# FOUNDATION INVESTIGATION REPORT

CONTRACT NO 85-70

Ministry of
Transportation and
Communications
Ontario

## INDEX



**NOTE**: For purposes of the contract, this report supersedes all other foundation reports prepared by or for the Ministry in connection with the above-mentioned project.

# EXPLANATION OF TERMS USED IN REPORT



N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ($c_u$) AS FOLLOWS:

| $c_u$ (kPa) | 0 - 12 | 12 - 25 | 25 - 50 | 50 - 100 | 100 - 200 | >200 |
|---|---|---|---|---|---|---|
| | *VERY SOFT* | *SOFT* | *FIRM* | *STIFF* | *VERY STIFF* | *HARD* |

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

| N (BLOWS/0.3m) | 0 - 5 | 5 - 10 | 10 - 30 | 30 - 50 | >50 |
|---|---|---|---|---|---|
| | *VERY LOOSE* | *LOOSE* | *COMPACT* | *DENSE* | *VERY DENSE* |

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

| RQD (%) | 0 - 25 | 25 - 50 | 50 - 75 | 75 - 90 | 90 - 100 |
|---|---|---|---|---|---|
| | *VERY POOR* | *POOR* | *FAIR* | *GOOD* | *EXCELLENT* |

JOINTING AND BEDDING:

| SPACING | 50mm | 50 - 300mm | 0.3m - 1m | 1m - 3m | >3m |
|---|---|---|---|---|---|
| JOINTING | *VERY CLOSE* | *CLOSE* | *MOD. CLOSE* | *WIDE* | *VERY WIDE* |
| BEDDING | *VERY THIN* | *THIN* | *MEDIUM* | *THICK* | *VERY THICK* |

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

| | | | |
|---|---|---|---|
| S S | SPLIT SPOON | T P | THINWALL PISTON |
| W S | WASH SAMPLE | O S | OSTERBERG SAMPLE |
| S T | SLOTTED TUBE SAMPLE | R C | ROCK CORE |
| B S | BLOCK SAMPLE | P H | T W ADVANCED HYDRAULICALLY |
| C S | CHUNK SAMPLE | P M | T W ADVANCED MANUALLY |
| T W | THINWALL OPEN | F S | FOIL SAMPLE |

### STRESS AND STRAIN

| | | |
|---|---|---|
| $u_w$ | kPa | PORE WATER PRESSURE |
| $r_u$ | 1 | PORE PRESSURE RATIO |
| $\sigma$ | kPa | TOTAL NORMAL STRESS |
| $\sigma'$ | kPa | EFFECTIVE NORMAL STRESS |
| $\tau$ | kPa | SHEAR STRESS |
| $\sigma_1, \sigma_2, \sigma_3$ | kPa | PRINCIPAL STRESSES |
| $\epsilon$ | % | LINEAR STRAIN |
| $\epsilon_1, \epsilon_2, \epsilon_3$ | % | PRINCIPAL STRAINS |
| E | kPa | MODULUS OF LINEAR DEFORMATION |
| G | kPa | MODULUS OF SHEAR DEFORMATION |
| $\mu$ | 1 | COEFFICIENT OF FRICTION |

### MECHANICAL PROPERTIES OF SOIL

| | | |
|---|---|---|
| $m_v$ | $kPa^{-1}$ | COEFFICIENT OF VOLUME CHANGE |
| $C_c$ | 1 | COMPRESSION INDEX |
| $C_s$ | 1 | SWELLING INDEX |
| $C_\alpha$ | 1 | RATE OF SECONDARY CONSOLIDATION |
| $c_v$ | $m^2/s$ | COEFFICIENT OF CONSOLIDATION |
| H | m | DRAINAGE PATH |
| $T_v$ | 1 | TIME FACTOR |
| U | % | DEGREE OF CONSOLIDATION |
| $\sigma'_{vo}$ | kPa | EFFECTIVE OVERBURDEN PRESSURE |
| $\sigma'_p$ | kPa | PRECONSOLIDATION PRESSURE |
| $\tau_f$ | kPa | SHEAR STRENGTH |
| $c'$ | kPa | EFFECTIVE COHESION INTERCEPT |
| $\phi'$ | -° | EFFECTIVE ANGLE OF INTERNAL FRICTION |
| $c_u$ | kPa | APPARENT COHESION INTERCEPT |
| $\phi_u$ | -° | APPARENT ANGLE OF INTERNAL FRICTION |
| $\tau_R$ | kPa | RESIDUAL SHEAR STRENGTH |
| $\tau_r$ | kPa | REMOULDED SHEAR STRENGTