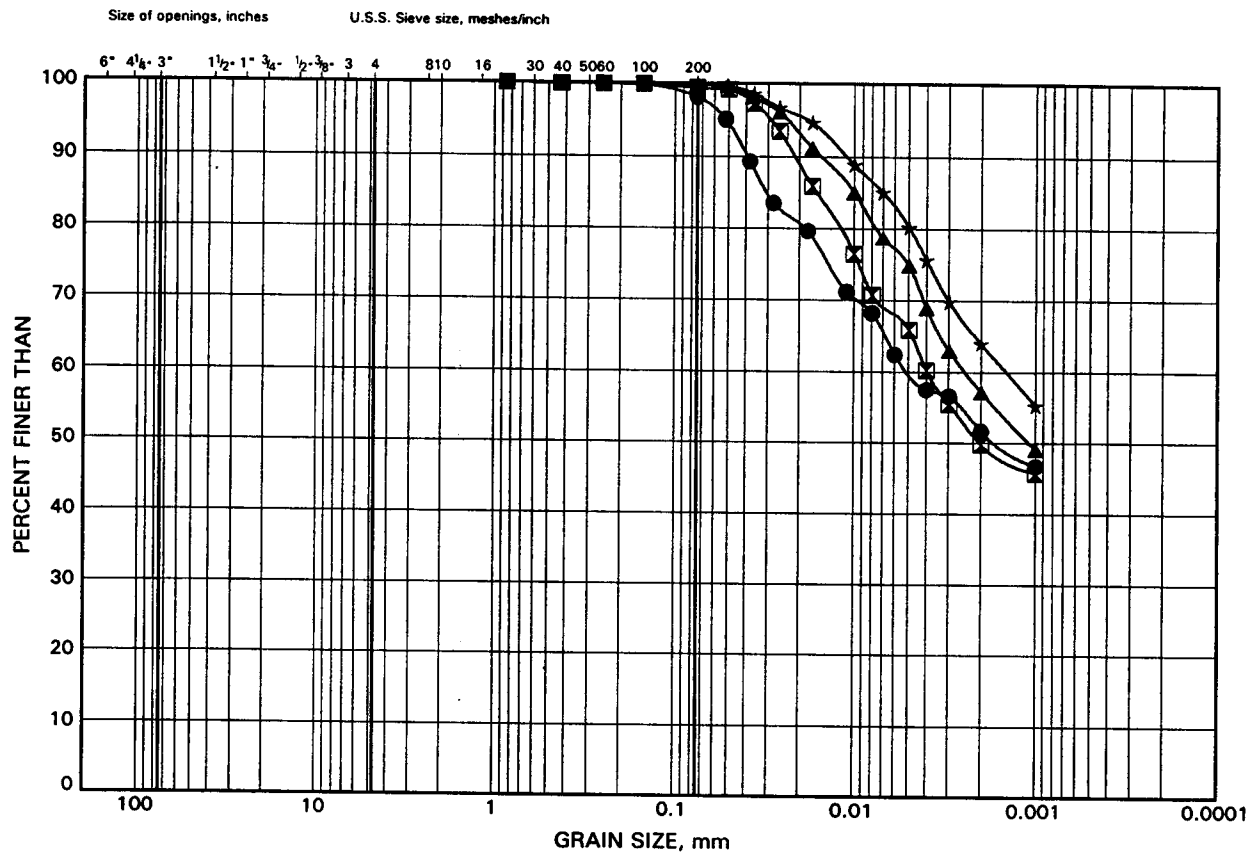


Prep'd WM
Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION GRAIN SIZE DISTRIBUTION

FIGURE B7

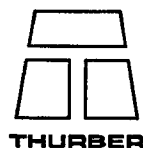
Lower Silty Clay - WBL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-2	42.90	44.02
◻	95-4	26.14	57.30
▲	95-4	35.28	48.16
★	95-5	30.71	52.69

Date May 1995
Project 451-90-03/04

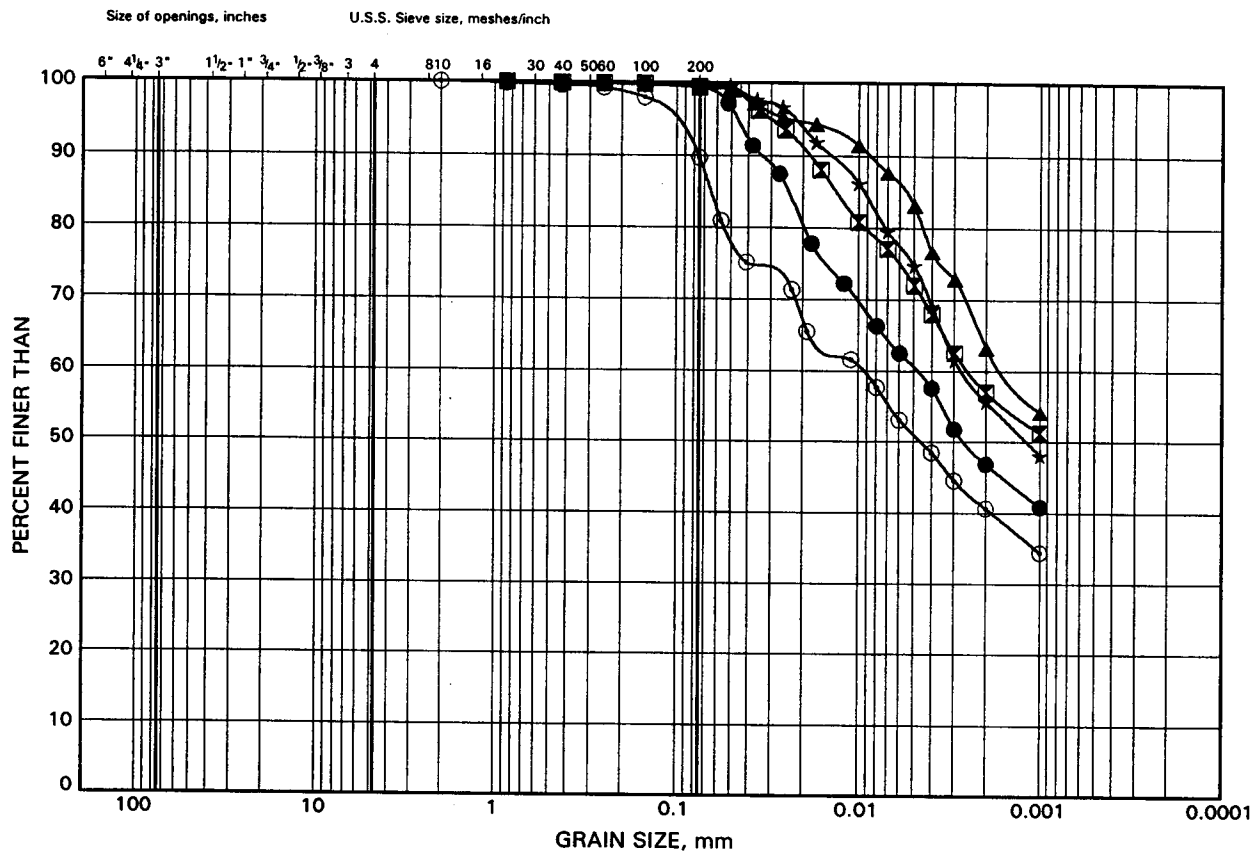


Prep'd WM
Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION GRAIN SIZE DISTRIBUTION

FIGURE B8

Lower Silty Clay - EBL

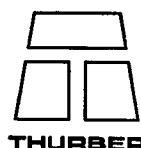


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-10	24.61	58.87
⊠	95-11	26.14	57.34
▲	95-12	30.71	52.74
★	95-13	36.81	46.94
⊙	95-9	45.95	41.21

THURBGSD 4001 95/05/26

Date May 1995
Project 451-90-03/04



Prep'd WM
Chkd. IC

FIGURE B9

Size of openings, inches

U.S.S. Sieve size, meshes/inch

6" 4 1/4" 3" 1 1/2" 1" 3/4" 1/2" 3/8" 3/4" 4 8 10 16 30 40 50 60 100 200

PERCENT FINER THAN

100 90 80 70 60 50 40 30 20 10 0

GRAIN SIZE, mm

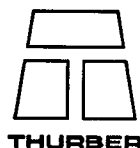
100 10 1 0.1 0.01 0.001 0.0001

Grain Size (mm)	Percent Finer (%)
100	100
75	100
60	100
48	100
38	100
30	100
25	100
20	100
16	100
12	100
10	100
8	98
6	95
4.75	90
3.75	82
3.0	75
2.5	66
2.0	58
1.5	49
1.18	42
1.0	38
0.85	35
0.75	30
0.60	27
0.50	25
0.425	22
0.354	19
0.300	18
0.250	15
0.200	14
0.150	12

COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-6	45.95	37.74

Date May 1995
Project 451-90-03/04



Prep'd WM
Chkd. IC

APPENDIX C

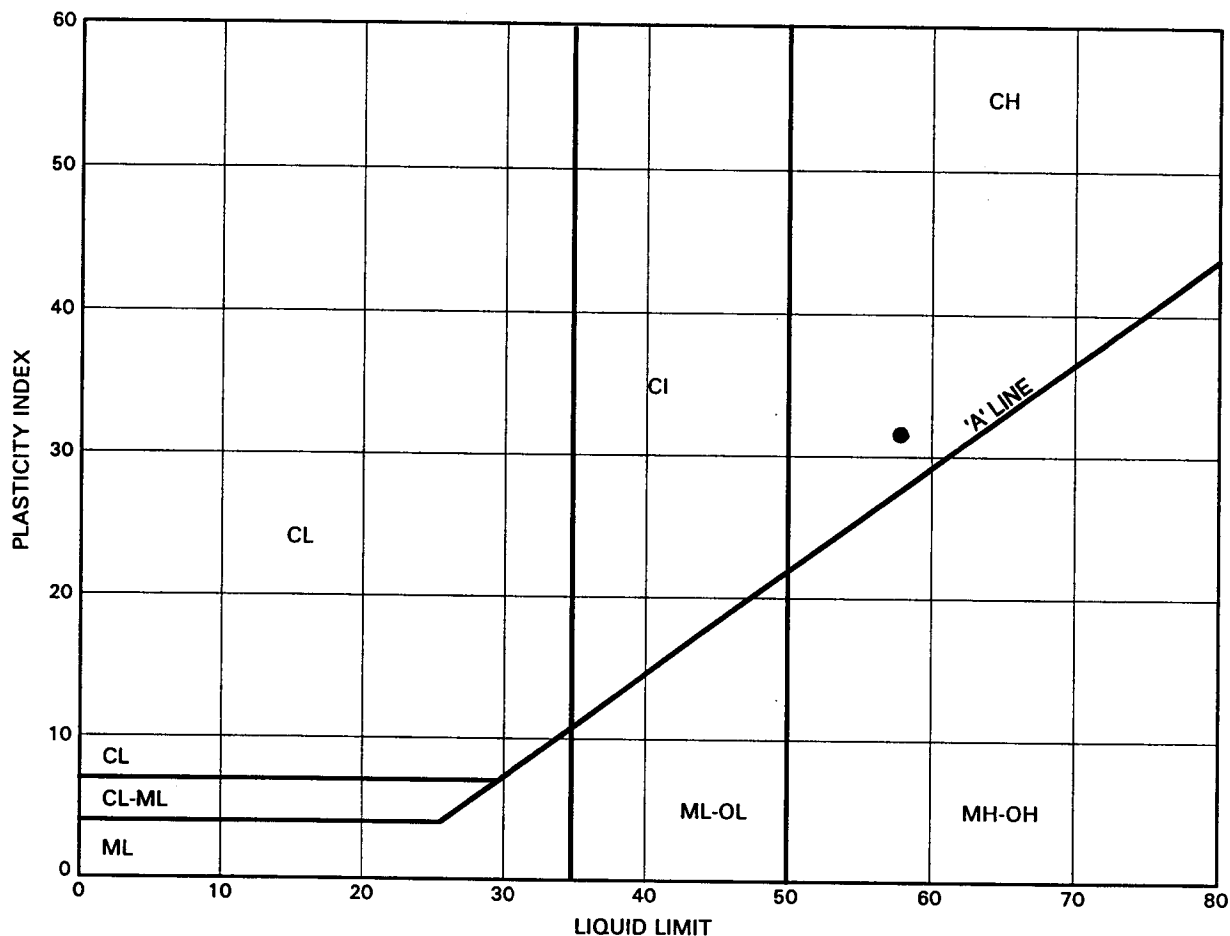
HIGHWAY 417, MISSISSIPPI RIVER BRIDGE

Atterberg Limits Results

MISSISSIPPI RIVER BRIDGE INVESTIGATION
ATTERBERG LIMITS TEST RESULTS

FIGURE C1

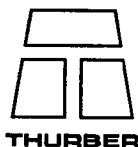
Dessicated Crust-Upper Silty Clay West Approach



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-9	3.28	83.88

Date May 1995

Project 451-90-03/04



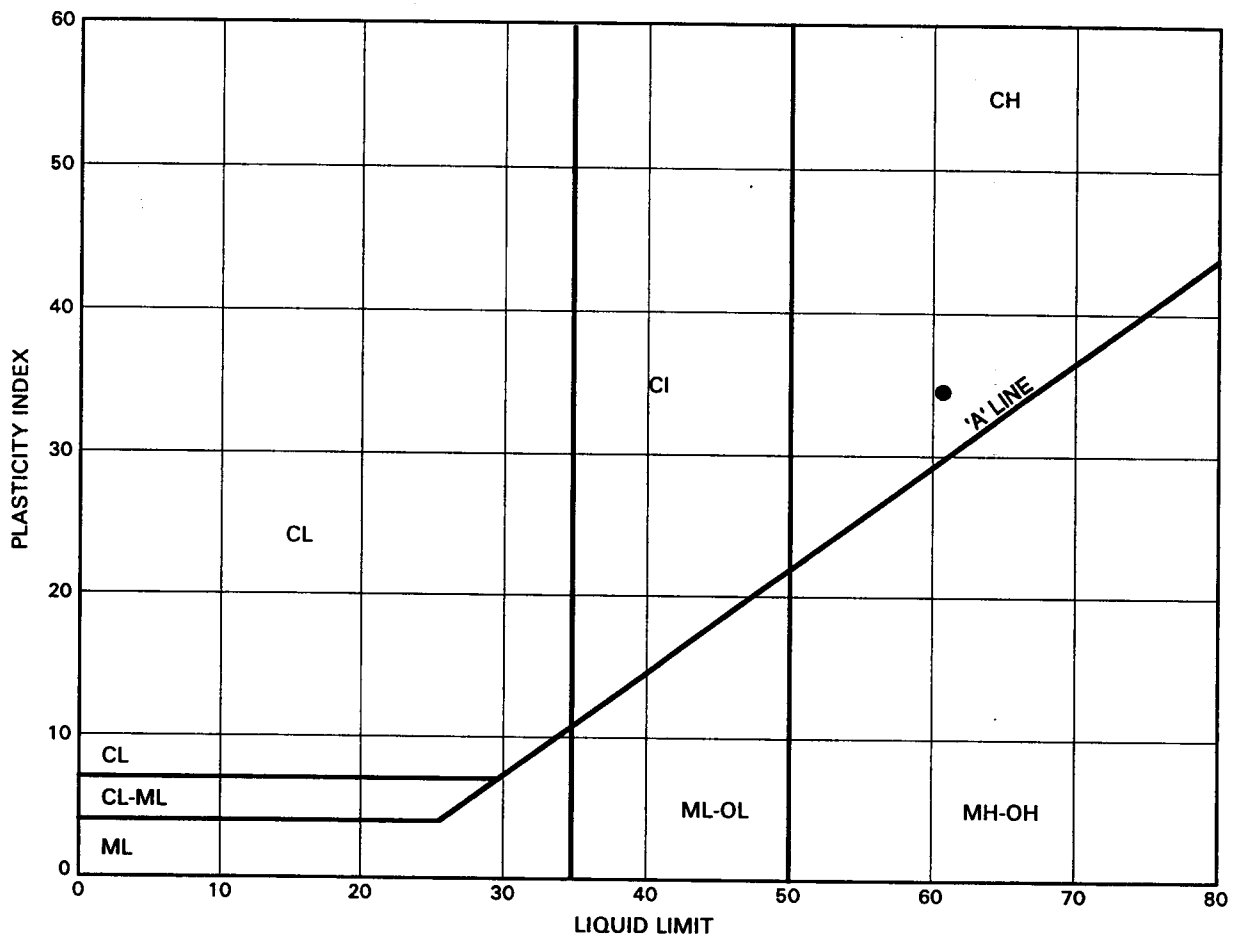
Prep'd WM

Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION
ATTERBERG LIMITS TEST RESULTS

FIGURE C2

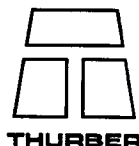
Dessicated Crust-Upper Silty Clay East Approach



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-7	3.28	84.78

Date May 1995

Project 451-90-03/04



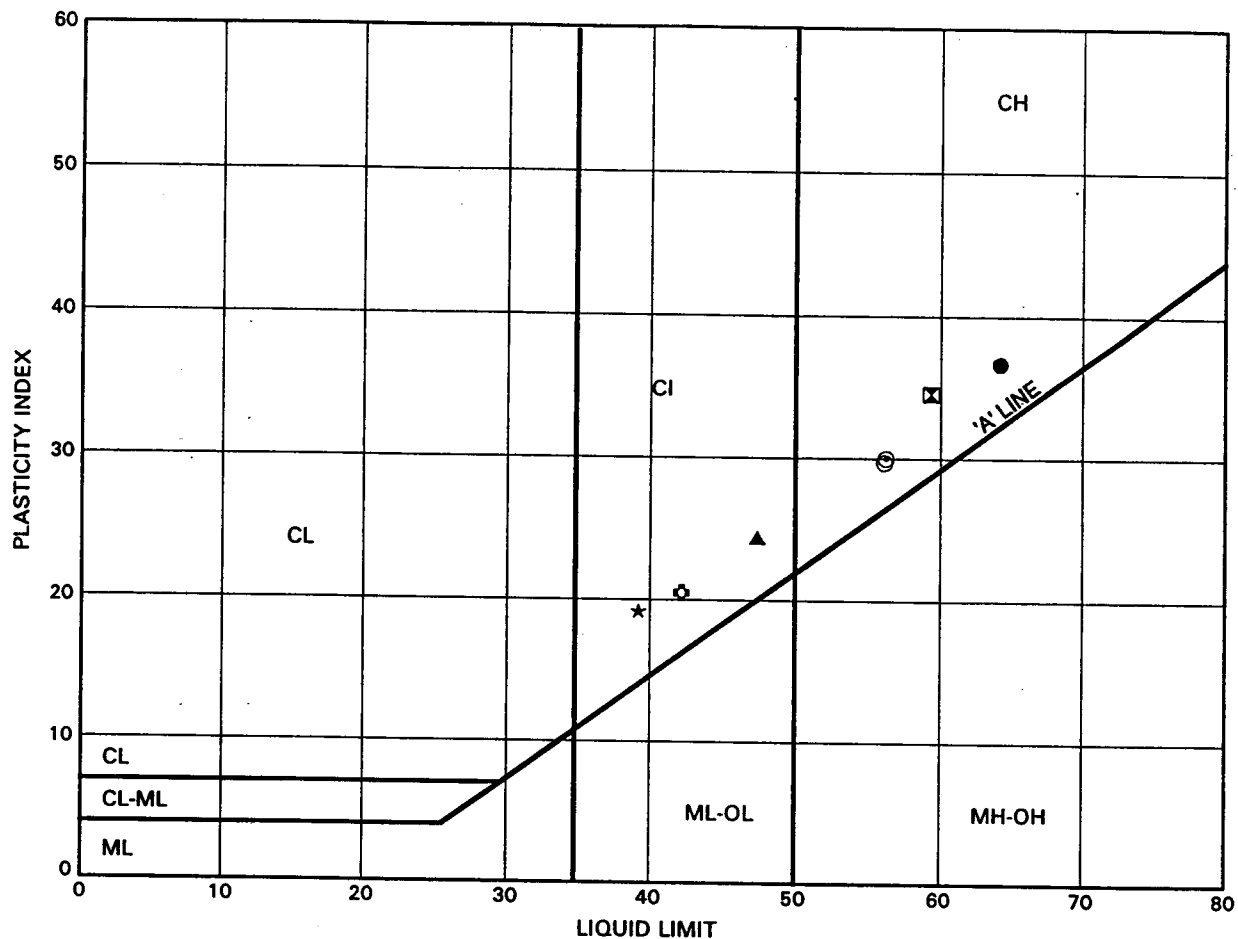
Prep'd WM

Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION ATTERBERG LIMITS TEST RESULTS

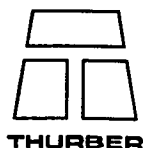
FIGURE C3

Upper Silty Clay - WBL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-1	9.37	82.08
⊠	95-1	12.42	79.03
▲	95-3	6.33	77.97
★	95-3	12.42	71.88
⊙	95-6	3.28	80.41
⊕	95-6	15.54	68.15
◦	95-7	7.85	80.25

Date May 1995
Project 451-90-03/04

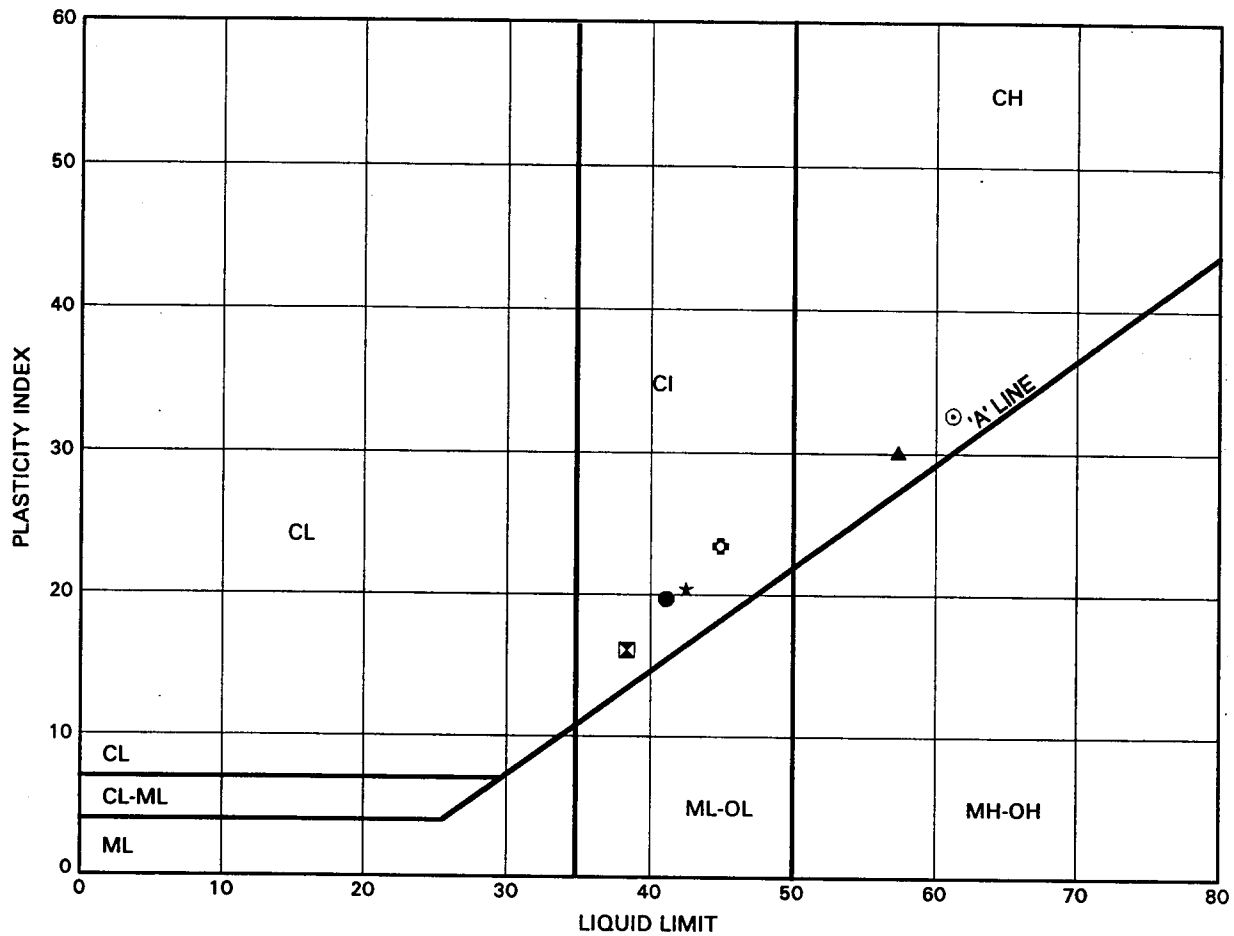


Prep'd WM
Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION ATTERBERG LIMITS TEST RESULTS

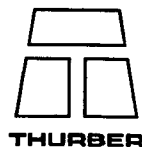
FIGURE C4

Upper Silty Clay - EBL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-10	12.42	71.06
⊠	95-12	18.52	64.93
▲	95-13	3.28	80.47
★	95-14	12.42	75.93
⊙	95-8	4.80	86.30
⊛	95-9	15.54	71.62

Date May 1995
Project 451-90-03/04

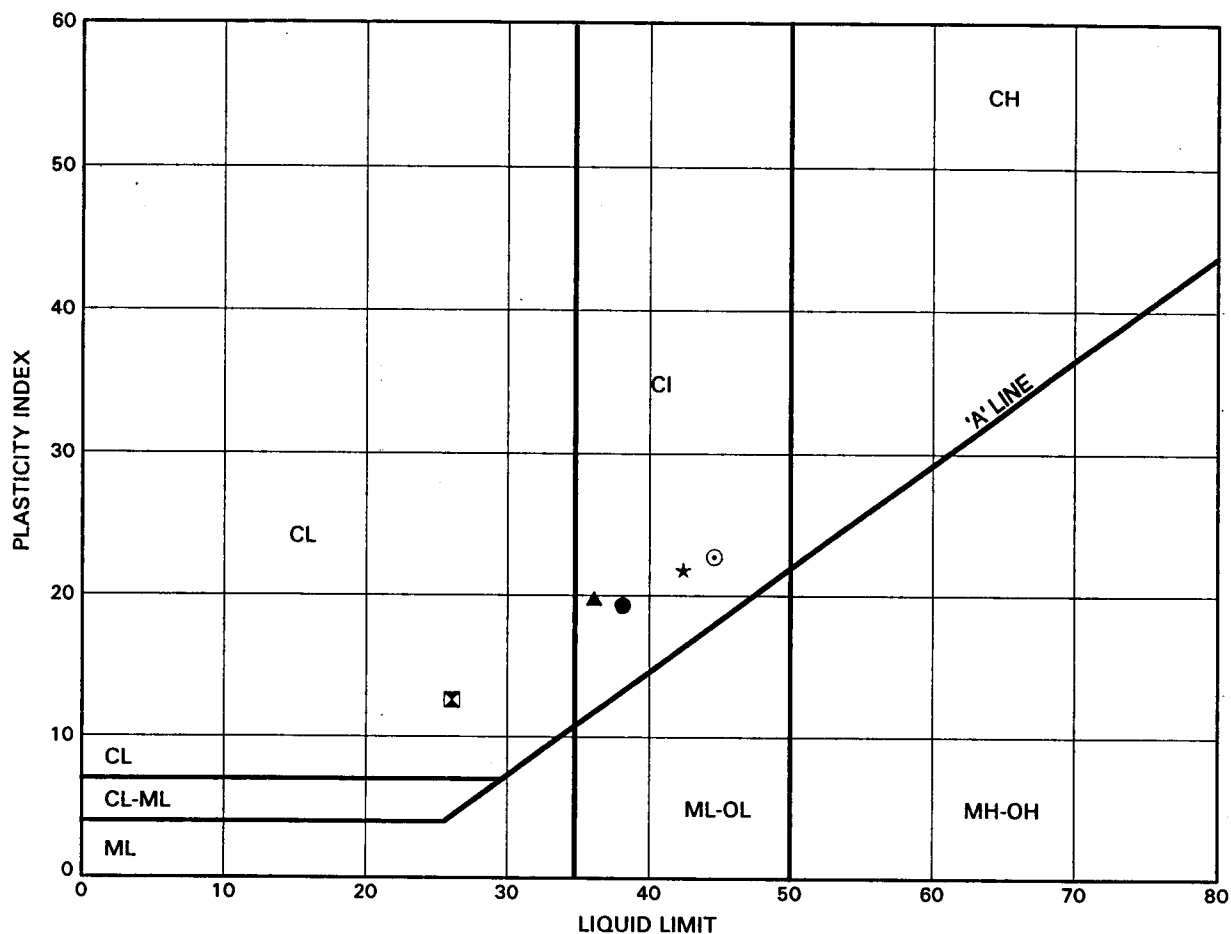


Prep'd WM
Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION
ATTERBERG LIMITS TEST RESULTS

FIGURE C5

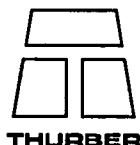
Lower Silty Clay - WBL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-2	24.69	62.23
⊠	95-2	42.90	44.02
▲	95-4	26.14	57.30
★	95-4	35.28	48.16
⊙	95-5	30.71	52.69

THURBALT 4001 95/05/26

Date May 1995
 Project 451-90-03/04

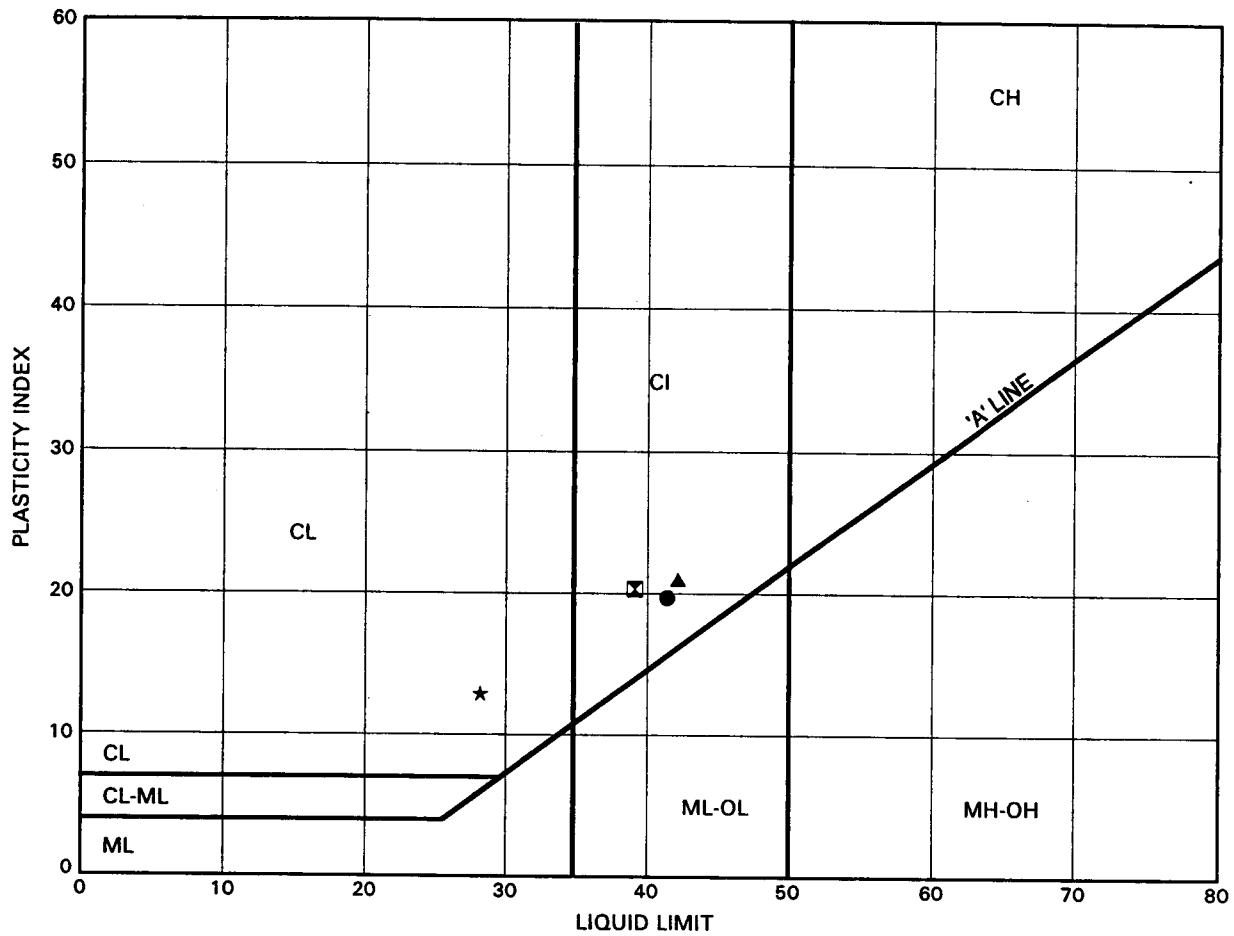


Prep'd WM
 Chkd. IC

MISSISSIPPI RIVER BRIDGE INVESTIGATION
ATTERBERG LIMITS TEST RESULTS

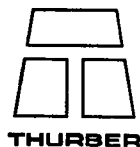
FIGURE C6

Lower Silty Clay - EBL

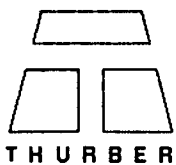
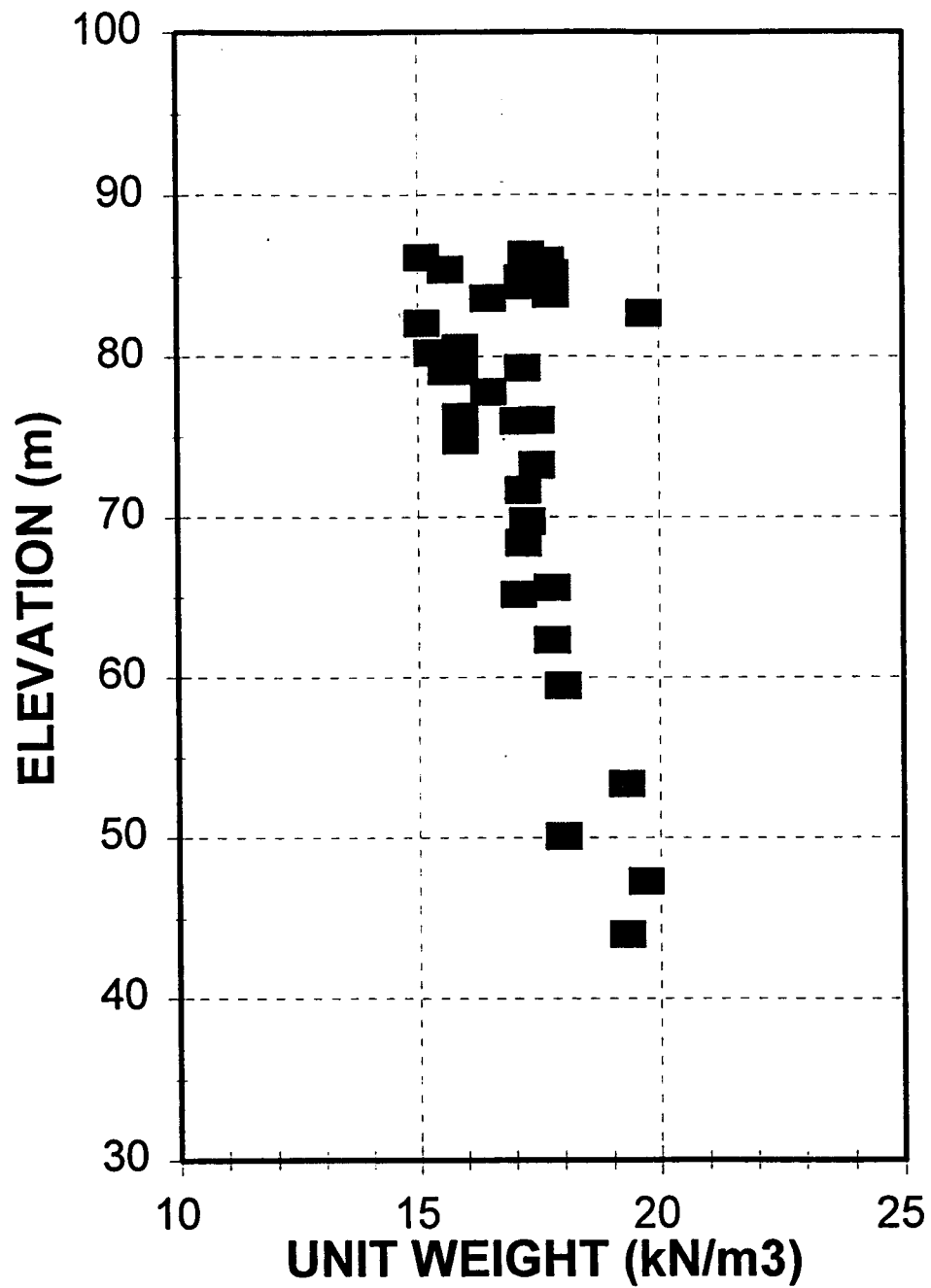


SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	95-11	26.14	57.34
⊠	95-13	24.69	59.06
▲	95-13	36.81	46.94
★	95-9	45.95	41.21

Date May 1995
 Project 451-90-03/04



Prep'd WM
 Chkd. IC



HIGHWAY 417/MISSISSIPPI RIVER BRIDGES
SUMMARY OF SILTY CLAY
UNIT WEIGHT DATA VERSES DEPTH

FIGURE C7

APPENDIX D

HIGHWAY 417, MISSISSIPPI RIVER BRIDGE

Consolidation Test Results

VOID RATIO - PRESSURE CURVES

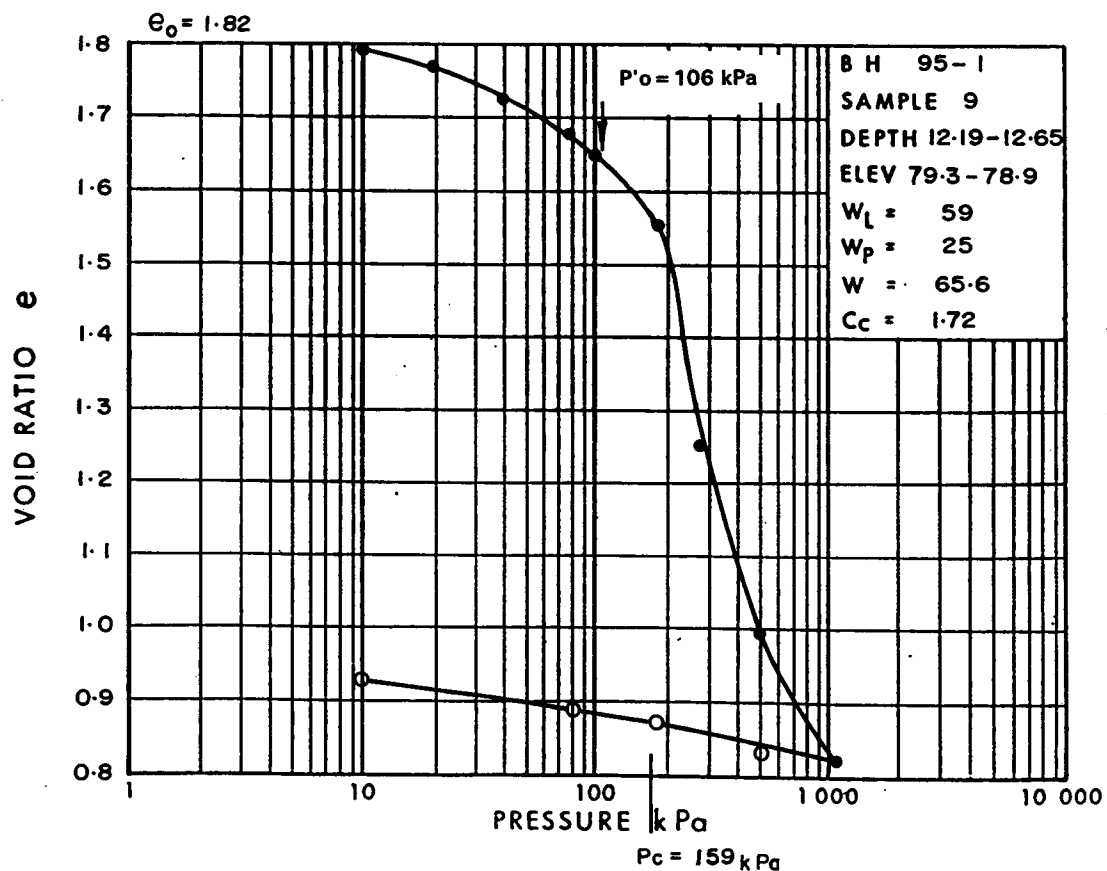
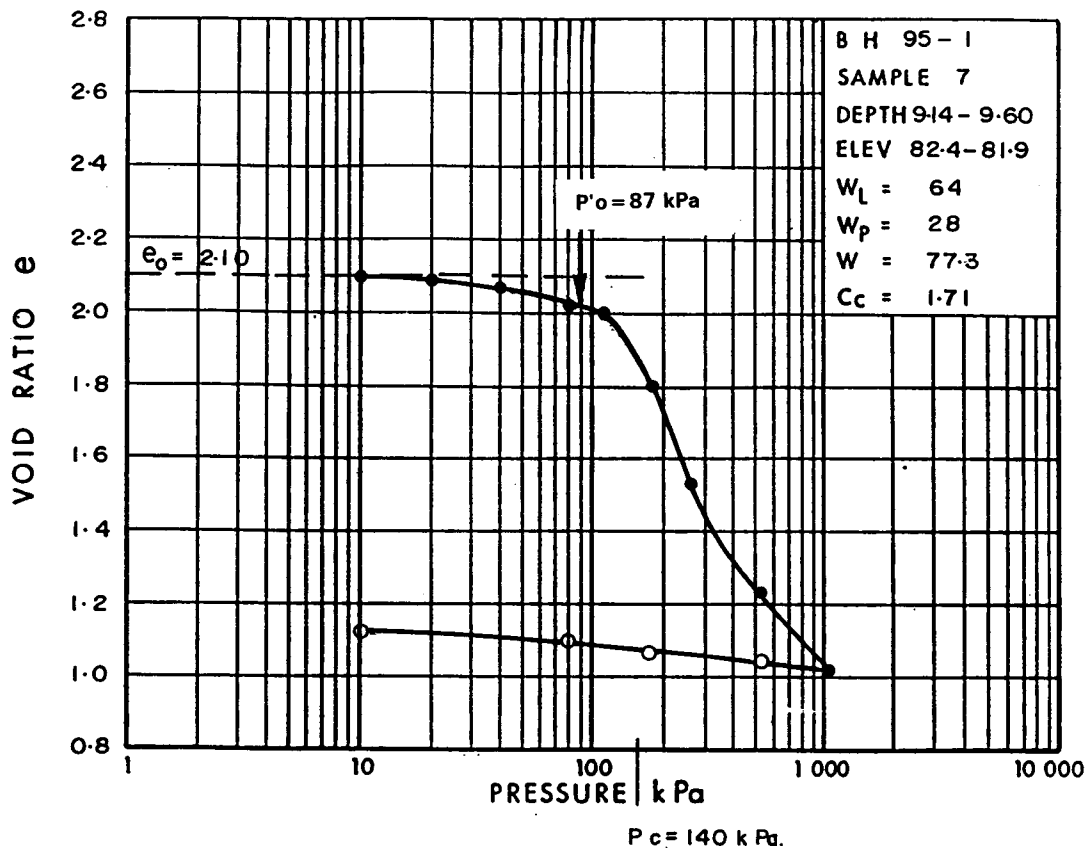


Fig D-1

VOID RATIO - PRESSURE CURVES

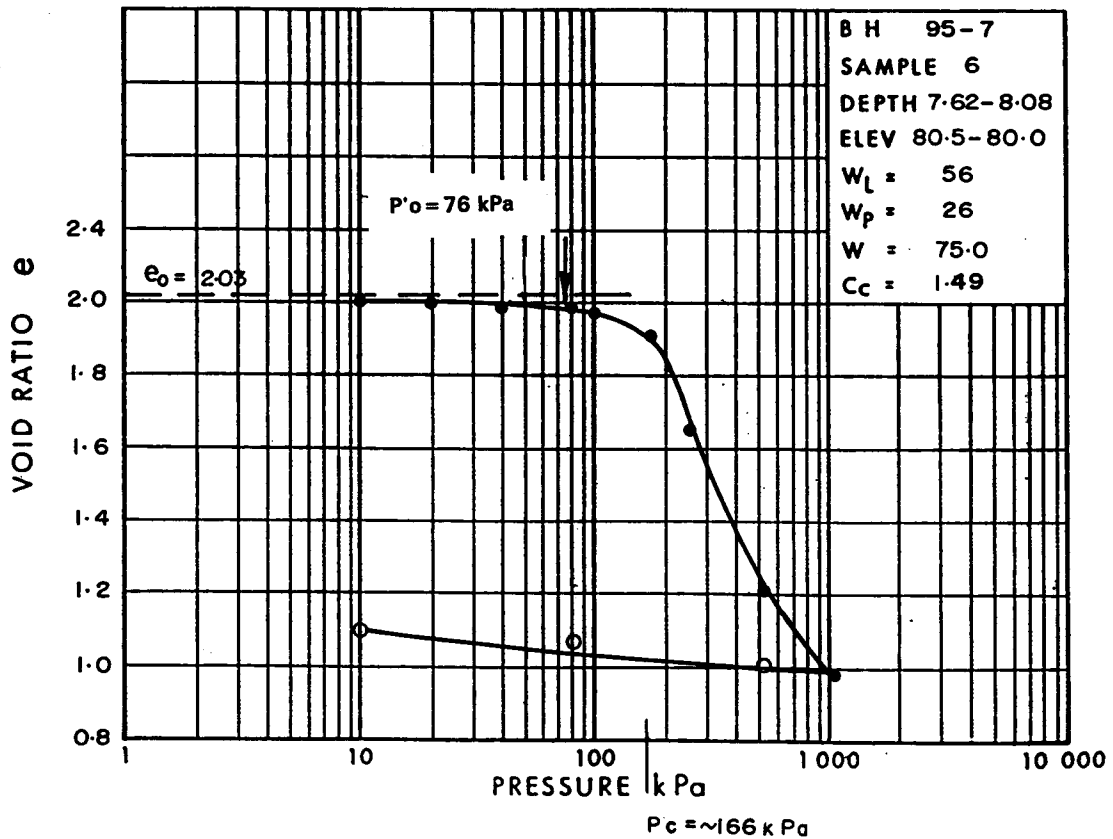
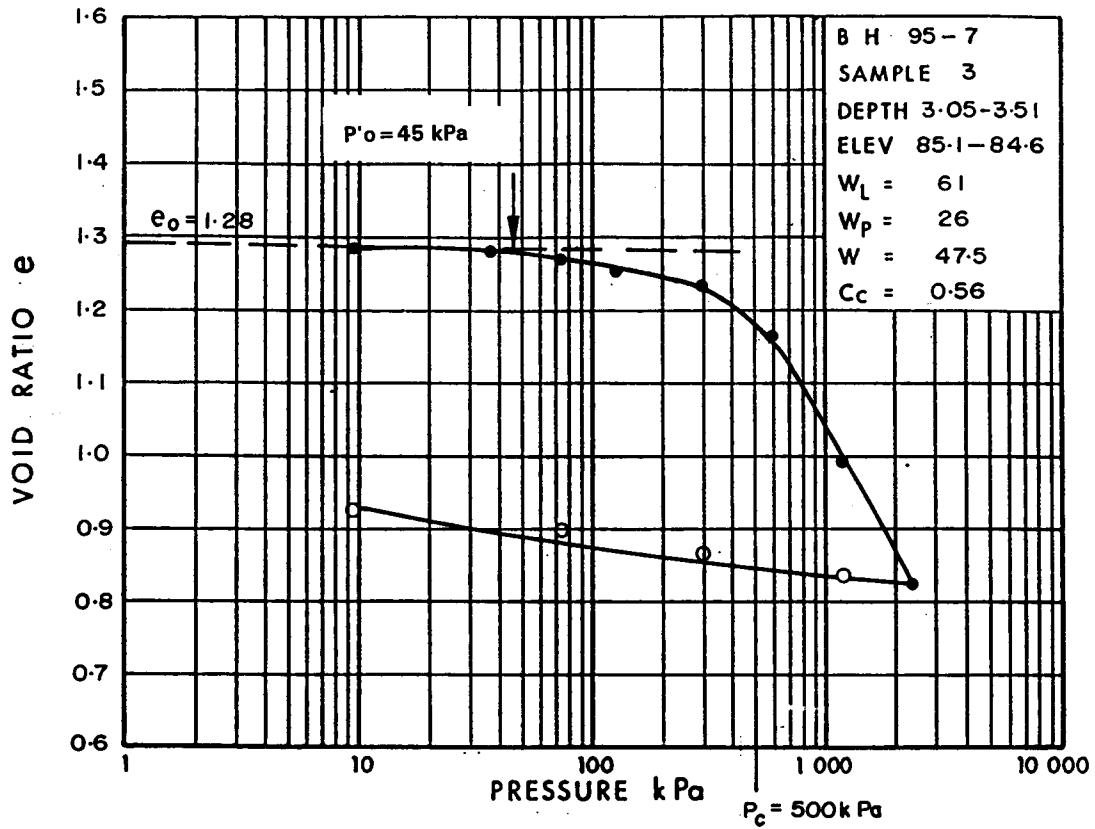


Fig D-2

W P 451-90-03/04

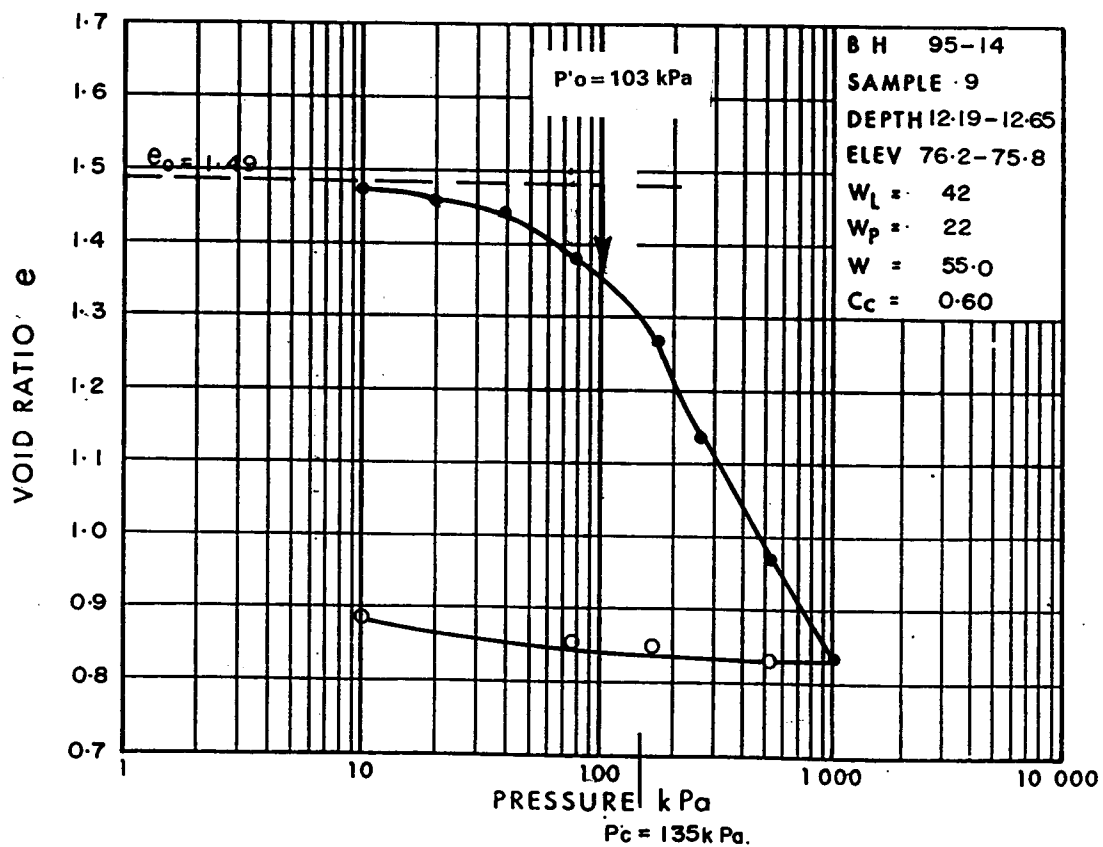
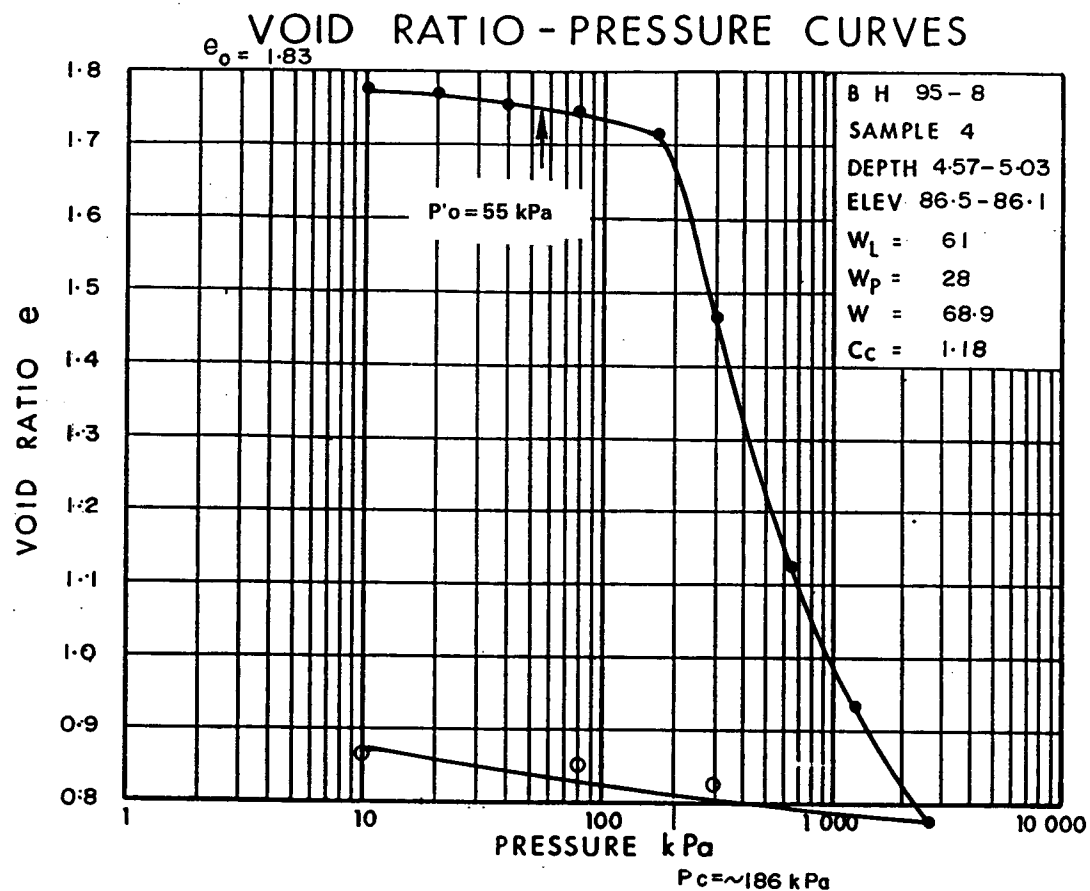


Fig D-3

APPENDIX E

HIGHWAY 417, MISSISSIPPI RIVER BRIDGE

Selected Stability Analyses

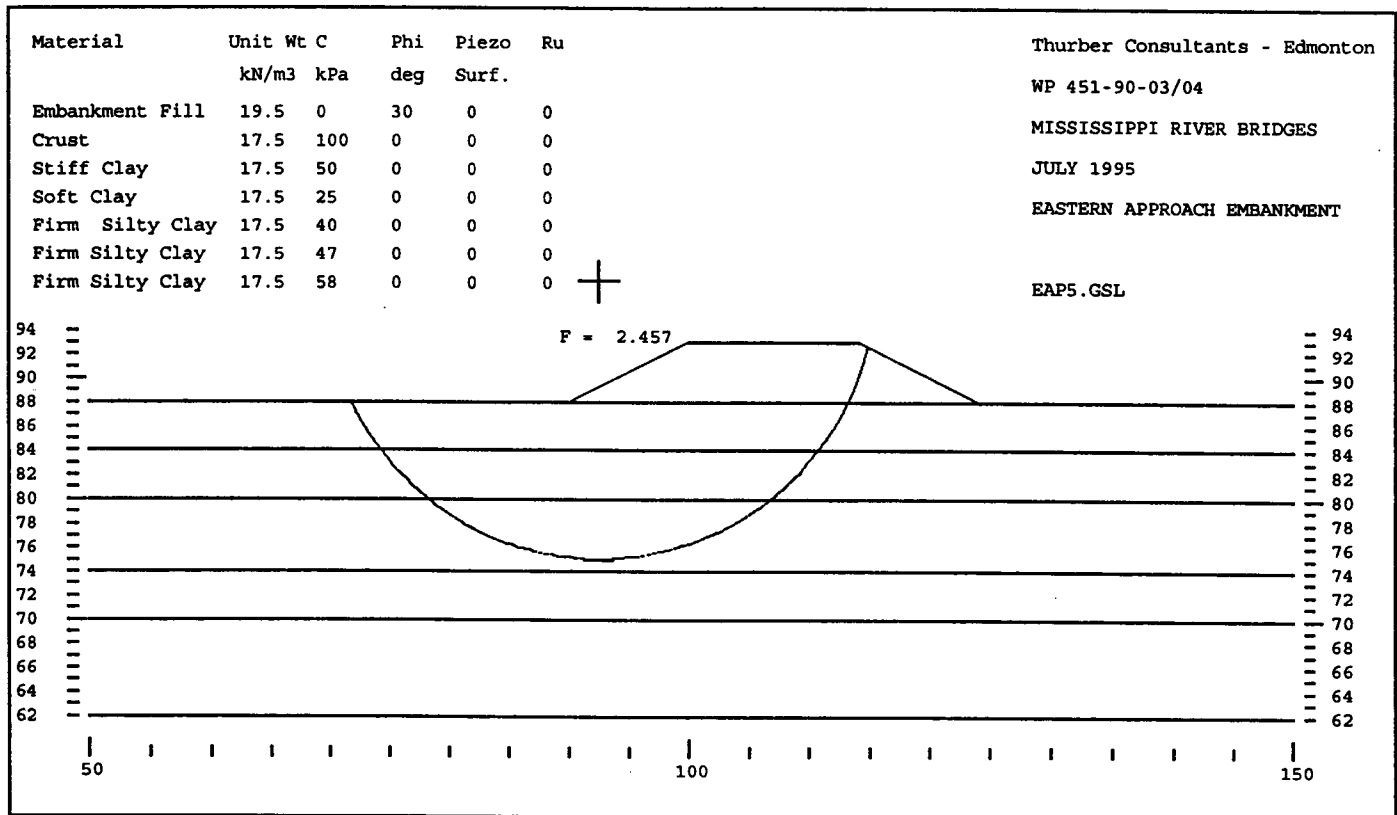


Figure E-1: Results of Stability Analysis for the East Approach, 5m high embankment with 2H:1V side slopes

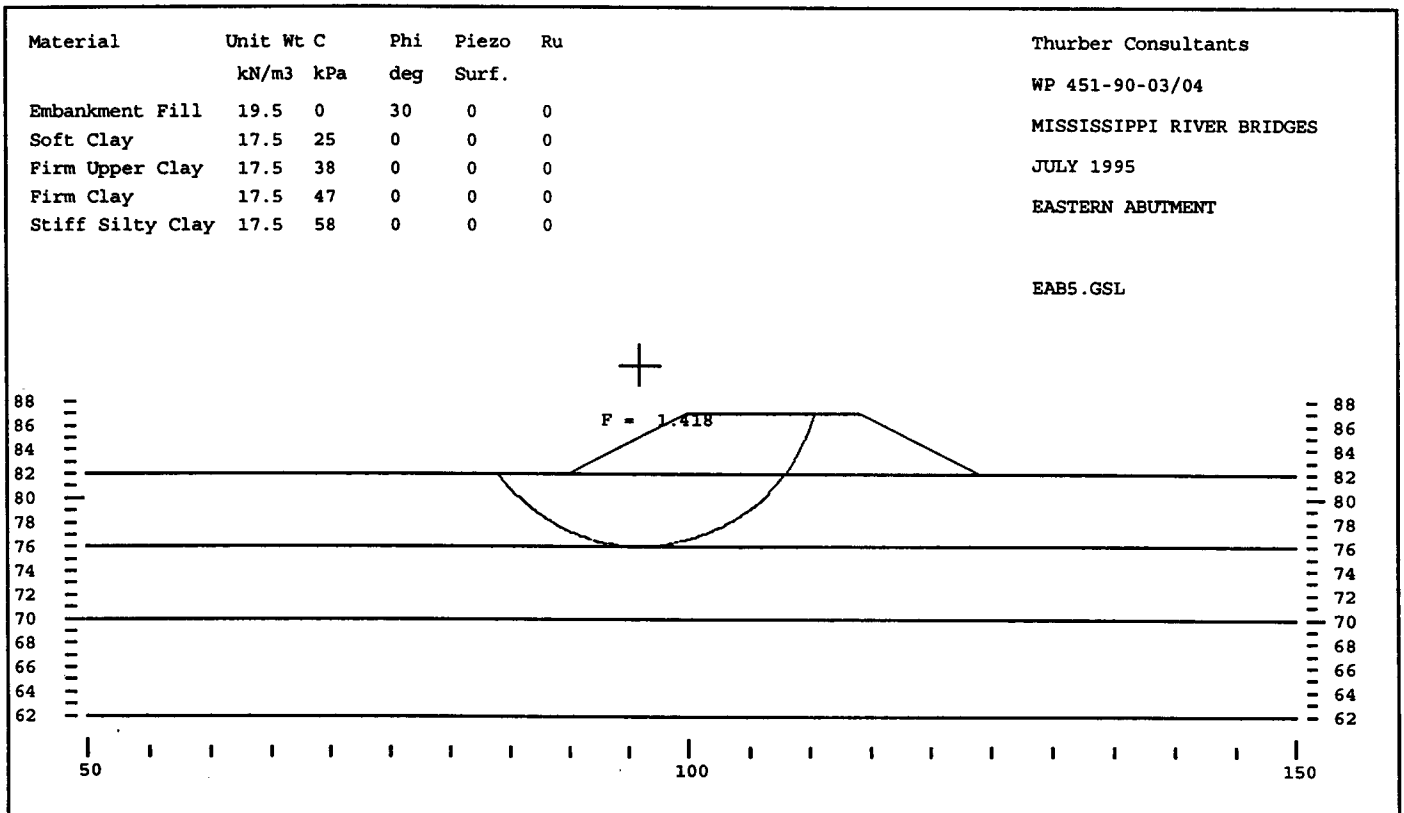


Figure E-2: Results of Stability Analysis for the East Abutment, 5m high embankment with 2H:1V side slopes

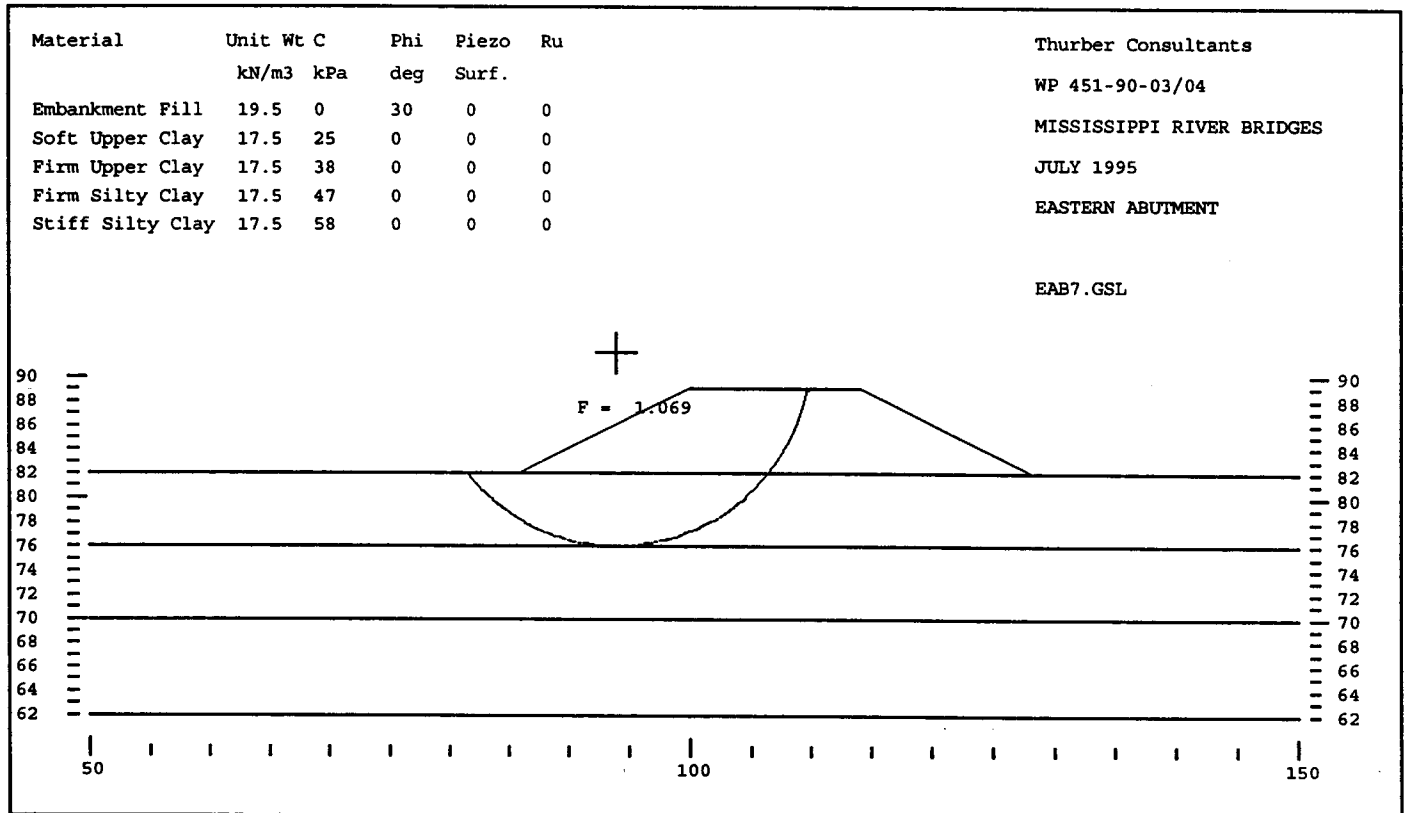


Figure E-3: Results of Stability Analysis for the East Abutment, 7m high embankment with 2H:1V side slopes

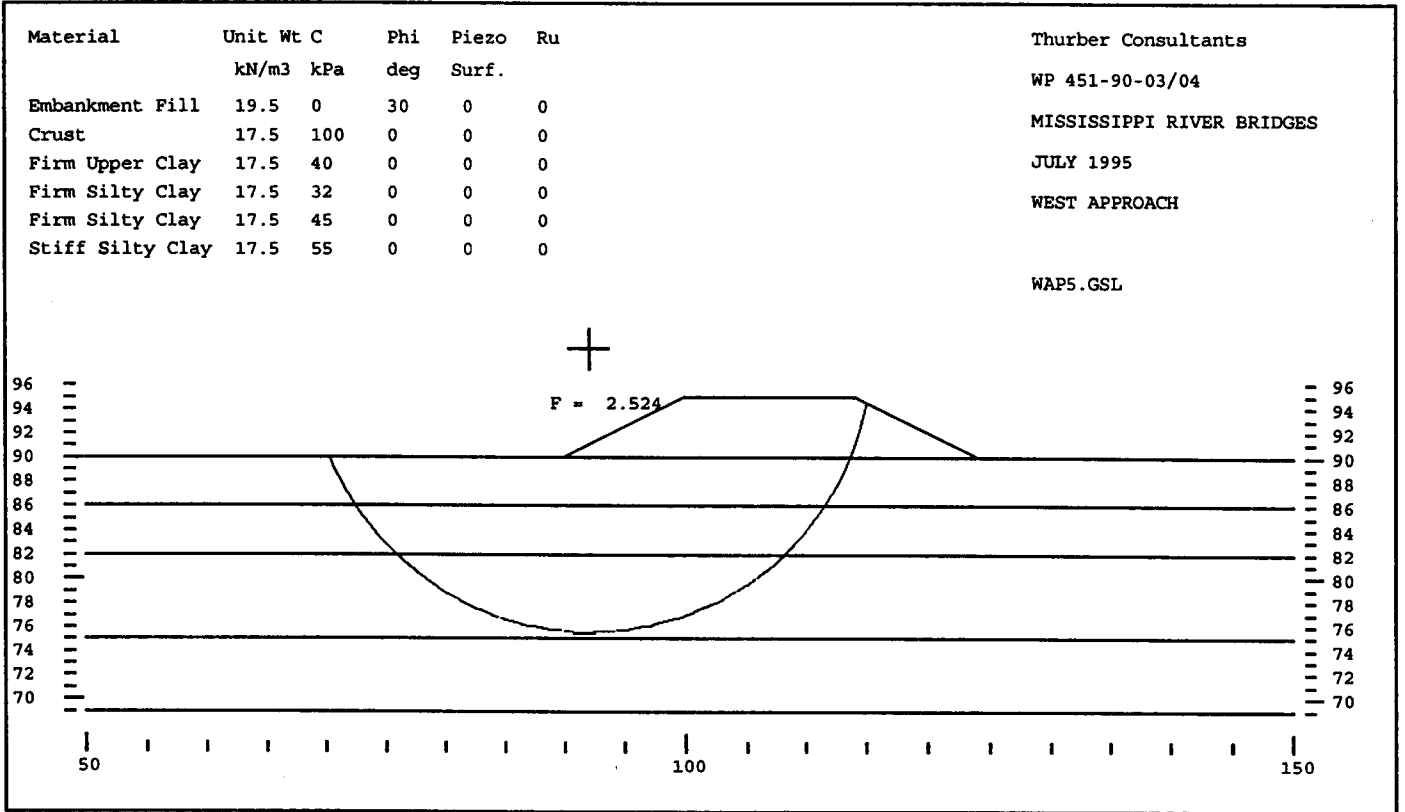


Figure E-4: Results of Stability Analysis for the West Approach, 5m high embankment with 2H:1V side slopes

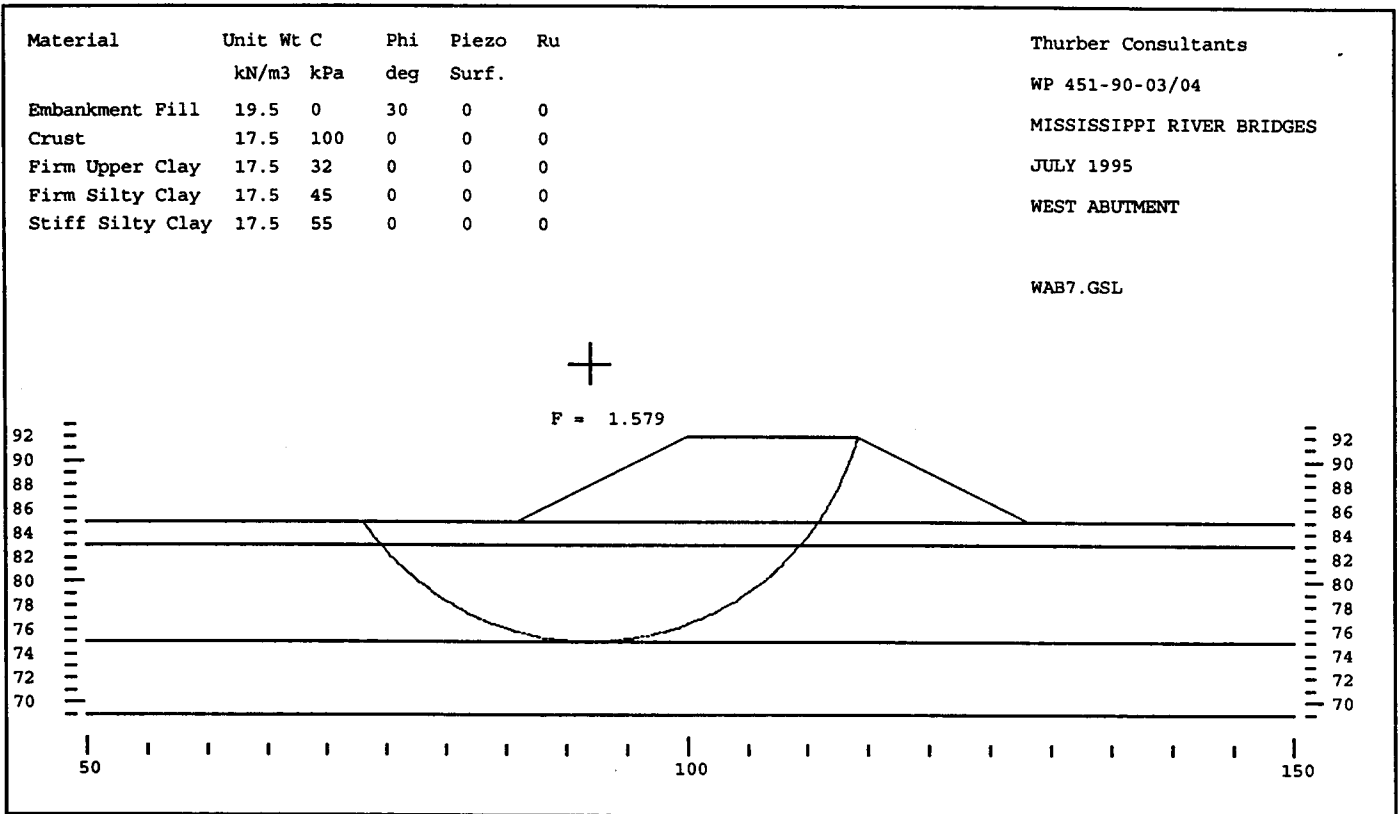


Figure E-5: Results of Stability Analysis for the West Abutment, 7m high embankment with 2H:1V side slopes

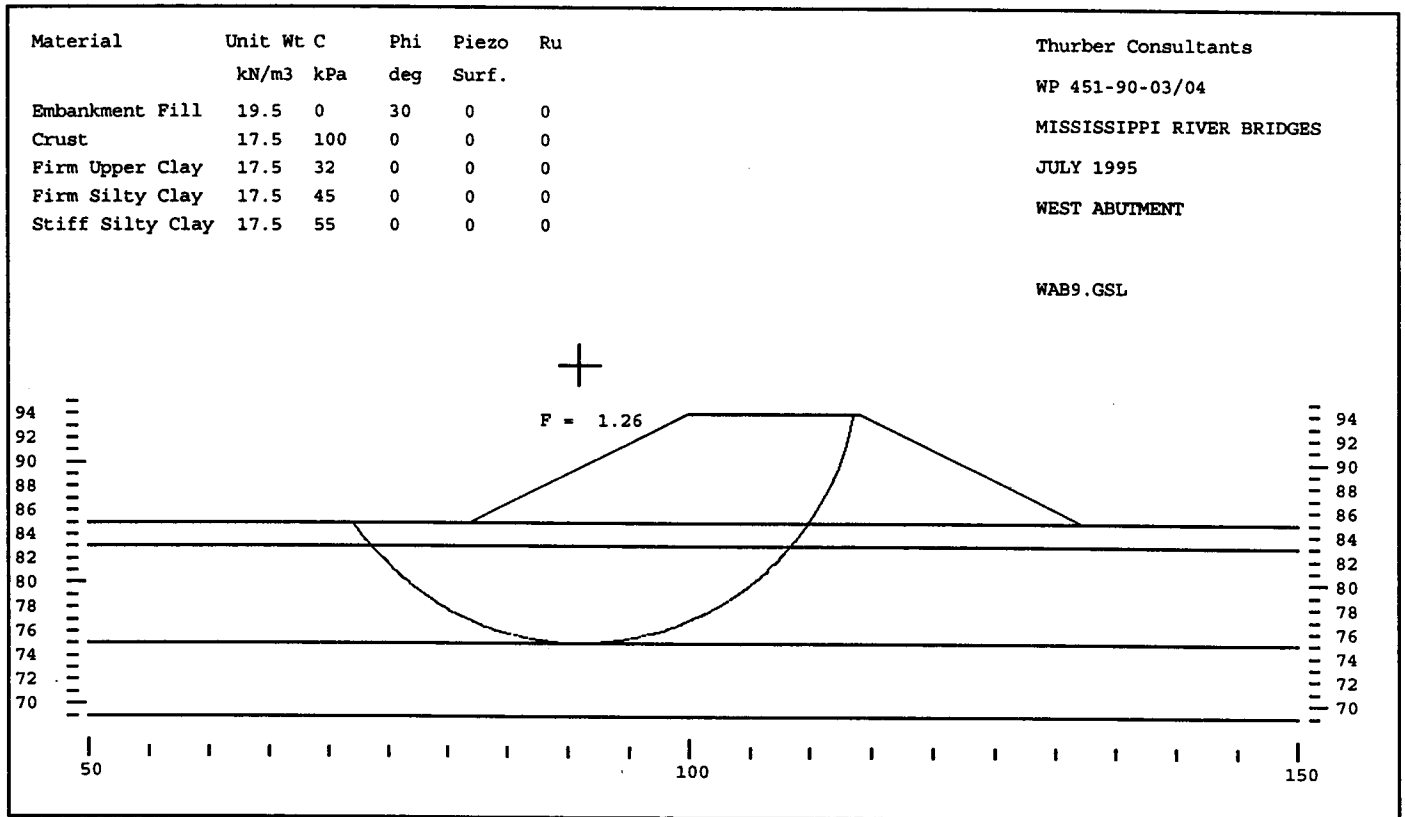


Figure E-6: Results of Stability Analysis for the West Abutment, 9m high embankment with 2H:1V side slopes

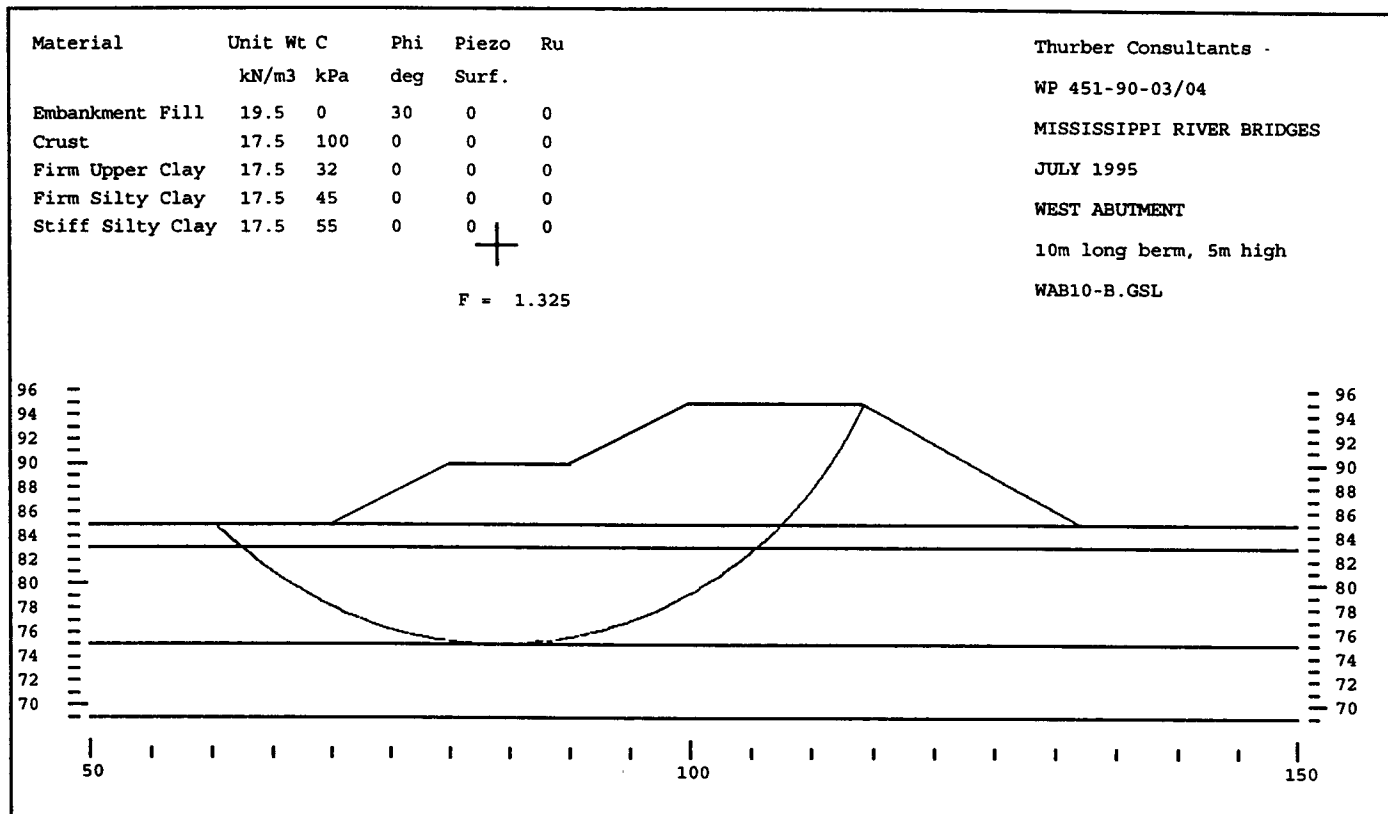


Figure E-7: Results of Stability Analysis for the West Abutment, 10m high embankment with 2H:1V side slopes and 10m long flanking berm, 5m high.

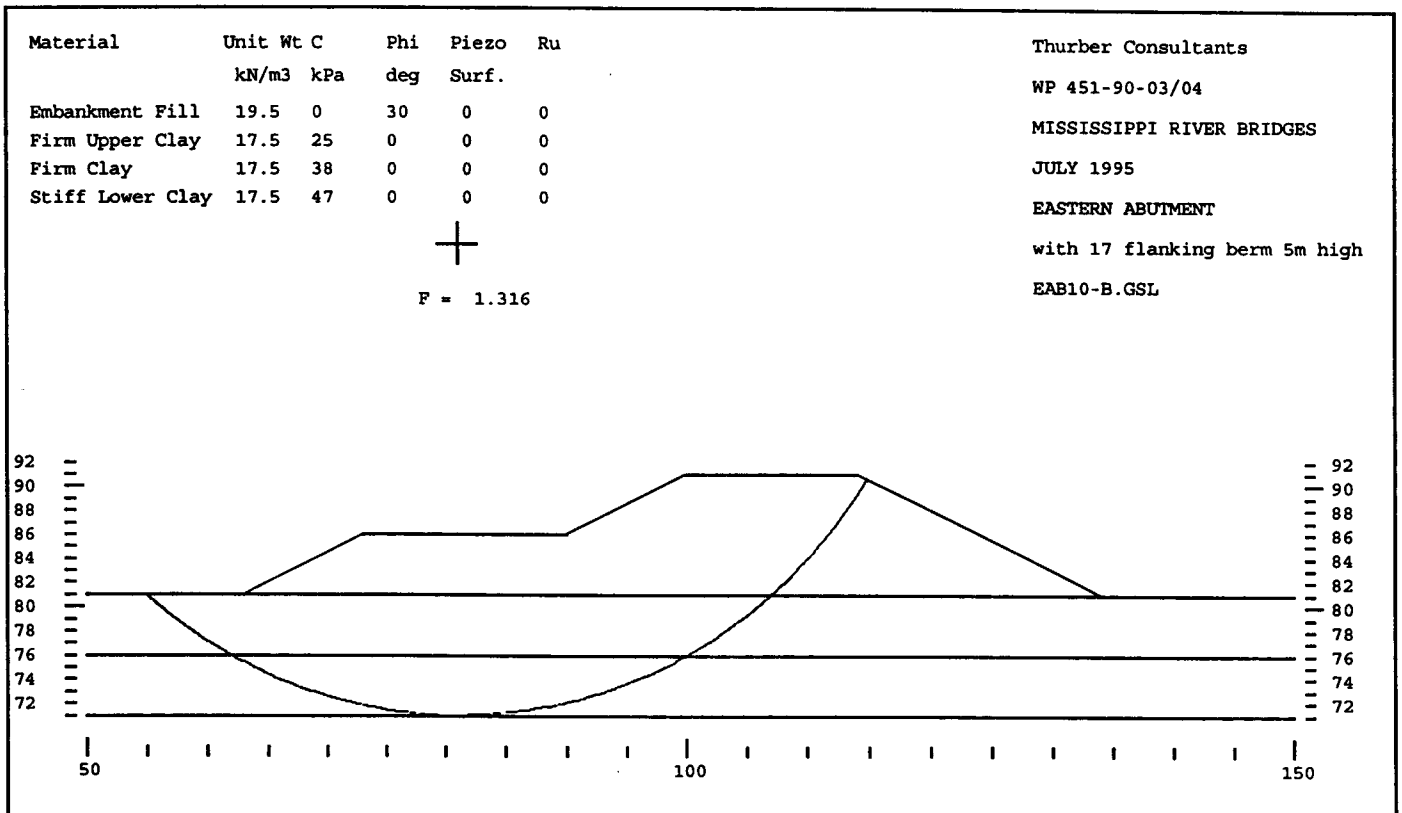
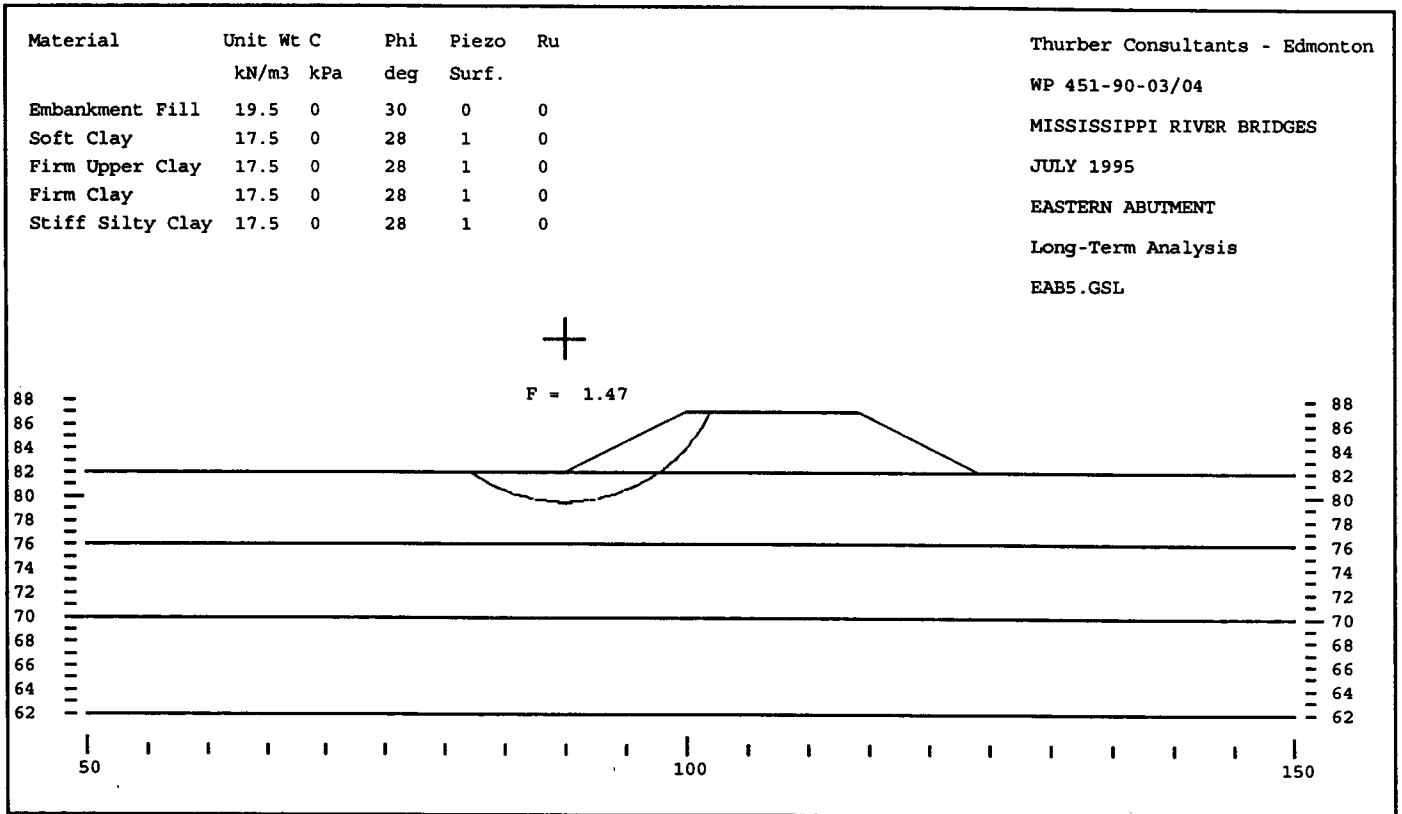


Figure E-8: Results of Stability Analysis for the East Abutment, 10m high embankment with 2H:1V side slopes and 17m long flanking berm, 5m high.



**Figure E-9: Results of Long -Term Stability Analysis,
 5m high Embankment with 2H:1V side slopes and
 Effective Friction Angle of 28 degrees in the Silty Clay**

APPENDIX F
HIGHWAY 417, MISSISSIPPI RIVER BRIDGE
Settlement Analyses

Highway 417 Mississippi River Bridge Eastern Approach Embankment

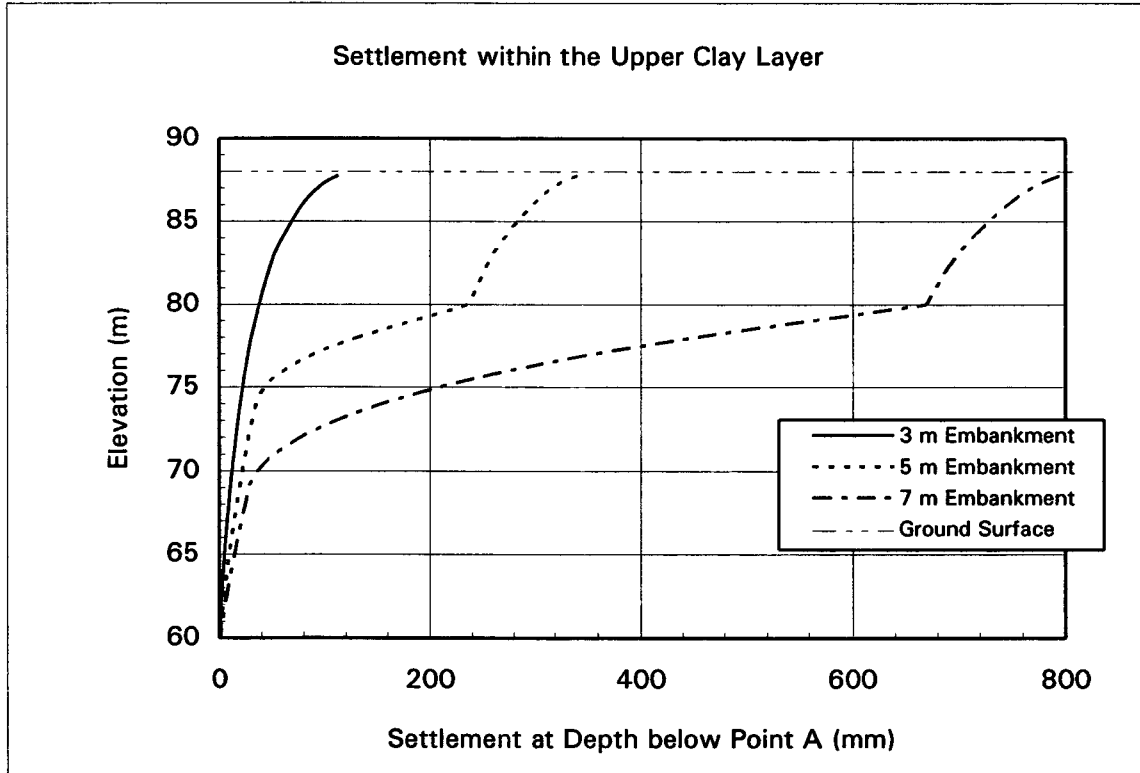
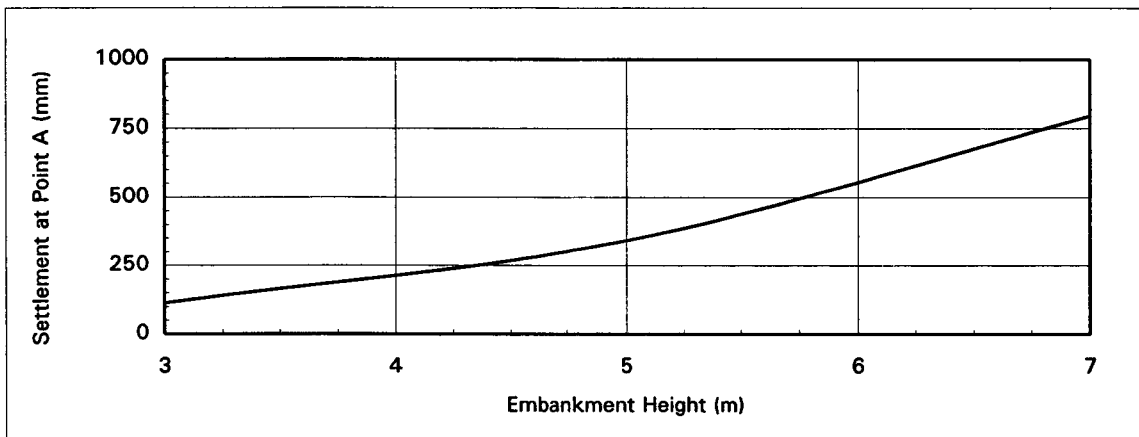
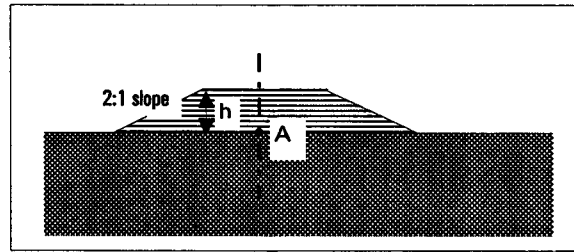


Figure F1

Highway 417 Mississippi River Bridge East Abutment Embankment

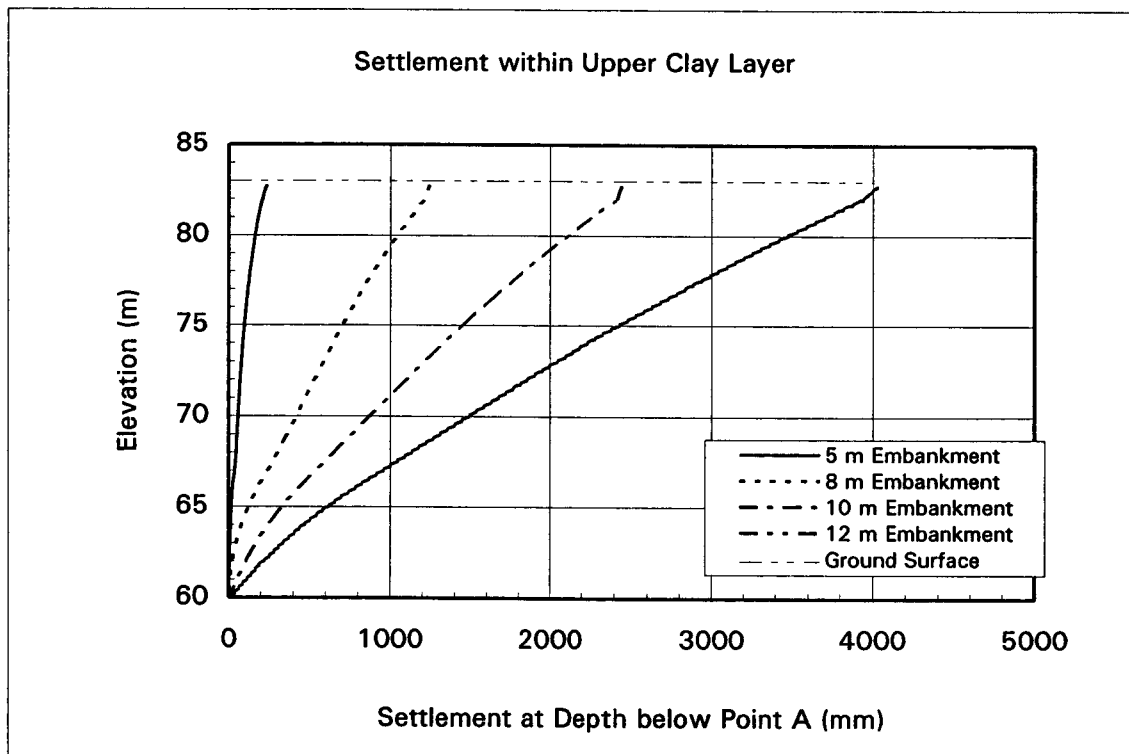
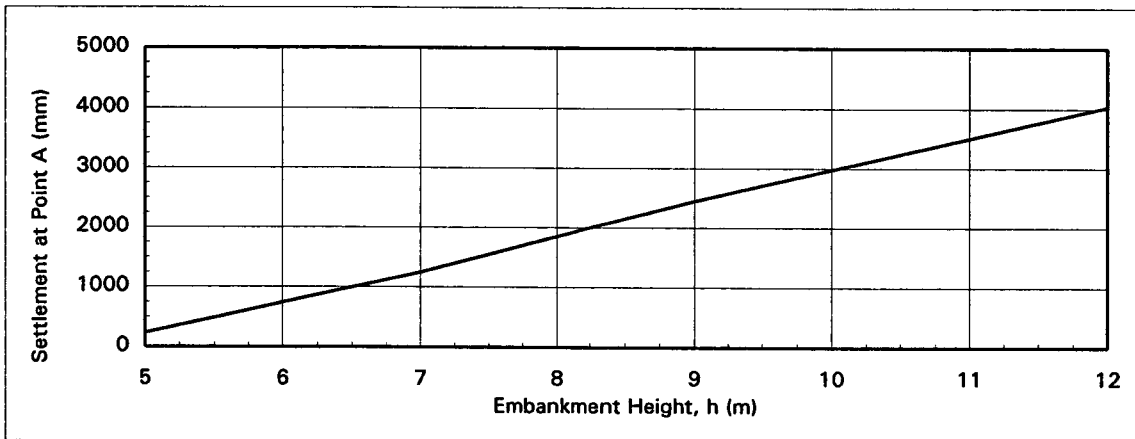
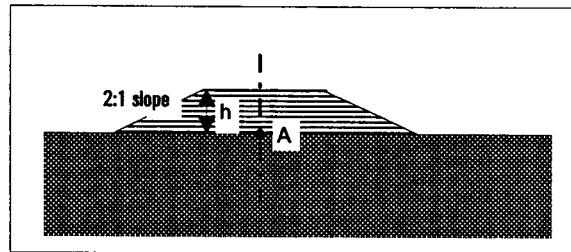


Figure F2

Highway 417 Mississippi River Bridge Western Approach Embankment

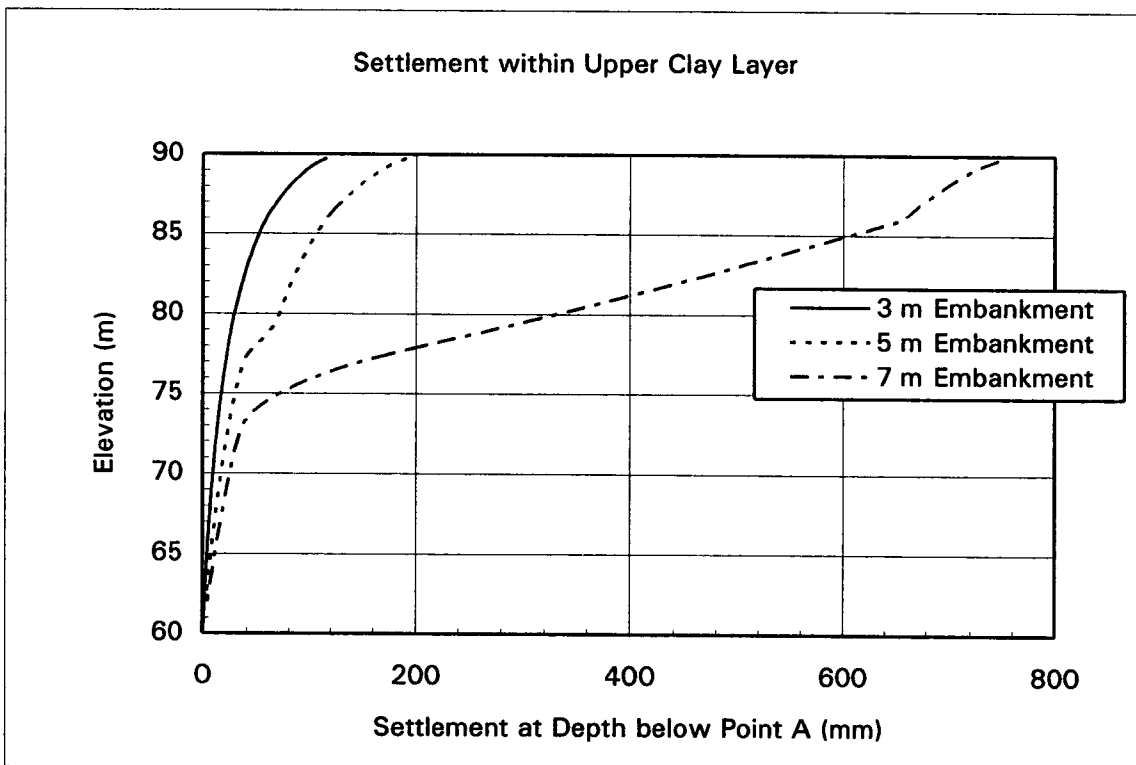
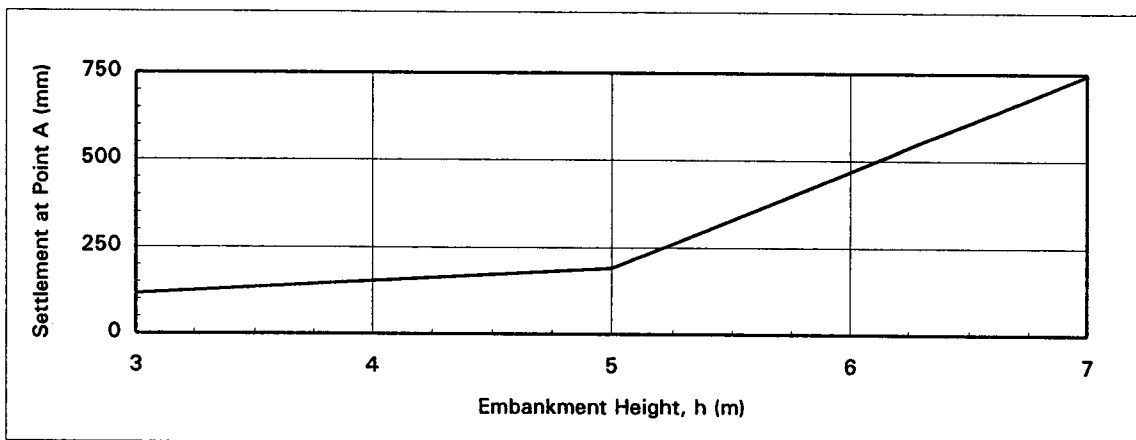
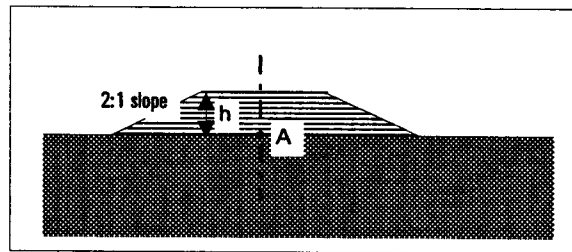


Figure F3

Highway 417 Mississippi River Bridge West Abutment Embankment

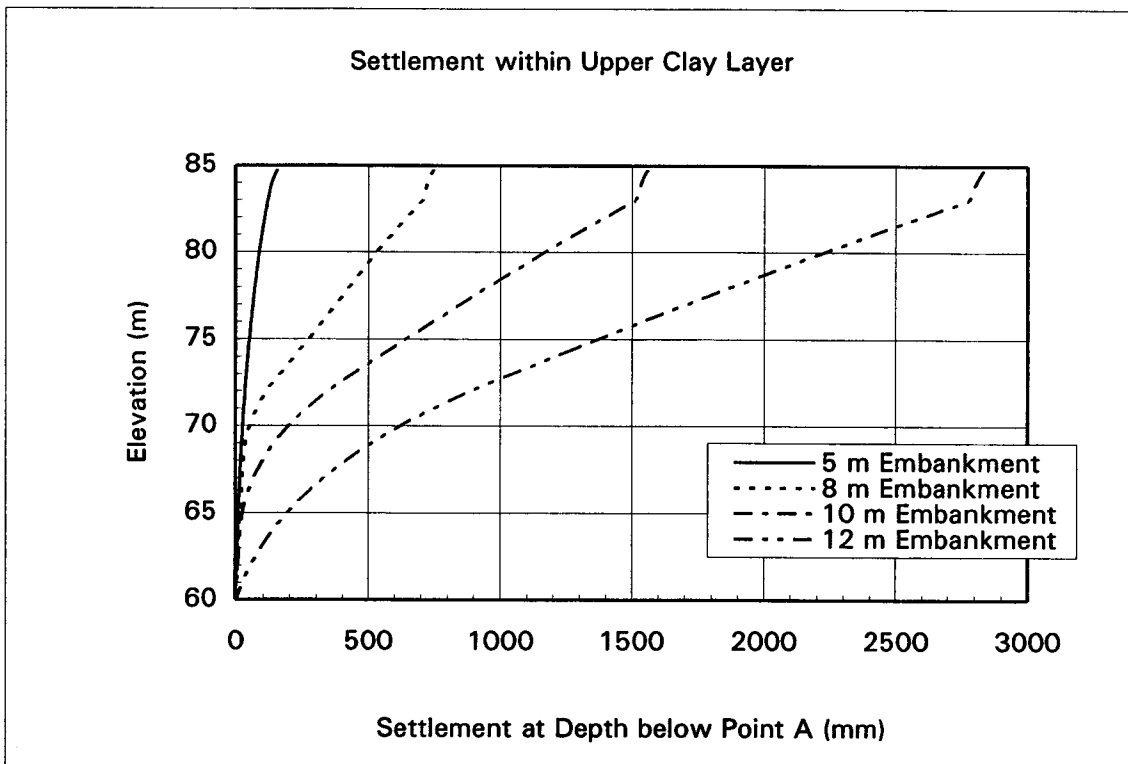
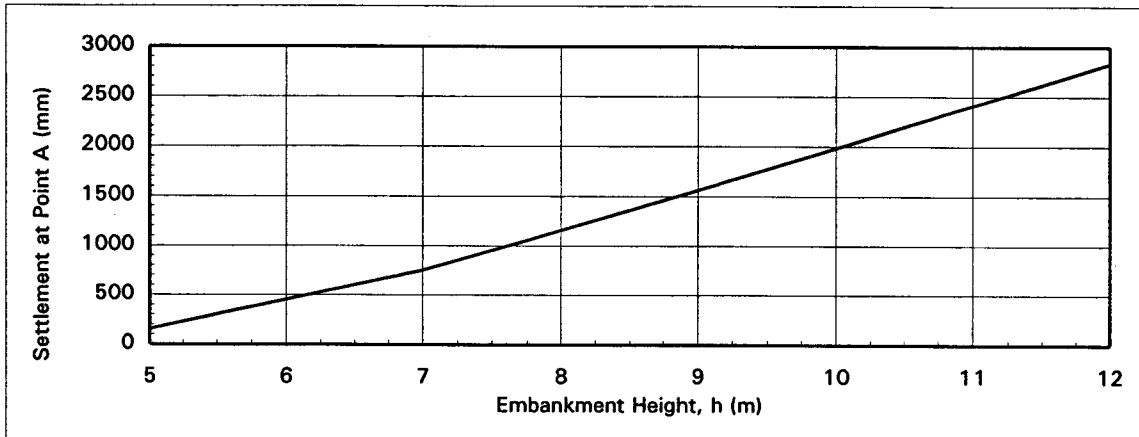
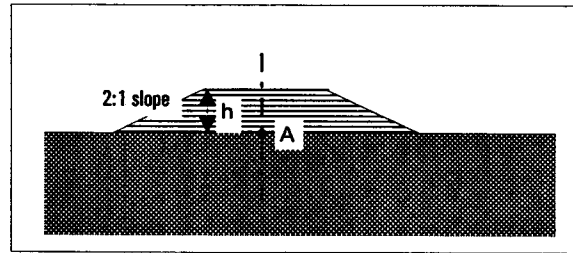


Figure F4

APPENDIX G

HIGHWAY 417, MISSISSIPPI RIVER BRIDGE

**Factual Report on Consolidation Test Results
as submitted by
Golder Associates Ltd., Mississauga, Ontario
dated May 1995**

Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561



**REPORT TO
Thurber Engineering**

**FACTUAL REPORT
GEOTECHNICAL LABORATORY TESTING
YOUR PROJECT 15-64-1**

Submitted to:

**Thurber Engineering
170 Evans Avenue
Suite 101
Etobicoke, Ontario
M8Z 5Y6**

Distribution:

**2 copies - Thurber Engineering
Etobicoke, Ontario**

**2 copies - Golder Associates Ltd.
Mississauga, Ontario**

May 1995

951-1323

Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561



May 12, 1995

951-1323

Thurber Engineering
170 Evans Avenue Suite 101
Etobicoke, Ontario
M8Z 5Y6

Attention: Mr. Ivan Corbett P.Eng.

RE: GEOTECHNICAL LABORATORY TESTING
Your Project 15-64-1

Dear Sirs:

This factual report presents and summarizes the results of a consolidation (oedometer) laboratory testing program which was carried out on six shelly tube samples. The samples were obtained by Thurber Engineering and delivered to Golder Associates Ltd. Mississauga, Ontario laboratory on April 11, 1995.

The consolidation tests were carried out in accordance with the ASTM D 2435-90, standard. Each test was carried out using a 24 hour loading duration for the load program as defined by Thurber Engineering with unloading in short time stages. Upon completion, conventional time fitting methods for the determination of t_{90} were determined and sent to Thurber Engineering for approval, prior to finalizing the calculations for coefficient of consolidation (cv), coefficient of volume decrease (mv) and hydraulic conductivity (k). The results of these tests are summarized on the attached figures 1-1 through 6-5 inclusive.

Golder Associates terms of reference for this project is to provide only a testing service for your defined test program. There is no engineering assessment expressed or implied by Golder Associates in regard to the test results obtained or the suitability of the test results for engineering purposes.

We trust that this report is satisfactory for your immediate purposes. Should you have any questions concerning the report, the laboratory test program carried out, the results obtained, or if you require any additional information, please do not hesitate to contact us.

Yours truly,

GOLDER ASSOCIATES LTD.

A handwritten signature in dark ink, appearing to read 'F. A. Rydgren', with a long horizontal flourish extending to the right.

F. A. Rydgren

Laboratory Supervisor

CONSOLIDATION SUMMARY

FIGURE 1-1

PROJECT 951-1323	SPECIFIC GRAVITY 2.70 assumed	DATE START 95-04-25
SAMPLE BH 95-1 / SA 7	DRY WEIGHT, gm 52.15	DATE COMPLETED 95-05-08
AREA (mm ²) 3166.03	SOLIDS HT. 2HS 6.101	

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	t50 sec	cv. t90 cm ² /s	t50	k cm/s	mv m ² /kN
0.00	18.908	2.099	18.908						
9.65	18.838	2.088	18.873	91		8.30E-03		3.14E-07	3.86E-04
19.31	18.786	2.079	18.812	94		7.98E-03		2.21E-07	2.83E-04
38.62	18.662	2.059	18.724	53		1.40E-02		4.67E-07	3.40E-04
77.24	18.443	2.023	18.553	88		8.29E-03		2.43E-07	3.00E-04
115.86	18.217	1.986	18.330	68		1.05E-02		3.18E-07	3.10E-04
176.10	17.195	1.819	17.706	188		3.54E-03		3.11E-07	8.97E-04
264.16	15.506	1.542	16.351	3530		1.61E-04		1.60E-08	1.01E-03
528.31	13.633	1.235	14.570	1290		3.49E-04		1.28E-08	3.75E-04
1055.08	12.305	1.017	12.969	514		6.94E-04		9.06E-09	1.33E-04
528.31	12.449	1.041	12.377						1.45E-05
176.10	12.616	1.068	12.533						2.51E-05
77.24	12.776	1.094	12.696						8.57E-05
9.65	12.904	1.115	12.840						9.97E-05

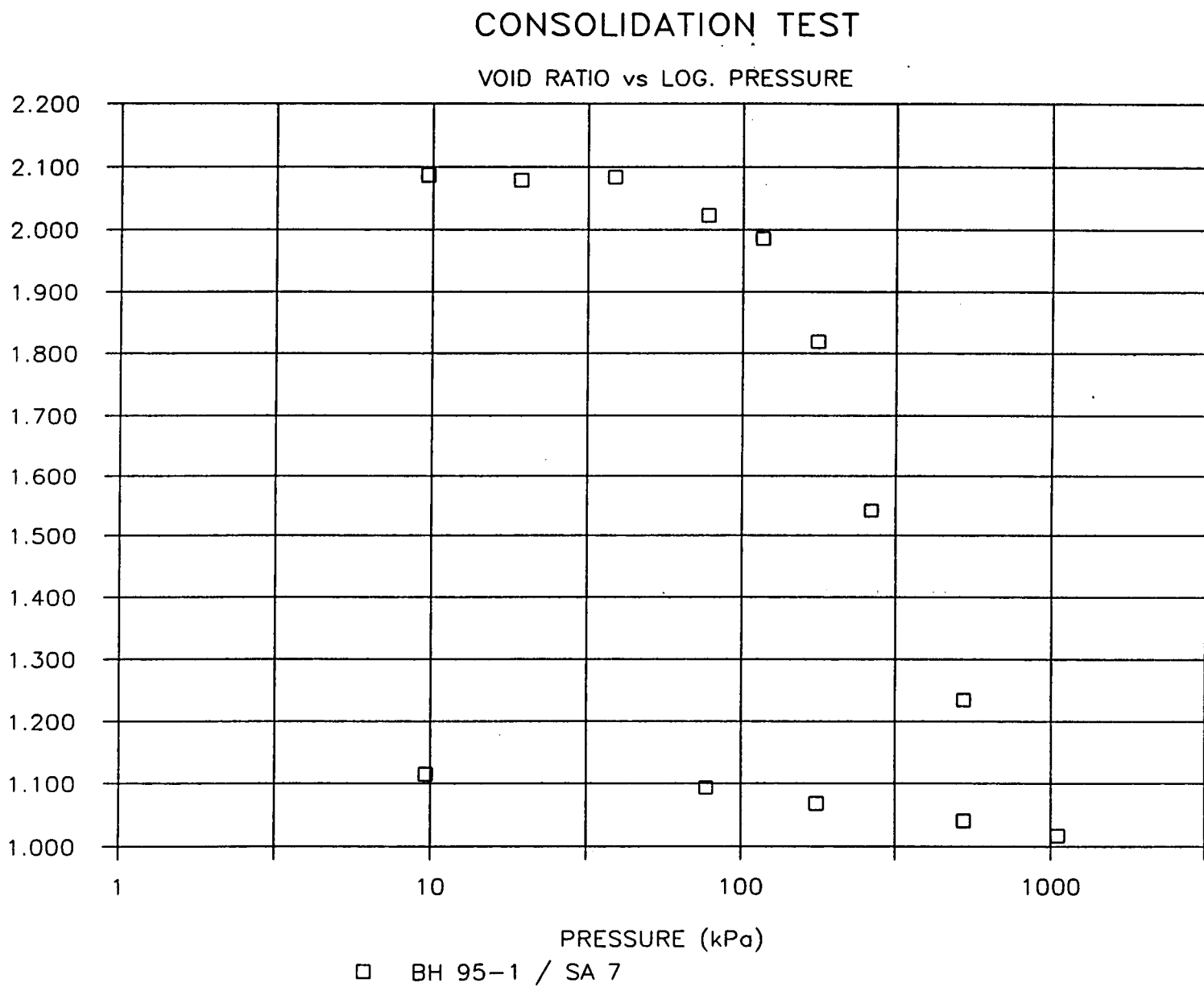
Notes:

k calculated using Cv based on t90 values.

Water Content % , initial	77.3	Liquid Limit %	n/a
Water Content % , final	43.2	Plastic Limit %	n/a
		Plastic Index %	n/a
Original Volume, cc	59.869	Liquidity Index	n/a
Volume of Solids, cc	19.31		
Volume of Voids, cc	40.55	Unit Weight, kN/m ³	15.10
Degree of Saturation %	99.42	Dry Unit Weight, kN/m ³	8.52

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 1-2



VOID RATIO

Golder Associates

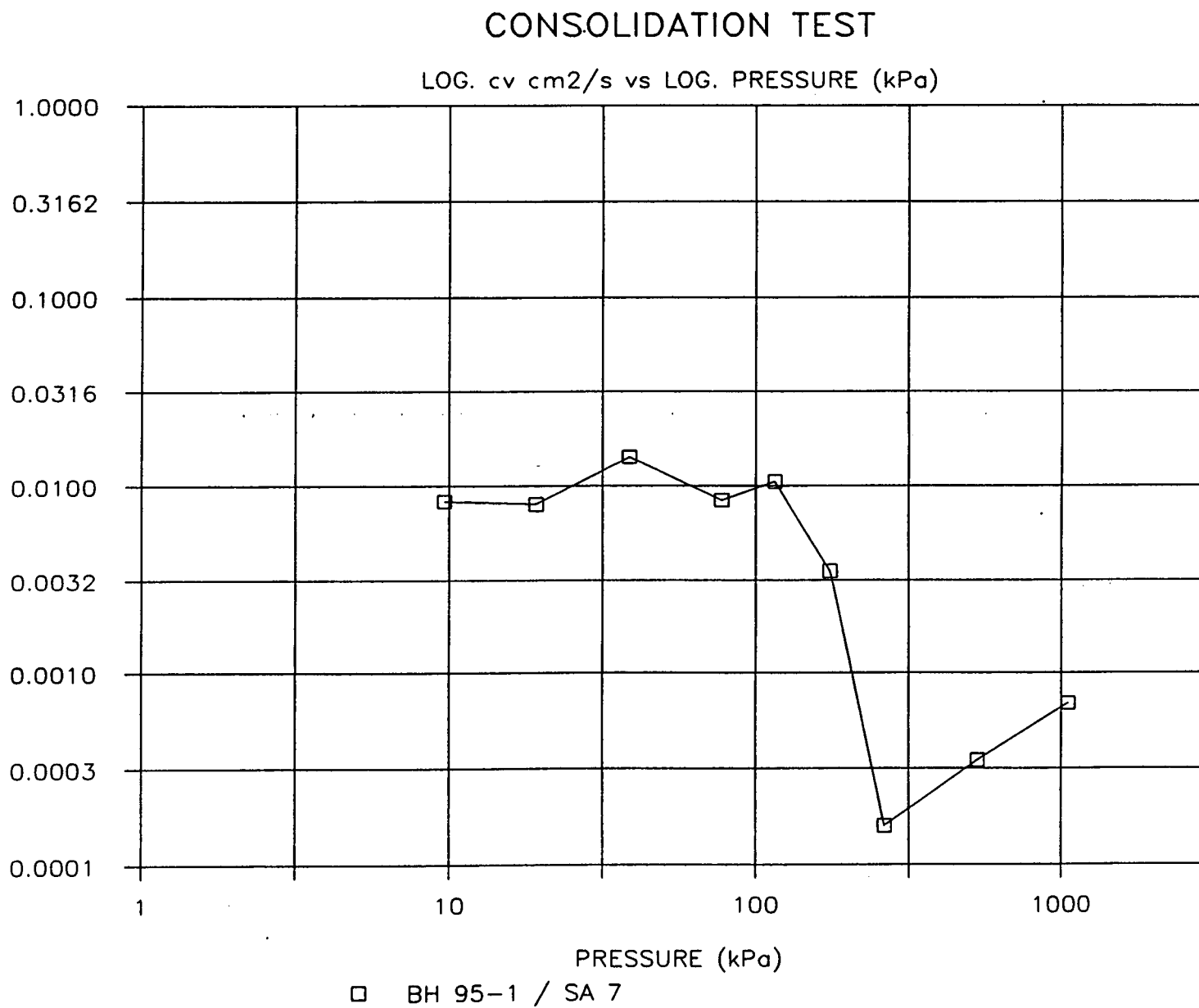


FIGURE 1-3

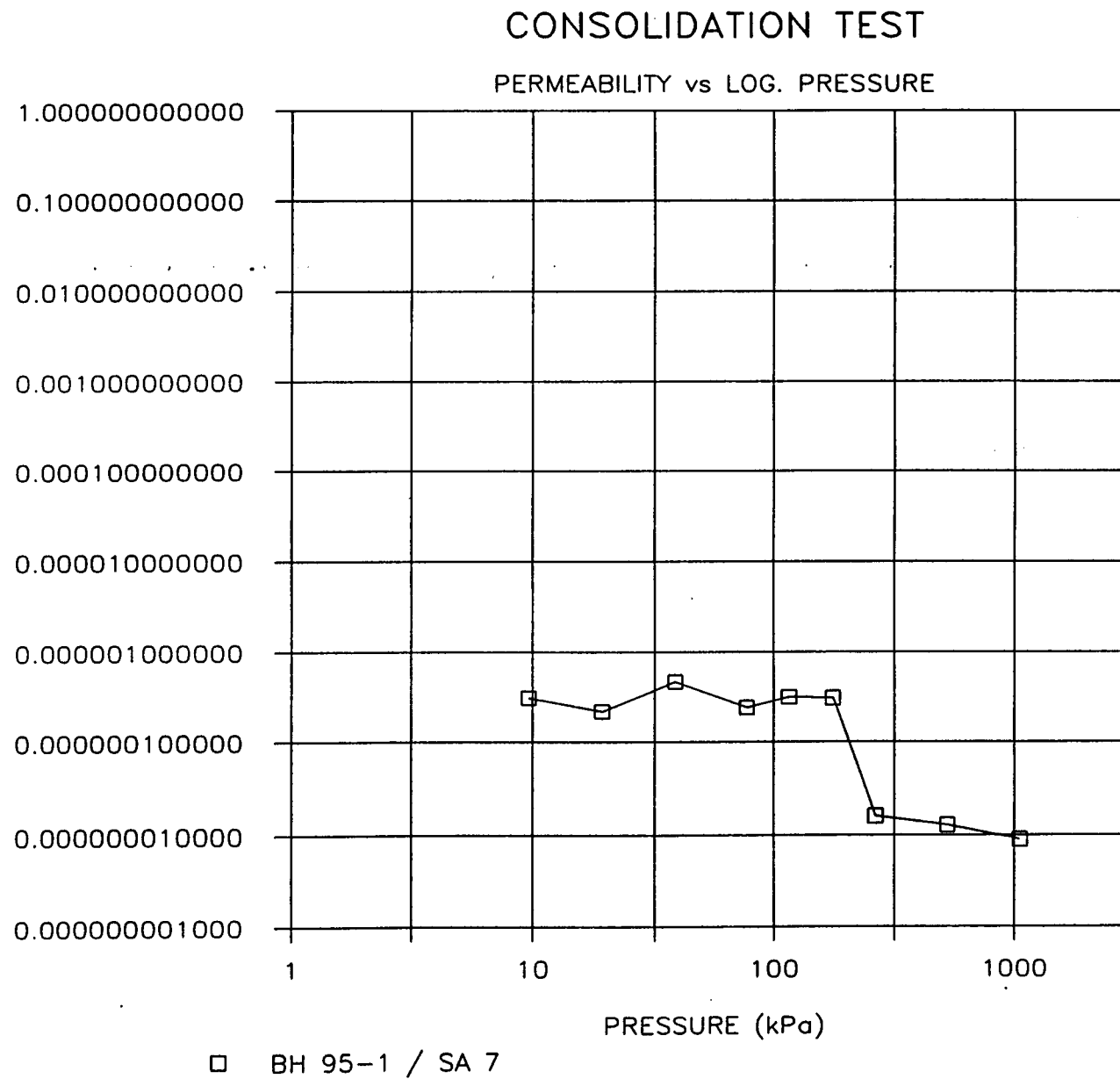


FIGURE 1-4

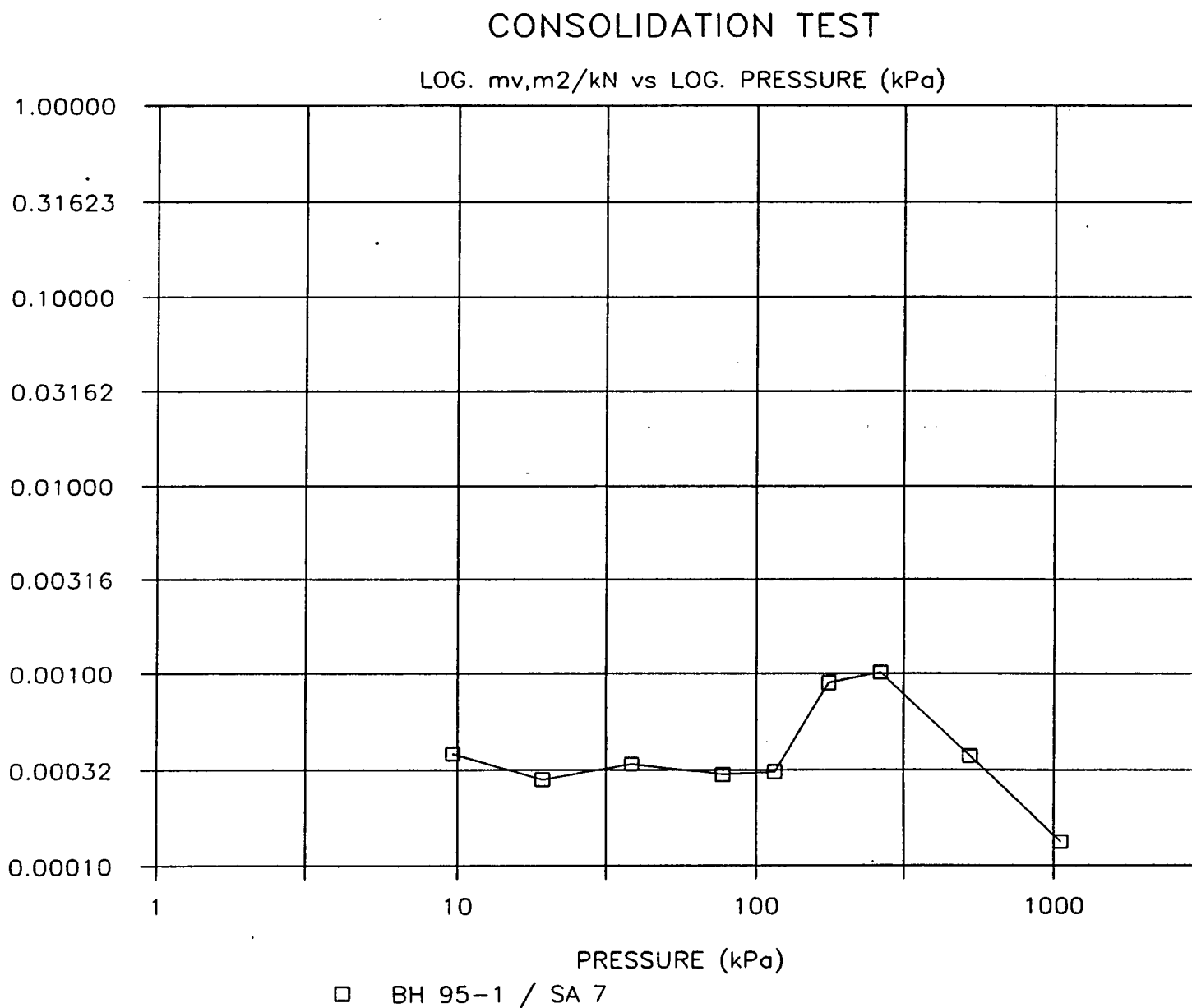


FIGURE 1-5

CONSOLIDATION SUMMARY

FIGURE 2-1

PROJECT 951-1323	SPECIFIC GRAVITY 2.70 assumed	DATE START 95-04-26
SAMPLE BH 95-1 / SA 9	DRY WEIGHT, gm 57.8	DATE COMPLETED 95-05-09
AREA(mm2) 3169.03	SOLIDS HT.2HS 6.755	

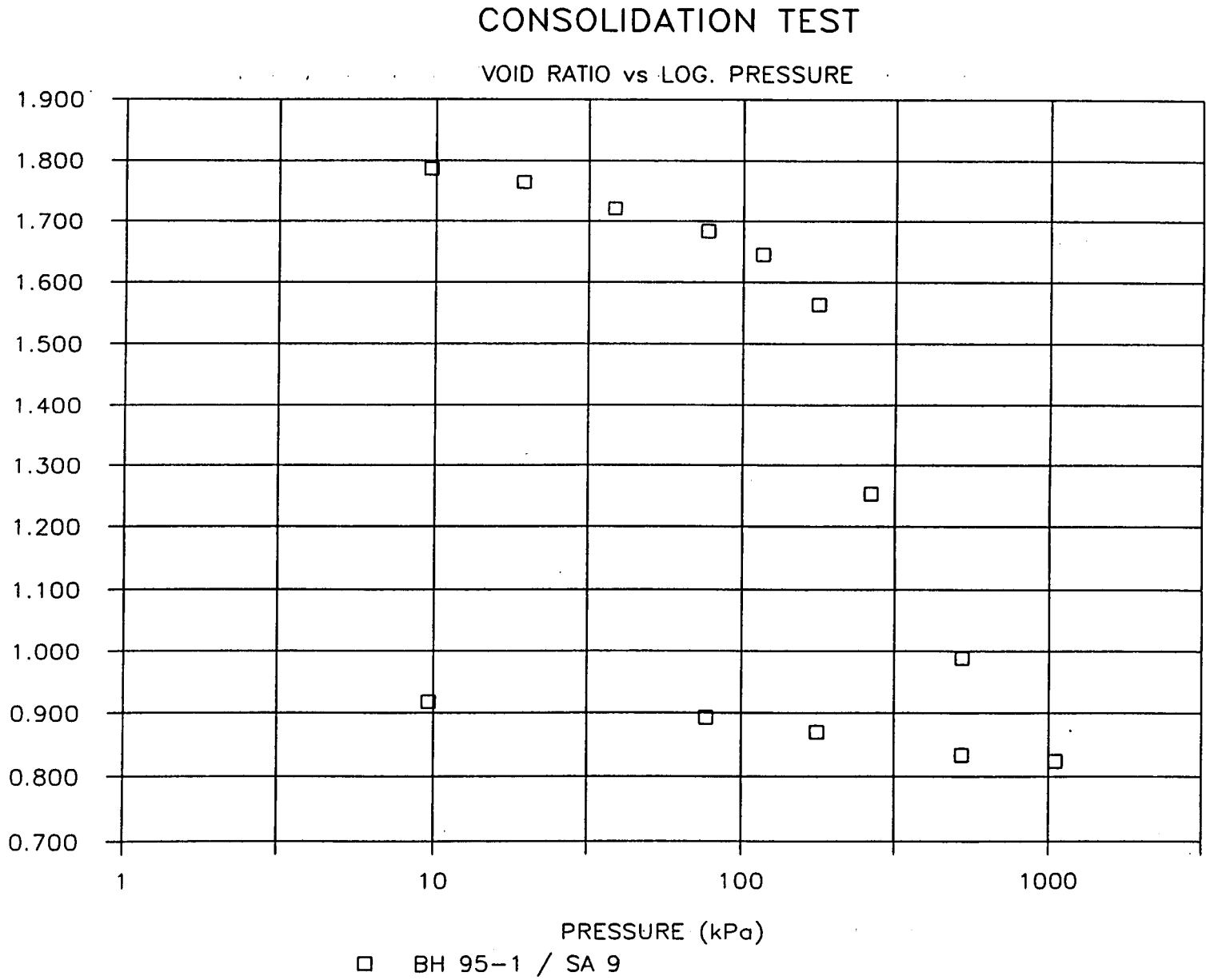
Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	t50 sec	cv. t90 cm2/s	t50	k cm/S	mv m2/kN
0.00	19.030	1.817	19.030						
9.65	18.827	1.787	18.928	126		6.03E-03		6.54E-07	1.11E-03
19.29	18.678	1.765	18.753	101		7.38E-03		5.85E-07	8.09E-04
38.58	18.382	1.721	18.530	92		7.91E-03		6.25E-07	8.06E-04
77.17	18.129	1.684	18.256	195		3.62E-03		1.22E-07	3.45E-04
115.75	17.868	1.645	17.999	206		3.33E-03		1.16E-07	3.56E-04
175.94	17.318	1.564	17.593	305		2.15E-03		1.01E-07	4.80E-04
263.91	15.217	1.253	16.268	5562		1.01E-04		1.24E-08	1.26E-03
527.81	13.433	0.989	14.325	1500		2.90E-04		1.01E-08	3.55E-04
1054.08	12.321	0.824	12.877	753		4.67E-04		5.08E-09	1.11E-04
527.81	12.388	0.834	12.355						6.69E-06
175.94	12.629	0.870	12.509						3.60E-05
77.17	12.789	0.893	12.709						8.53E-05
9.65	12.953	0.917	12.871						1.27E-04

Notes:

k calculated using Cv based on t90 values.

Water Content % , initial	65.6	Liquid Limit %	n/a
Water Content % , final	43.5	Plastic Limit %	n/a
		Plastic Index %	n/a
Original Volume, cc	60.312	Liquidity Index	n/a
Volume of Solids, cc	21.41		
Volume of Voids, cc	38.90	Unit Weight, kN/m3	15.60
Degree of Saturation %	97.47	Dry Unit Weight, kN/m3	9.42

VOID RATIO



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 2-2

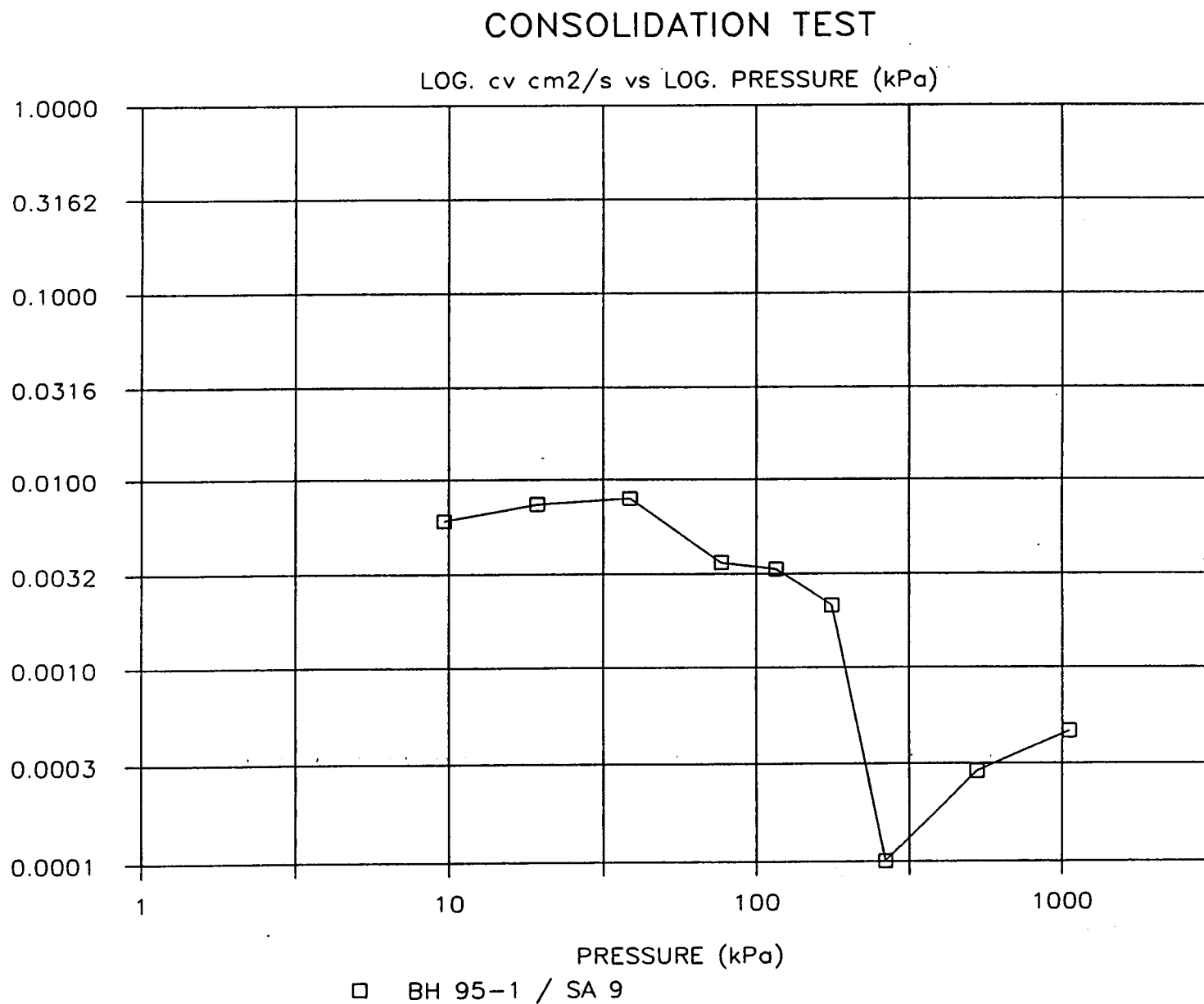


FIGURE 2-3

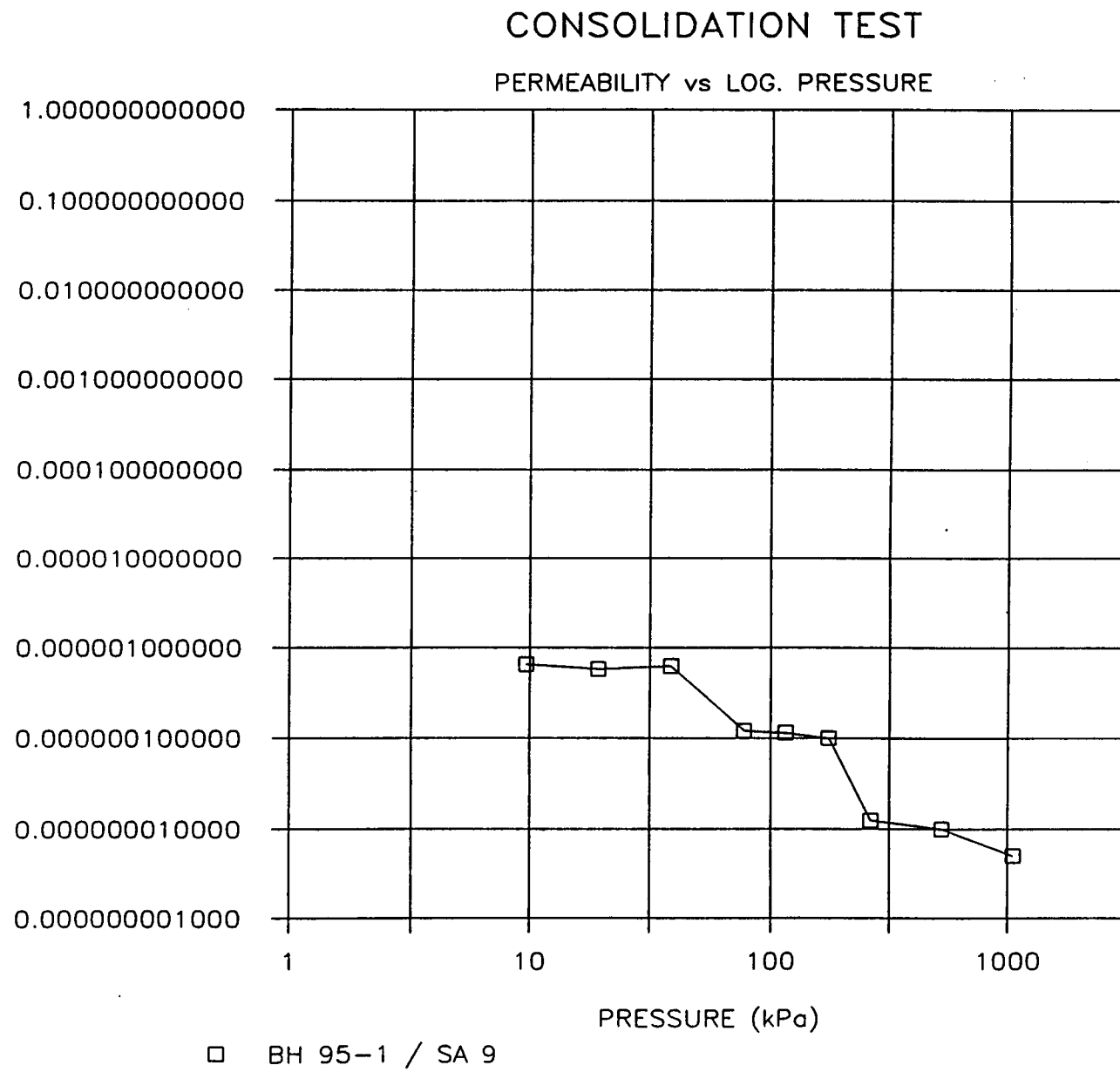


FIGURE 2-4

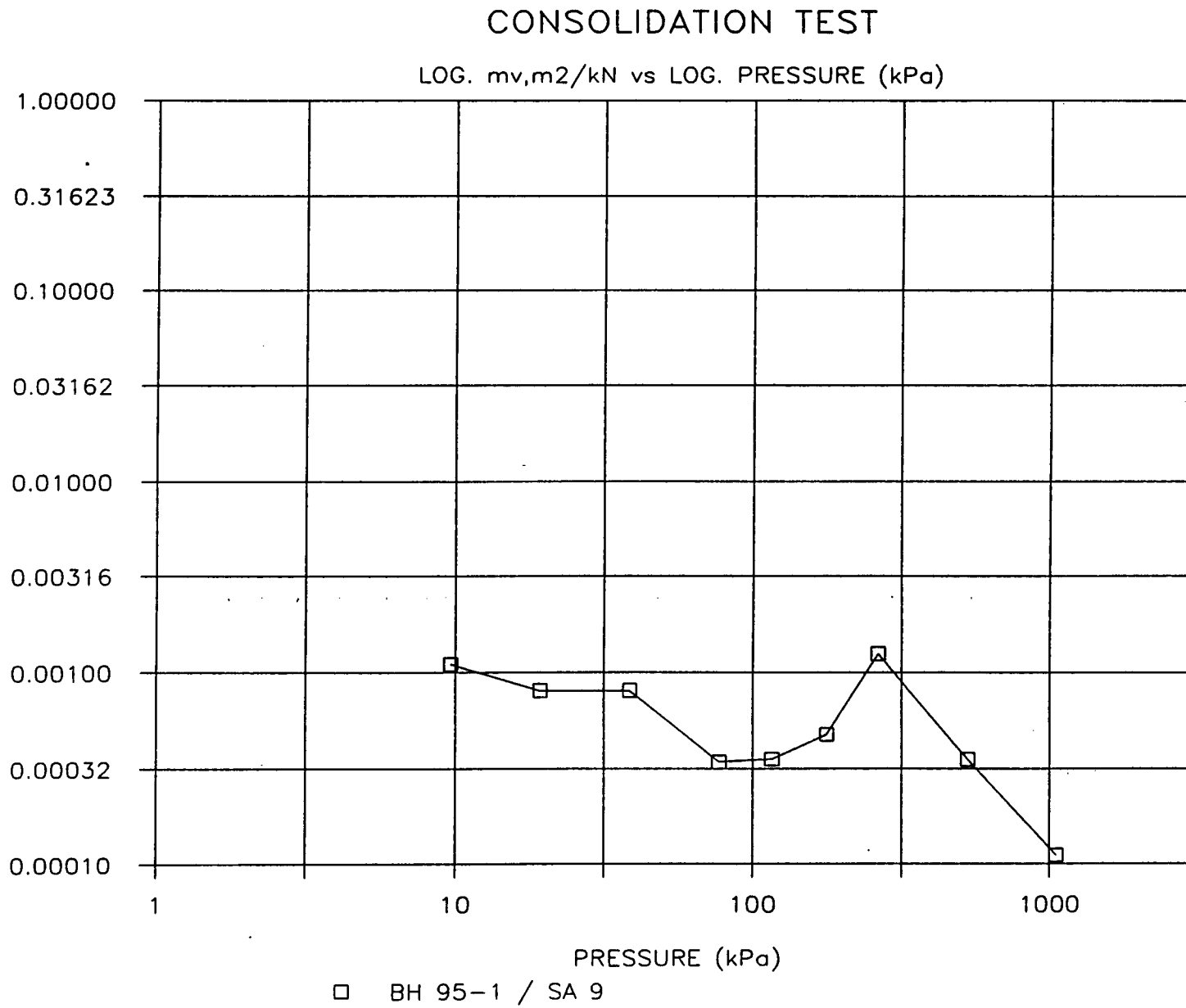


FIGURE 2-5

CONSOLIDATION SUMMARY

FIGURE 3-1

PROJECT 951-1323 SPECIFIC GRAVITY 2.70 assumed DATE START 95-04-12
SAMPLE BH 95-7 / SA 3 DRY WEIGHT,gm 71.54 DATE COMPLETED 95-04-25
AREA(mm2) 3157.06 SOLIDS HT.2HS 8.393

Load	Corr. Height	Void Ratio	Average Height	t90	t50	cv.	t50	k	mv
kPa	mm		mm	sec	sec	cm2/s		cm/S	m2/kN
0.00	19.100	1.276	19.100						
9.68	19.108	1.277	19.104	swelling					
19.36	19.094	1.275	19.101	37		2.09E-02		1.51E-07	7.36E-05
38.73	19.064	1.271	19.079	72		1.07E-02		8.52E-08	8.11E-05
77.46	19.004	1.264	19.034	49		1.57E-02		1.24E-07	8.08E-05
154.92	18.890	1.251	18.947	72		1.06E-02		8.01E-08	7.73E-05
309.83	18.691	1.227	18.790	45		1.66E-02		1.10E-07	6.72E-05
619.66	18.228	1.172	18.460	44		1.64E-02		1.26E-07	7.82E-05
1254.82	16.752	0.996	17.490	469		1.38E-03		1.65E-08	1.22E-04
2509.63	15.308	0.824	16.030	1016		5.36E-04		3.17E-09	6.02E-05
1254.82	15.384	0.833	15.346						3.17E-06
309.83	15.636	0.863	15.510						1.40E-05
77.46	15.918	0.897	15.777						6.36E-05
9.68	16.109	0.919	16.013						1.47E-04

Notes:

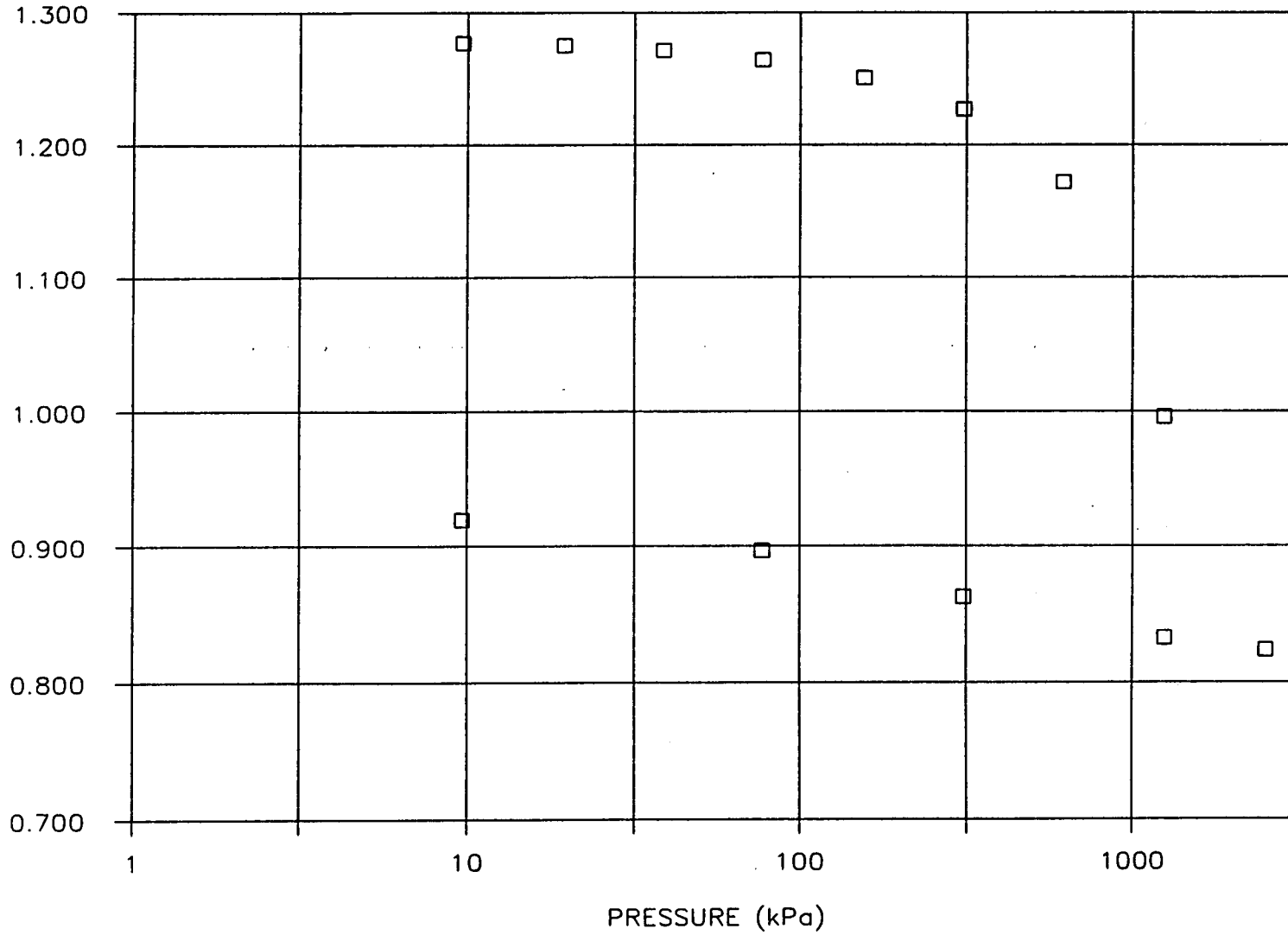
k calculated using Cv based on t90 values.

Water Content % ,initial	47.5	Liquid Limit %	n/a
Water Content % , final	36.0	Plastic Limit %	n/a
		Plastic Index %	n/a
Original Volume,cc	60.299	Liquidity Index	n/a
Volume of Solids,cc	26.50		
Volume of Voids,cc	33.80	Unit Weight,kN/m3	17.20
Degree of Saturation %	100.53	Dry Unit Weight,kN/m3	11.66

Golder Associates

CONSOLIDATION TEST

VOID RATIO vs LOG. PRESSURE



□ BH 95-7 / SA 3

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 3-2

VOID RATIO

Golder Associates

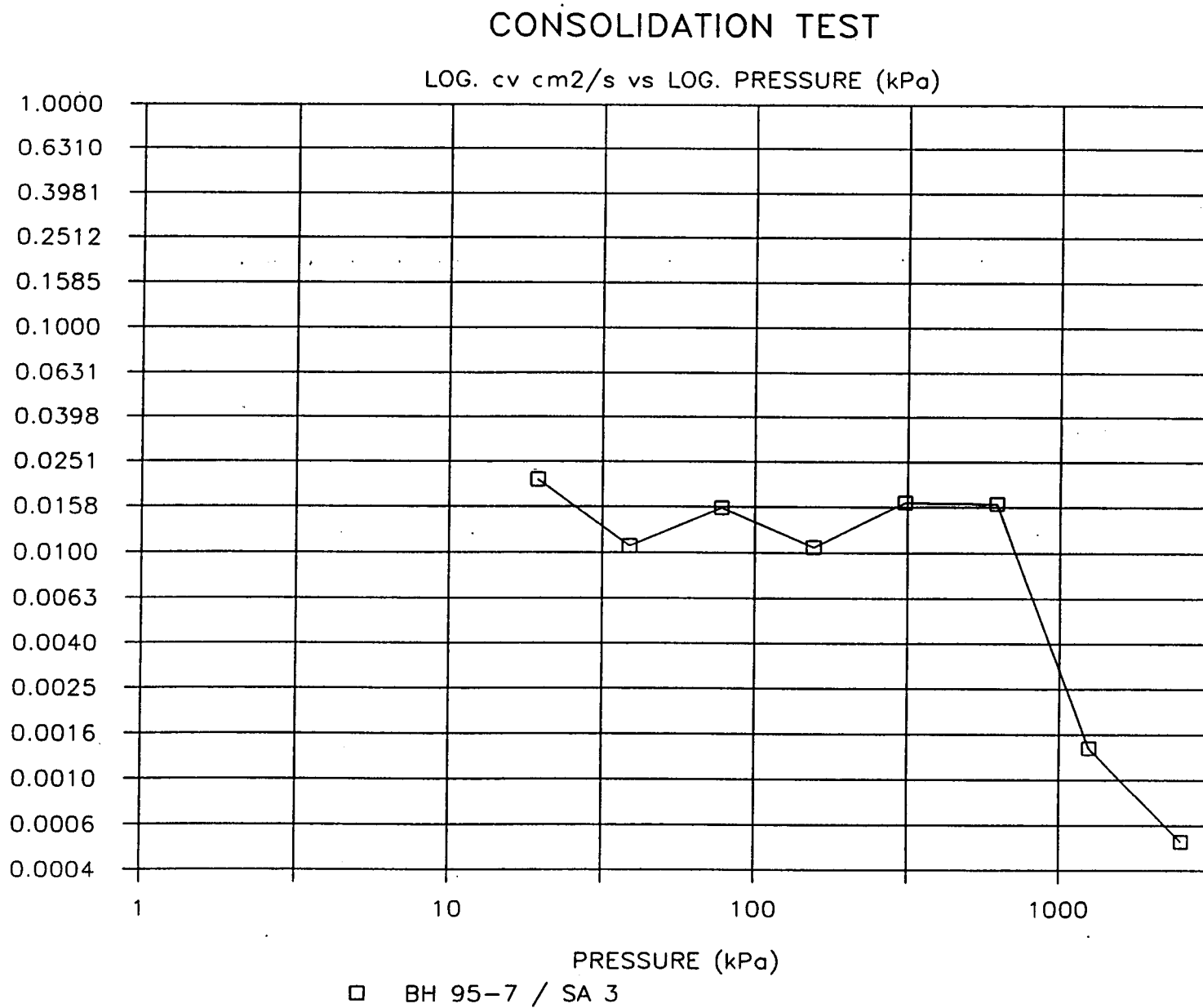


FIGURE 3-3

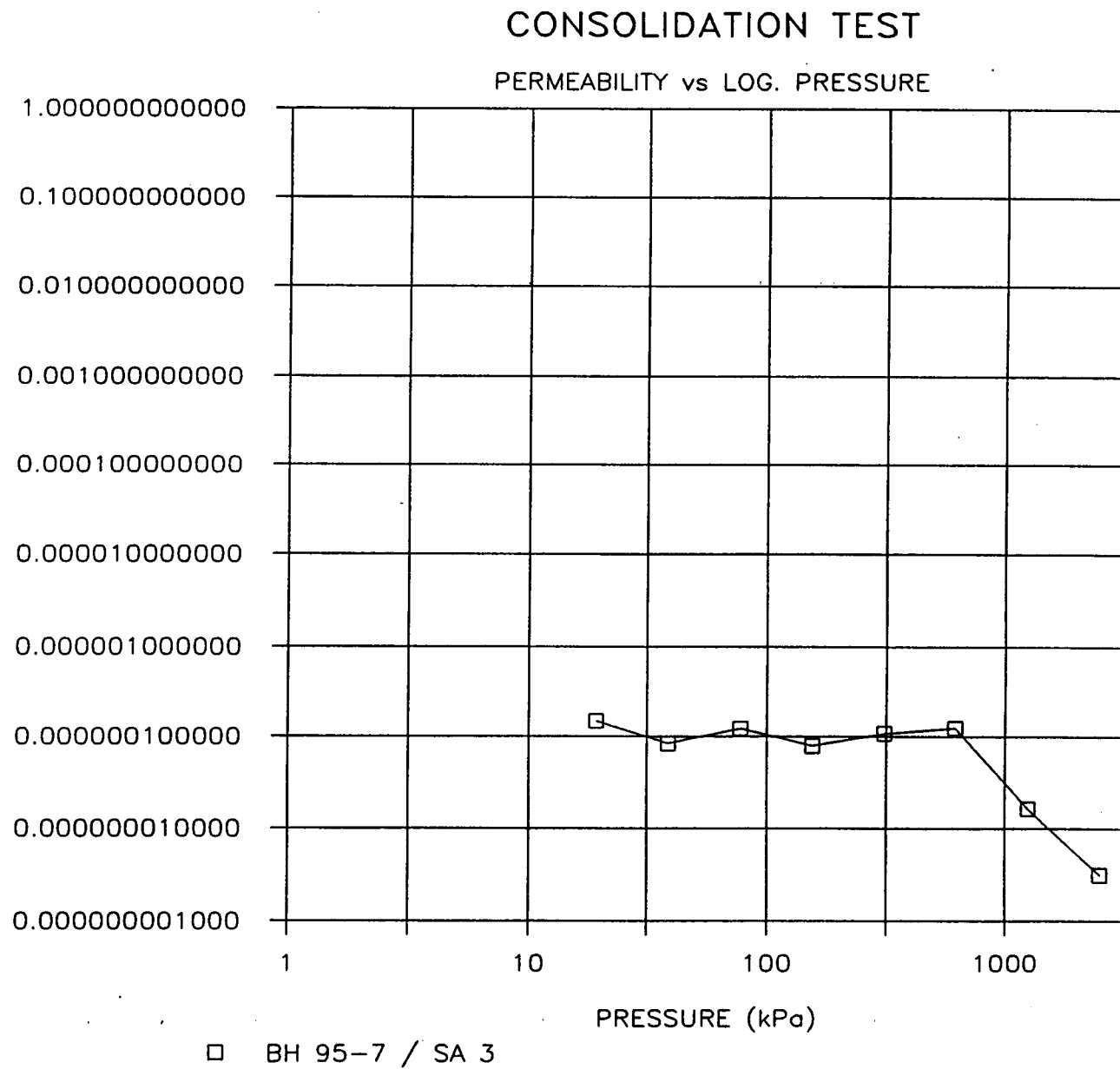


FIGURE 3-4

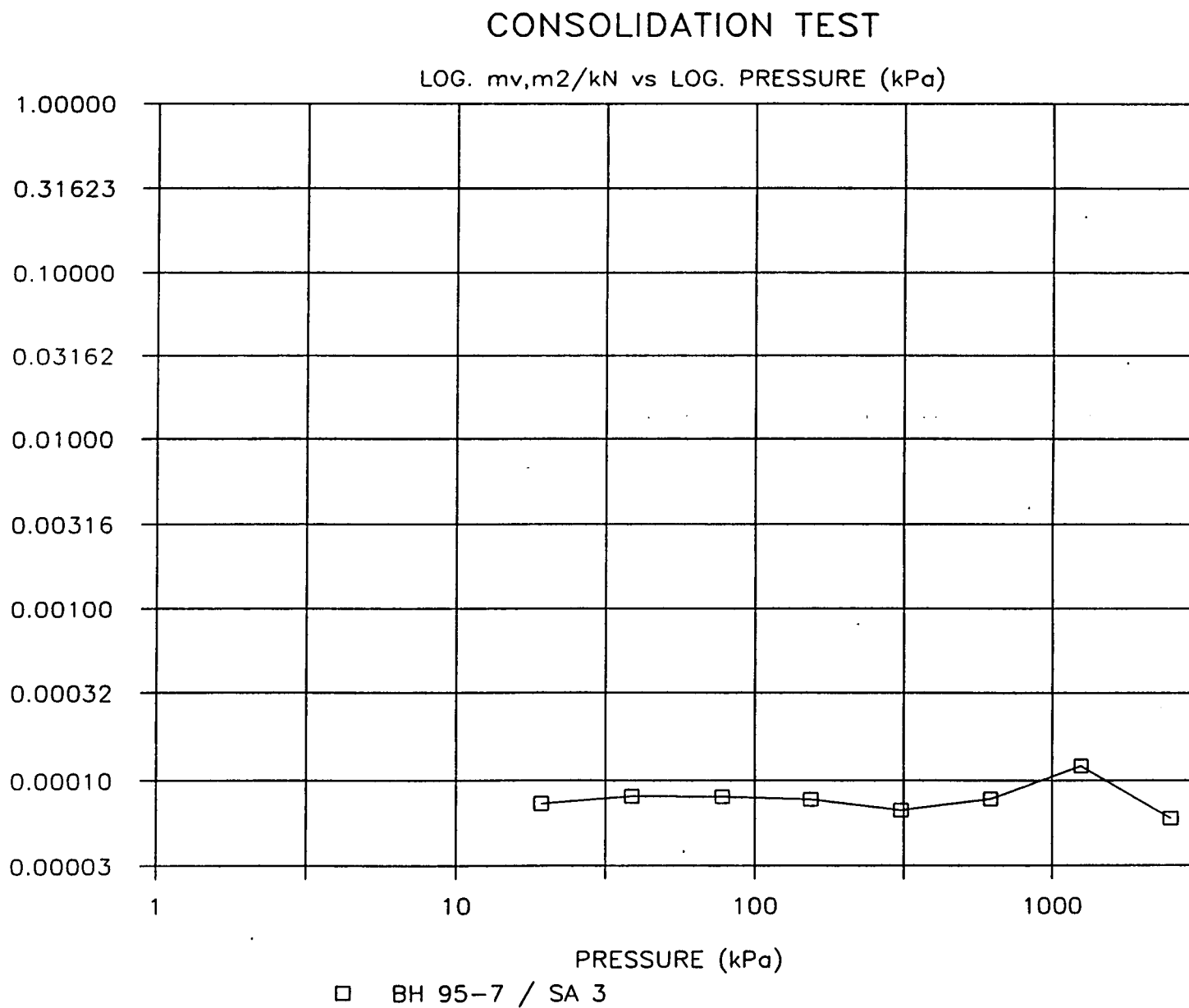


FIGURE 3-5

CONSOLIDATION SUMMARY

FIGURE 4-1

PROJECT 951-1323	SPECIFIC GRAVITY 2.70 assumed	DATE START 95-04-12
SAMPLE BH 95-7 / SA 6	DRY WEIGHT, gm 53.74	DATE COMPLETED 95-04-26
AREA(mm2) 3157.06	SOLIDS HT.2HS 6.305	

Load	Corr. Height	Void Ratio	Average Height	t90	t50	cv.	t50	k	mv
kPa	mm		mm	sec	sec	cm2/s		cm/s	m2/kN
0.00	19.100	2.030	19.100						
9.68	19.058	2.023	19.079	30		2.57E-02		5.73E-07	2.27E-04
19.36	19.021	2.017	19.040	75		1.02E-02		2.01E-07	2.00E-04
38.73	18.965	2.008	18.993	90		8.50E-03		1.26E-07	1.51E-04
77.46	18.850	1.990	18.908	43		1.76E-02		2.69E-07	1.56E-04
116.19	18.725	1.970	18.787	68		1.10E-02		1.83E-07	1.69E-04
176.60	18.463	1.929	18.594	60		1.22E-02		2.71E-07	2.27E-04
264.91	16.630	1.638	17.547	156		4.18E-03		4.46E-07	1.09E-03
529.81	13.912	1.207	15.271	1441		3.43E-04		1.81E-08	5.37E-04
1058.07	12.580	0.995	13.246	531		7.01E-04		9.06E-09	1.32E-04
529.81	12.636	1.004	12.608						5.55E-06
176.60	12.812	1.032	12.724						2.61E-05
77.46	13.017	1.065	12.914						1.08E-04
9.68	13.278	1.106	13.147						2.02E-04

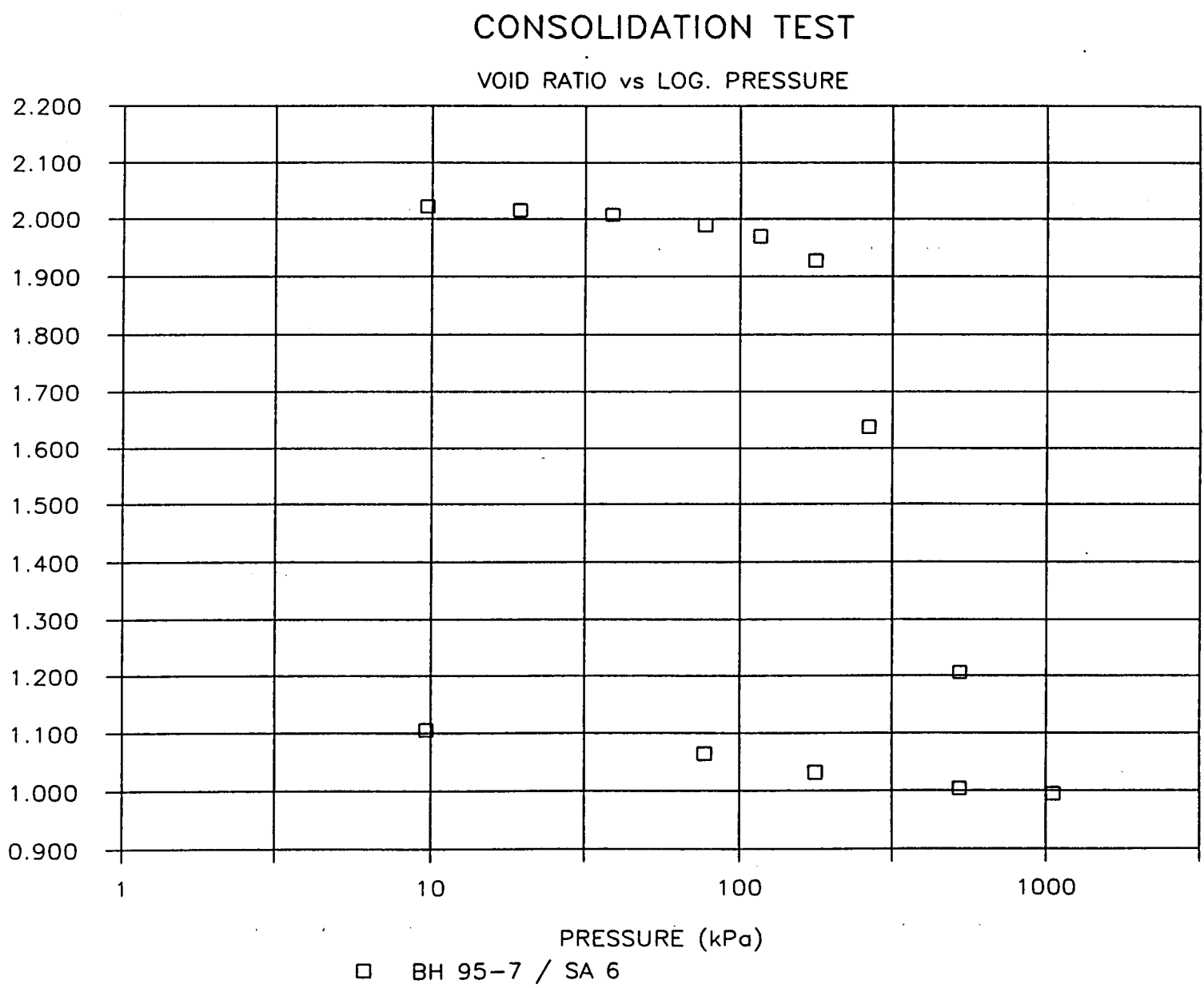
Notes:

k calculated using Cv based on t90 values.

Water Content % , initial	75.0	Liquid Limit %	n/a
Water Content % , final	45.2	Plastic Limit %	n/a
		Plastic Index %	n/a
Original Volume, cc	60.299	Liquidity Index	n/a
Volume of Solids, cc	19.90		
Volume of Voids, cc	40.40	Unit Weight, kN/m3	15.30
Degree of Saturation %	99.77	Dry Unit Weight, kN/m3	8.74

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 4-2



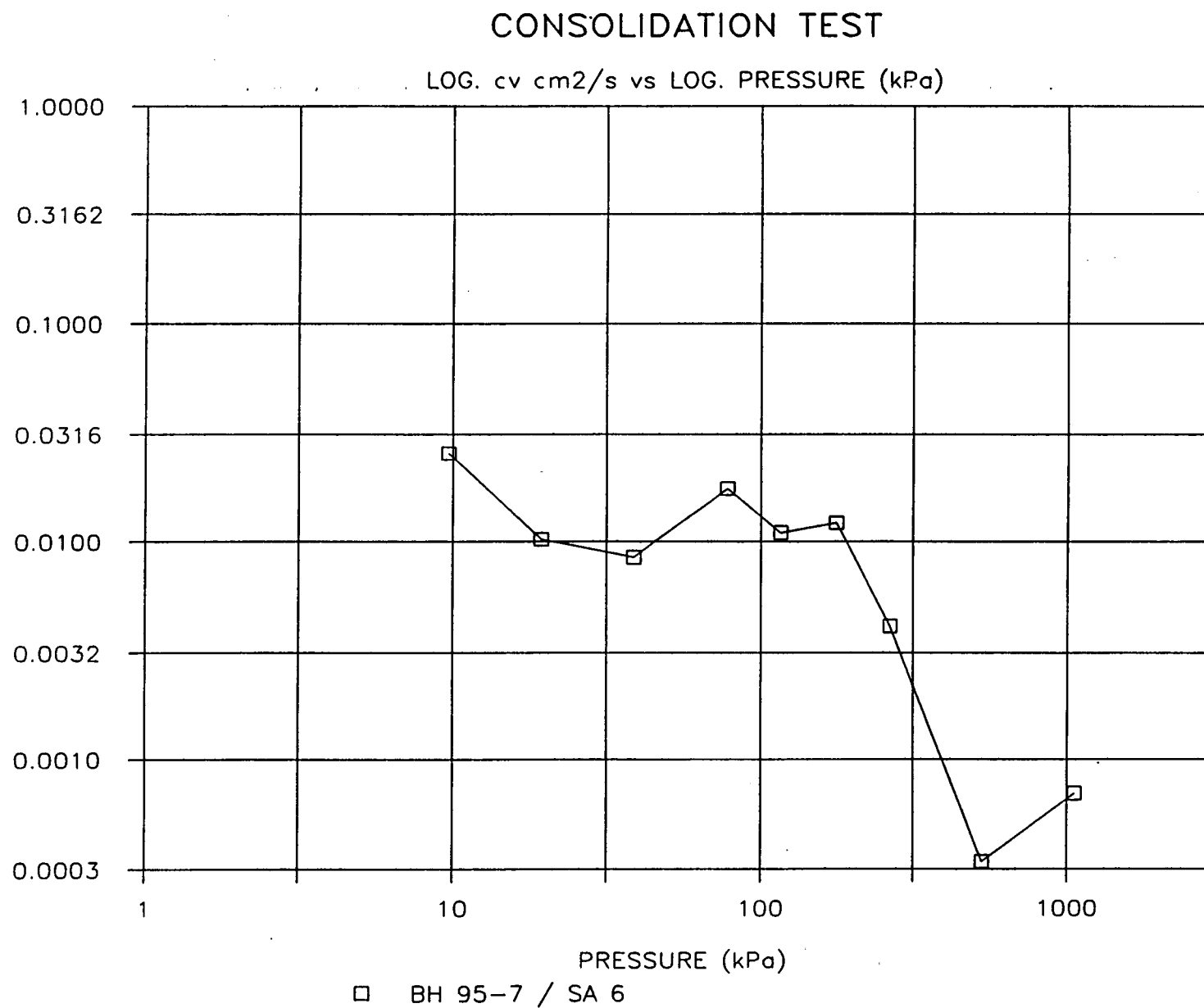


FIGURE 4-3

PERMEABILITY, cm/s

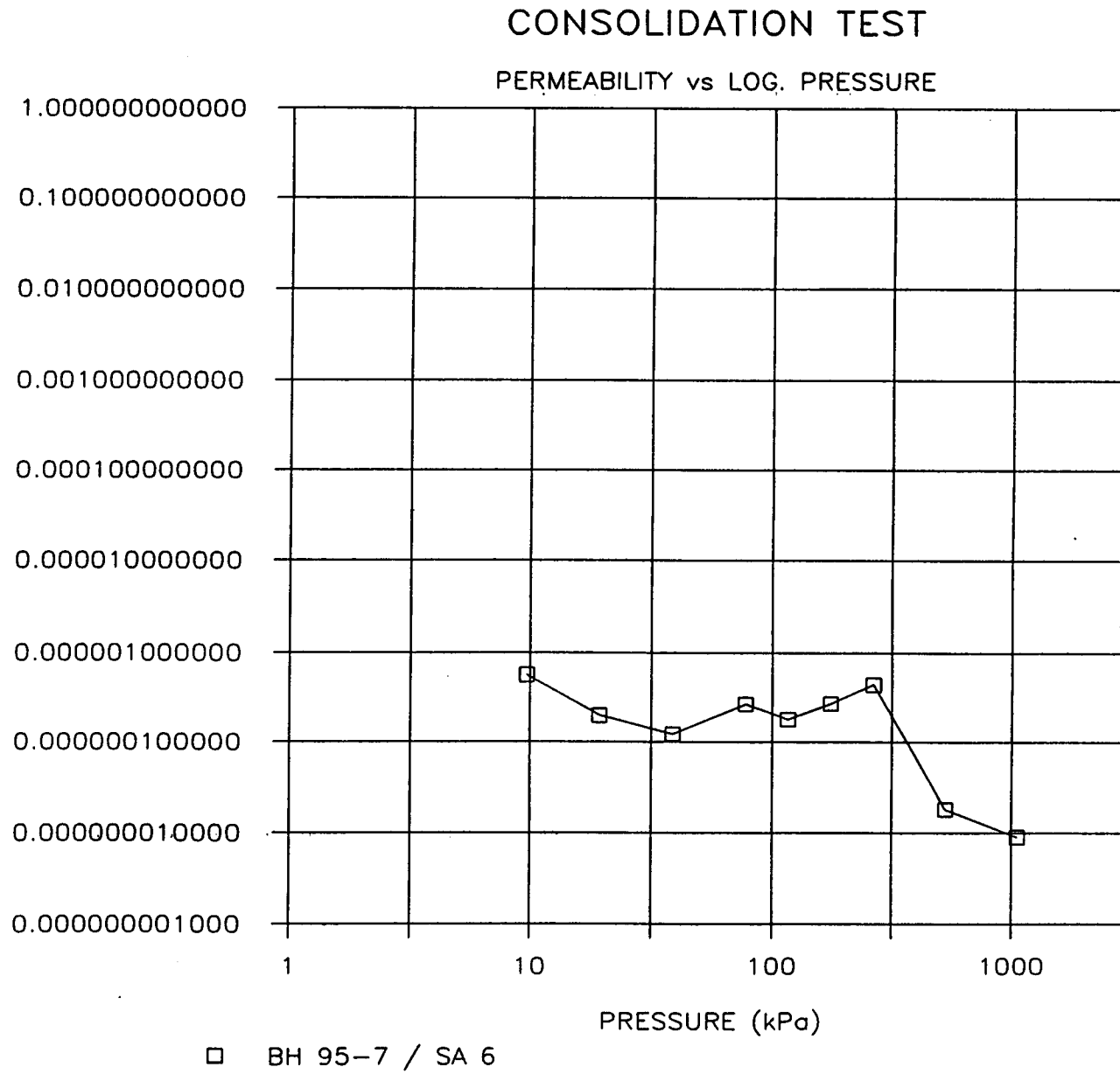


FIGURE 4-4

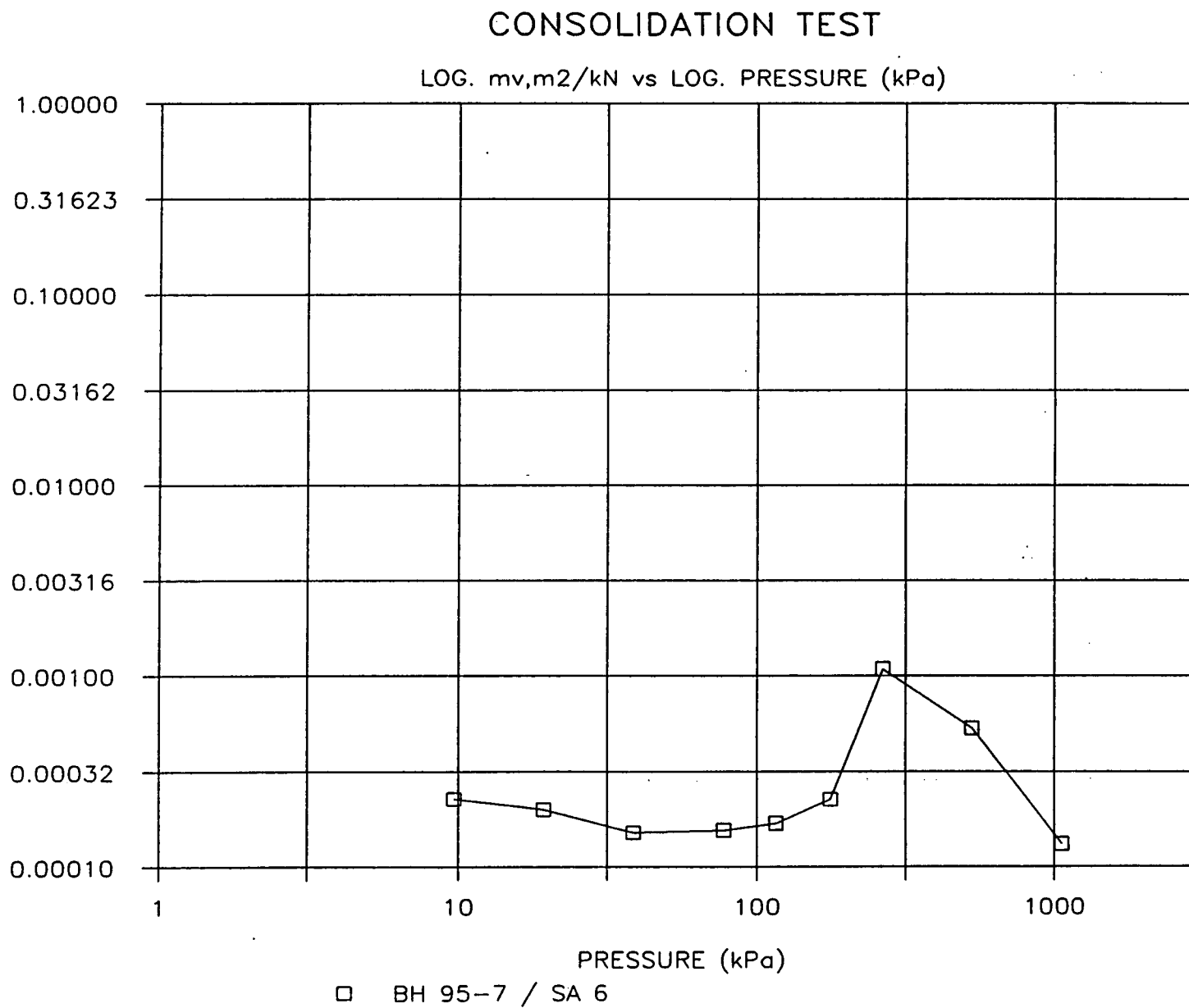


FIGURE 4-5

CONSOLIDATION SUMMARY

FIGURE 5-1

PROJECT	951-1323	SPECIFIC GRAVITY	2.70 assumed	DATE START	95-04-12
SAMPLE	BH 95-8 / SA 4	DRY WEIGHT, gm	57.55	DATE COMPLETED	95-04-26
AREA(mm2)	3157.06	SOLIDS HT.2HS	6.751		

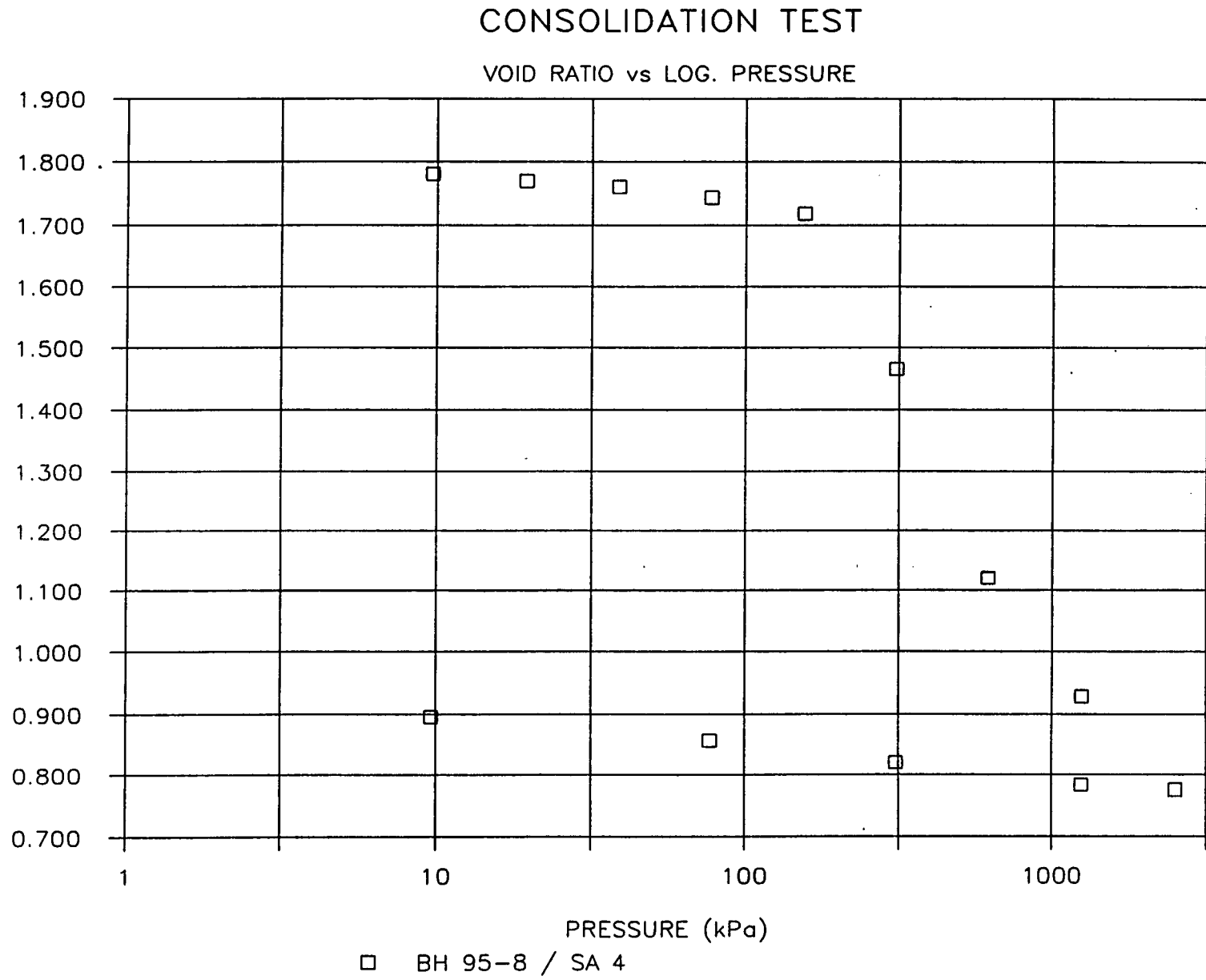
Load	Corr. Height	Void Ratio	Average Height	t90	t50	cv.	t90	t50	k	mv
kPa	mm		mm	sec	sec	cm2/s			cm/S	m2/kN
0.00	19.100	1.829	19.100							
9.68	18.779	1.781	18.939	41		1.85E-02			3.16E-06	1.74E-03
19.36	18.703	1.770	18.741	25		2.98E-02			1.19E-06	4.08E-04
38.73	18.638	1.761	18.671	30		2.46E-02			4.24E-07	1.76E-04
77.46	18.527	1.744	18.583	65		1.13E-02			1.66E-07	1.50E-04
154.92	18.355	1.719	18.441	78		9.24E-03			1.06E-07	1.17E-04
309.83	16.649	1.466	17.502	444		1.46E-03			8.27E-08	5.77E-04
619.66	14.314	1.120	15.481	1500		3.39E-04			1.31E-08	3.95E-04
1254.82	13.019	0.928	13.666	886		4.47E-04			4.67E-09	1.07E-04
2509.63	11.988	0.776	12.504	604		5.49E-04			2.31E-09	4.30E-05
1254.82	12.044	0.784	12.016							2.35E-06
309.83	12.291	0.820	12.167							1.37E-05
77.46	12.539	0.857	12.415							5.61E-05
9.68	12.800	0.896	12.670							2.01E-04

Notes:

k calculated using Cv based on t90 values.

Water Content % , initial	68.9	Liquid Limit %	n/a
Water Content % , final	38.0	Plastic Limit %	n/a
		Plastic Index %	n/a
Original Volume, cc	60.299	Liquidity Index	n/a
Volume of Solids, cc	21.31		
Volume of Voids, cc	38.98	Unit Weight, kN/m3	15.70
Degree of Saturation %	101.71	Dry Unit Weight, kN/m3	9.30

VOID RATIO



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 5-2

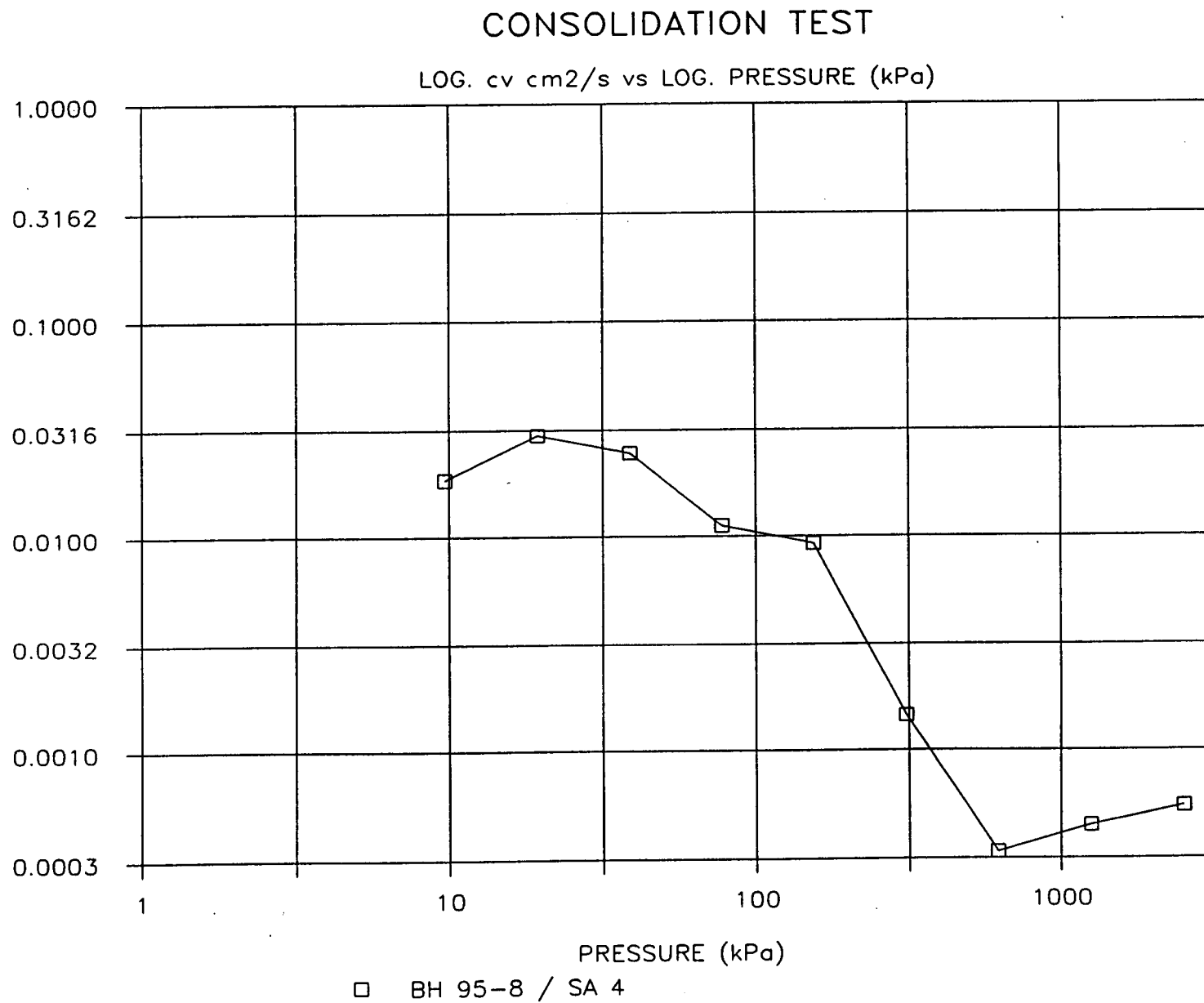


FIGURE 5-3

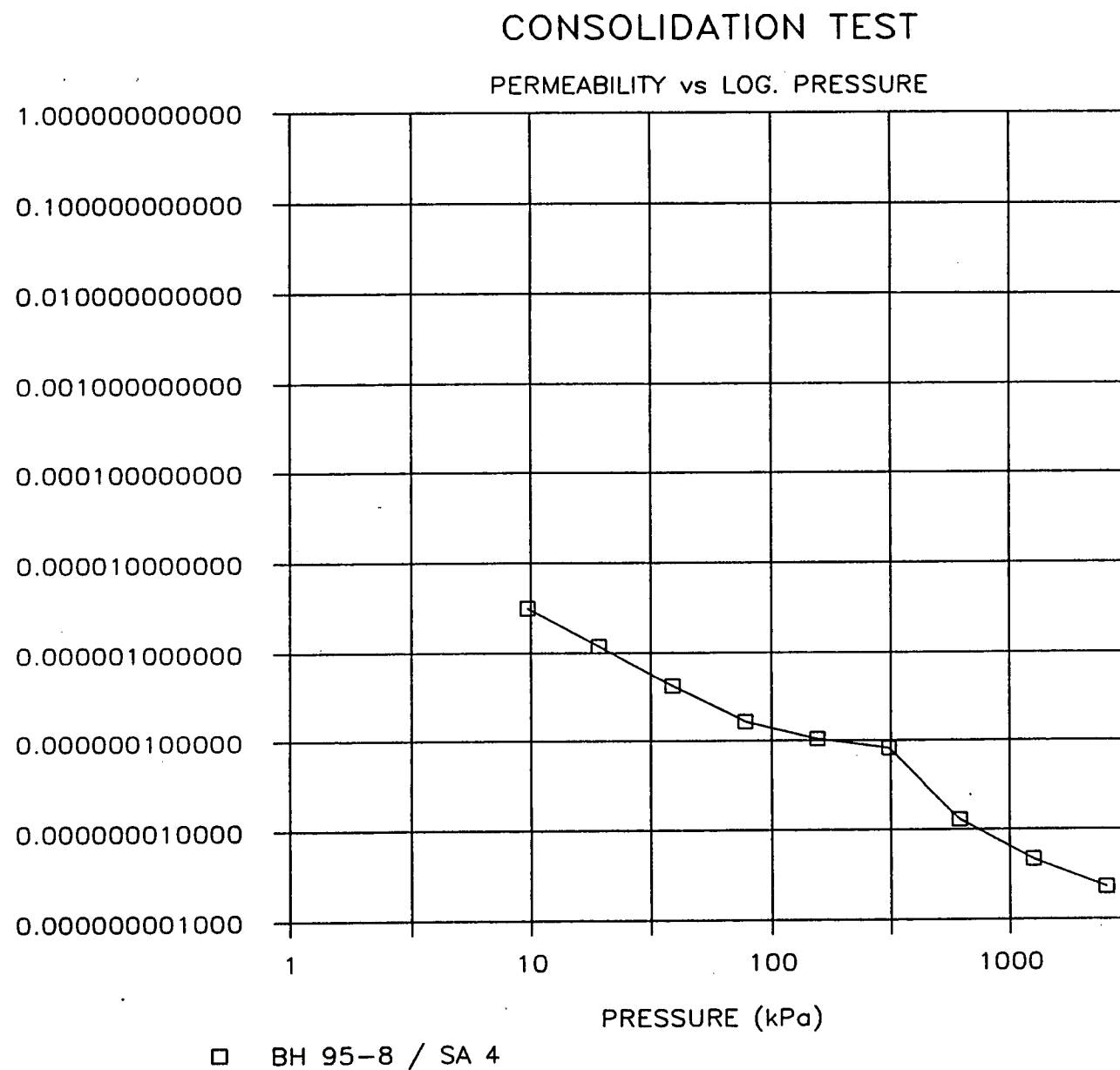


FIGURE 5-4

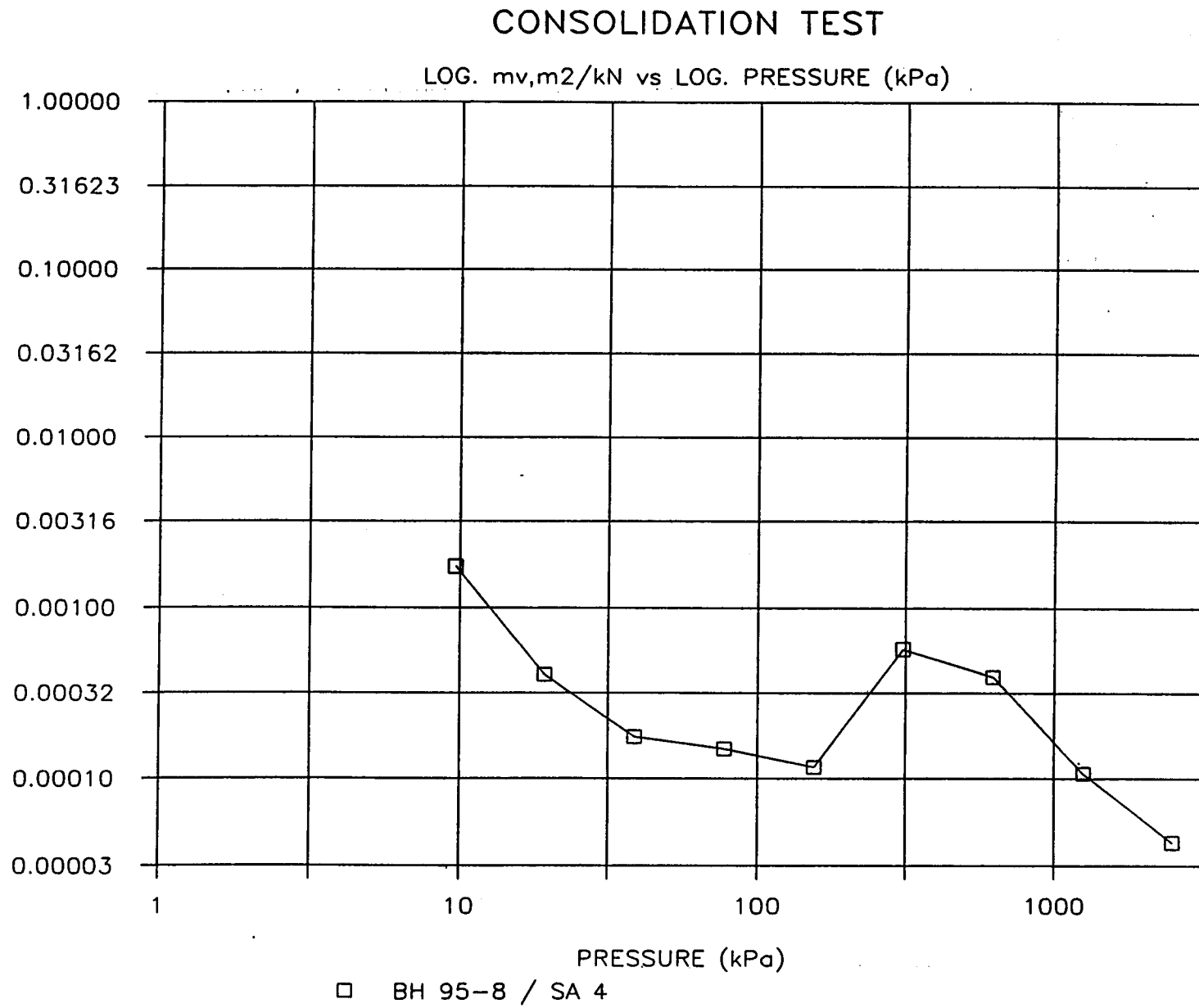


FIGURE 5-5

CONSOLIDATION SUMMARY

FIGURE 6-1

PROJECT 951-1323 SPECIFIC GRAVITY 2.70 assumed DATE START 95-04-12
 SAMPLE BH 95-14 / SA 9 DRY WEIGHT, gm 65.5 DATE COMPLETED 95-04-26
 AREA(mm²) 3157.06 SOLIDS HT.2HS 7.684

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	t50 sec	cv. t90 cm ² /s	t50	k cm/s	mv m ² /kN
0.00	19.100	1.486	19.100						
9.68	19.013	1.474	19.056	142		5.42E-03		2.51E-07	4.72E-04
19.36	18.906	1.460	18.960	189		4.03E-03		2.28E-07	5.77E-04
38.73	18.713	1.435	18.809	93		8.06E-03		4.14E-07	5.23E-04
77.46	18.312	1.383	18.512	139		5.23E-03		2.78E-07	5.42E-04
116.19	17.993	1.342	18.152	103		6.78E-03		2.86E-07	4.31E-04
176.60	17.464	1.273	17.728	221		3.01E-03		1.36E-07	4.59E-04
264.91	16.437	1.139	16.951	156		3.90E-03		2.33E-07	6.09E-04
529.81	15.107	0.966	15.772	362		1.46E-03		3.75E-08	2.63E-04
1058.07	14.079	0.832	14.593	212		2.13E-03		2.13E-08	1.02E-04
529.81	14.092	0.834	14.085						1.31E-06
176.60	14.198	0.848	14.145						1.57E-05
77.46	14.295	0.860	14.246						5.12E-05
9.68	14.508	0.888	14.401						1.65E-04

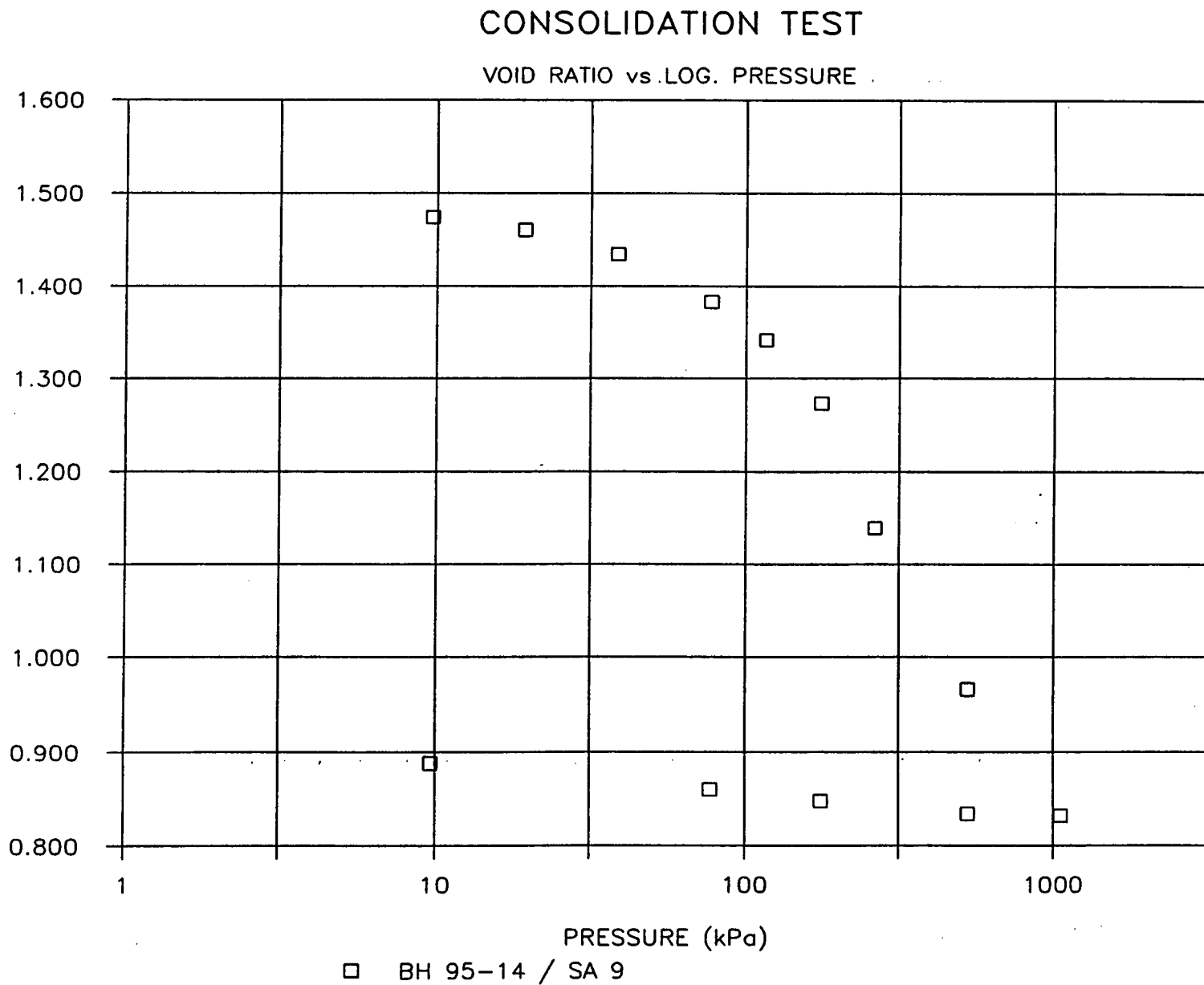
Notes:

k calculated using Cv based on t90 values.

Water Content % , initial	55.0	Liquid Limit %	n/a
Water Content % , final	34.9	Plastic Limit %	n/a
Original Volume, cc	60.299	Plastic Index %	n/a
Volume of Solids, cc	24.26	Liquidity Index	n/a
Volume of Voids, cc	36.04	Unit Weight, kN/m ³	16.40
Degree of Saturation %	99.96	Dry Unit Weight, kN/m ³	10.58

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 6-2



VOID RATIO

Golder Associates

COEFFICIENT OF CONSOLIDATION, cm^2/s

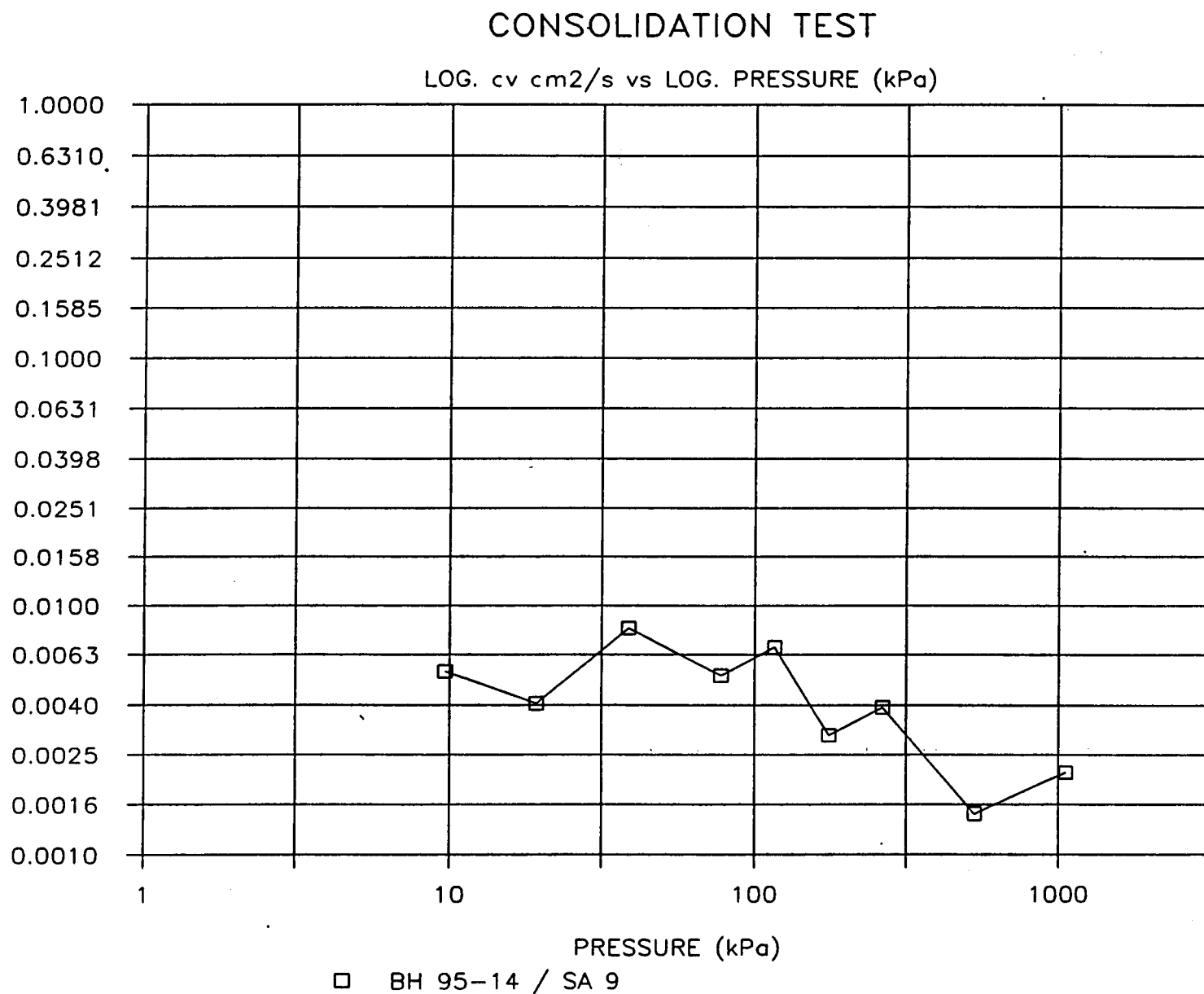


FIGURE 6-3

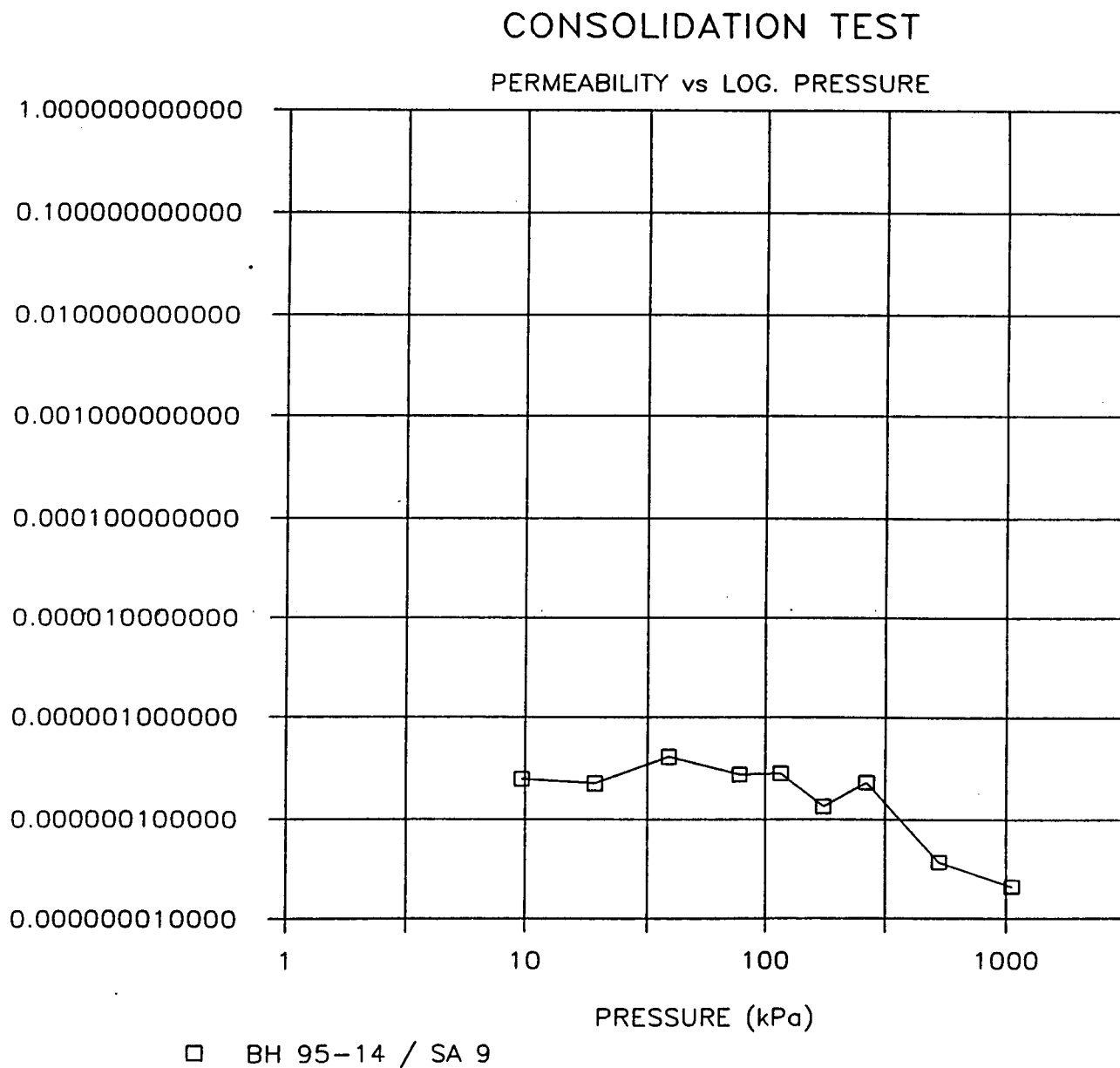


FIGURE 6-4

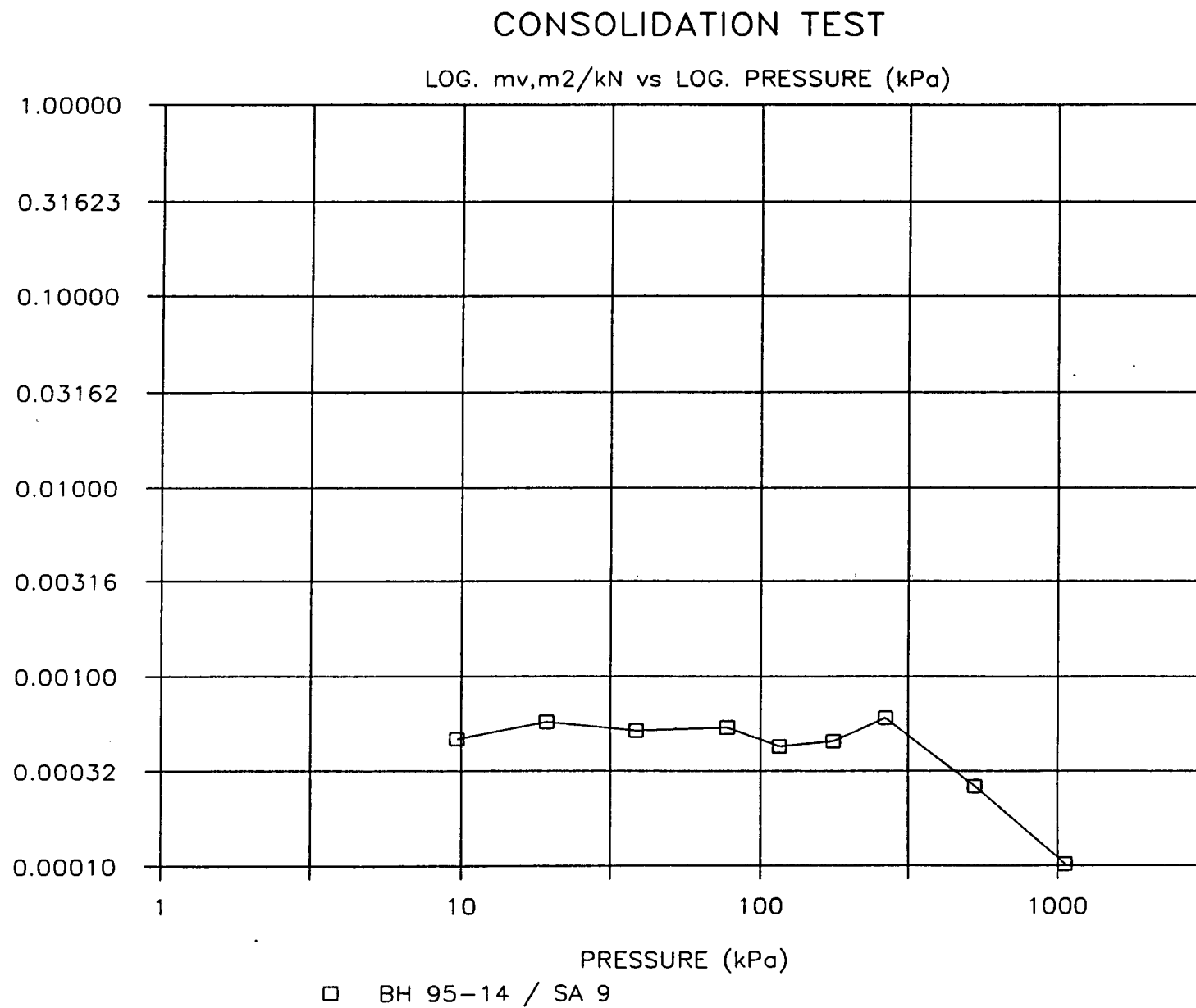
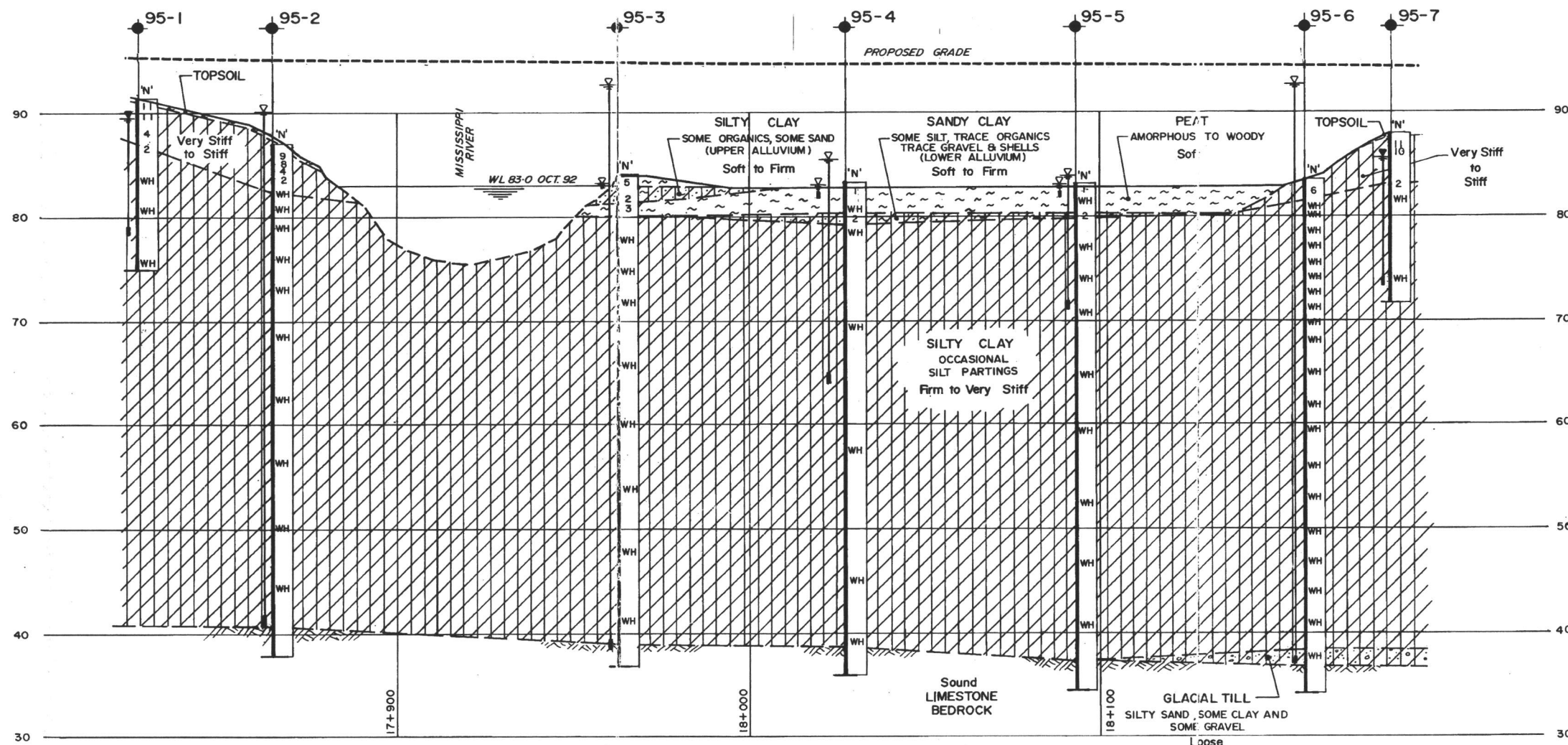
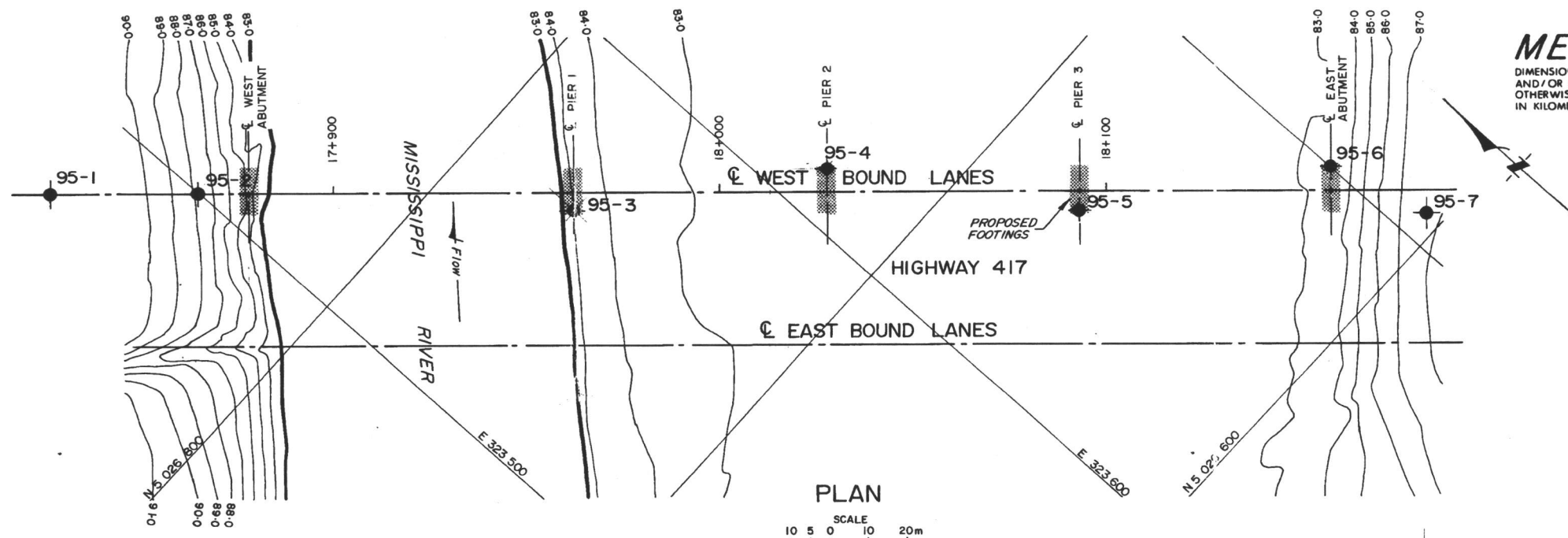


FIGURE 6-5



METRIC

DIMENSIONS ARE IN METRES AND / OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

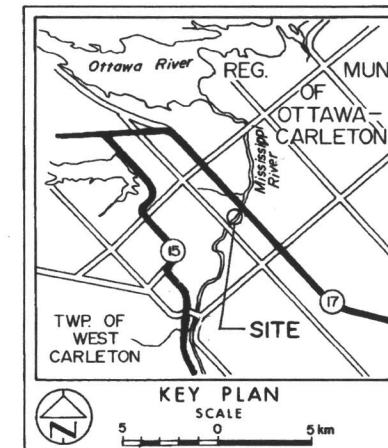
CONT No
WP No451-90-03/04

MISSISSIPPI RIVER BRIDGE
HWY 417
BORE HOLE LOCATIONS & SOIL STRATA



SHEET

THURBER ENGINEERING LTD.



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WH Weight of Hammer
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation Apr. 1995
- PIEZOMETER
- ARTESIAN WATER

No	ELEVATION	CO ORDINATES	
		NORTH	EAST
95-1	91.5	5 026 872.5	323 474.5
95-2	86.9	5 026 844.1	323 500.1
95-3	84.3	5 026 768.0	323 560.5
95-4	83.0	5 026 727.7	323 612.9
95-5	83.0	5 026 671.4	323 647.5
95-6	83.4	5 026 631.1	323 699.9
95-7	88.1	5 026 603.7	323 708.3

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

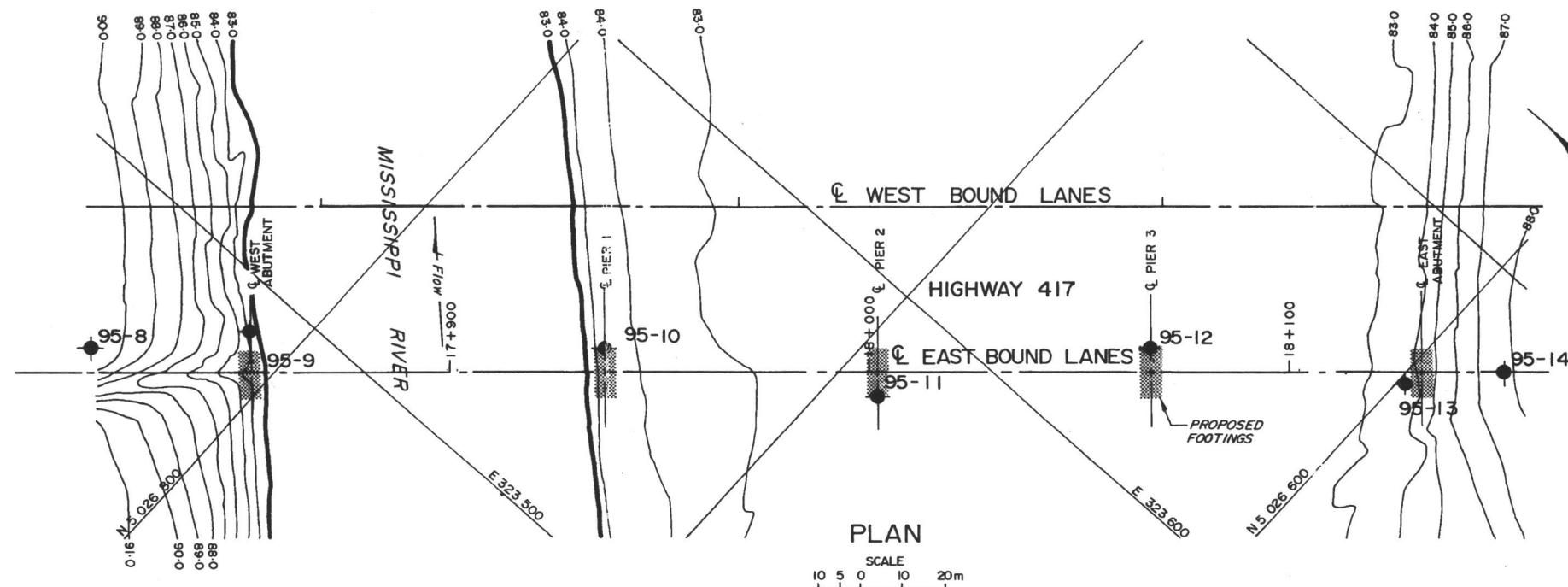
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

DATE	BY	DESCRIPTION
15 May 1995	CB	CHECKED
15 May 1995	IC	CHECKED

Geocres No 31F-117

HWY No 417	DIST 42
SUBM'D	CHECKED CB
DRAWN DW	CHECKED IC

REF. No. E-66-417-1

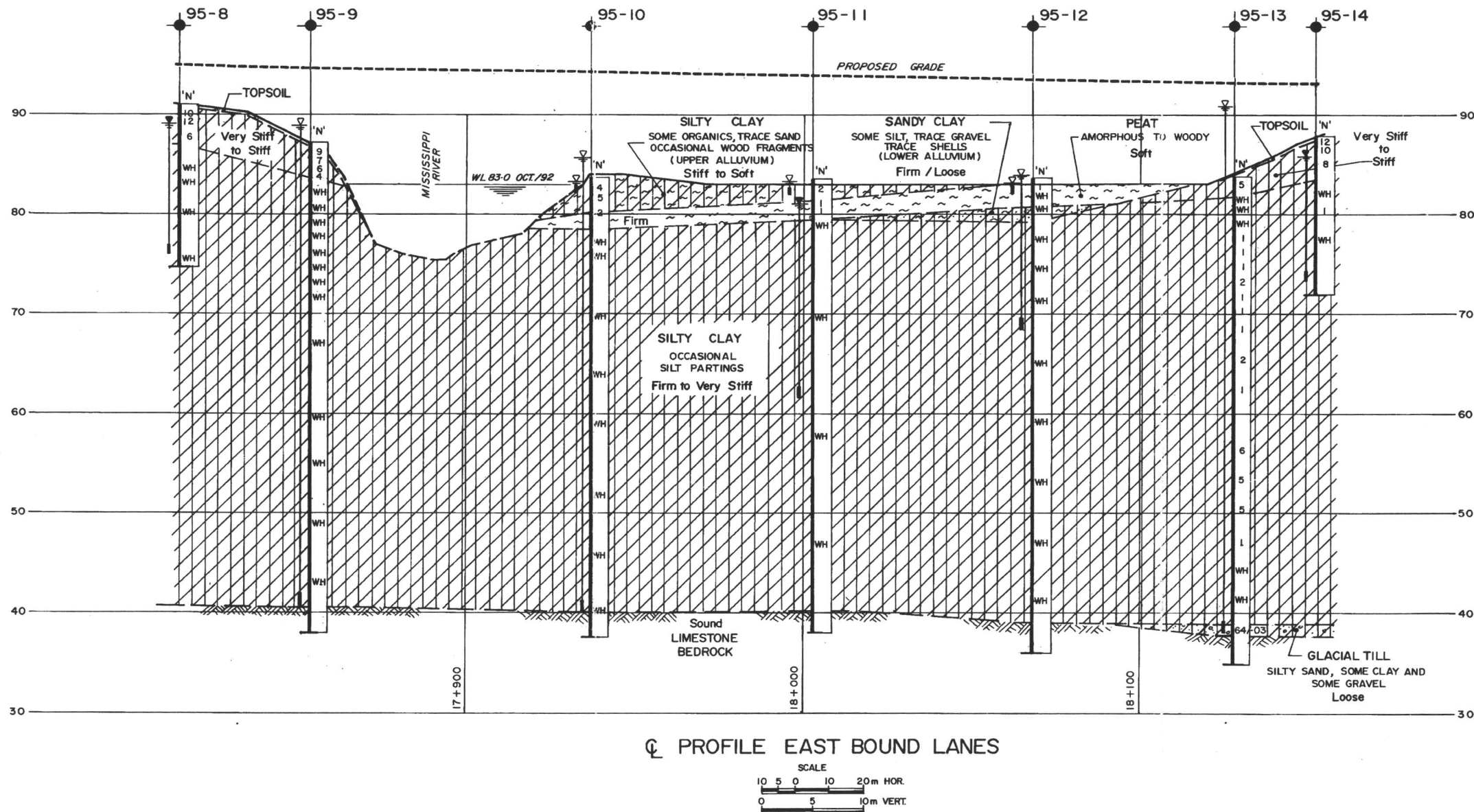
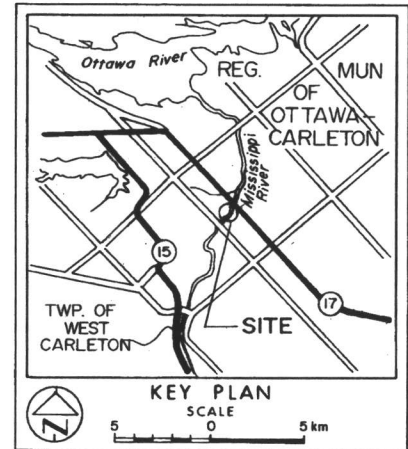


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No.451-90-03/04
MISSISSIPPI RIVER BRIDGE
HWY 417
BORE HOLE LOCATIONS & SOIL STRATA



THURBER ENGINEERING LTD.



LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WH Weight of Hammer
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation Apr. 1995
- PIEZOMETER
- ARTESIAN WATER

No	ELEVATION	CO-ORDINATES NORTH	EAST
95-8	91.1	5 026 836.3	323 460.5
95-9	87.2	5 026 810.4	323 489.6
95-10	83.4	5 026 745.3	323 542.8
95-11	83.1	5 026 689.0	323 577.4
95-12	83.1	5 026 648.7	323 629.8
95-13	83.5	5 026 597.4	323 663.9
95-14	88.4	5 026 580.1	323 683.5

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

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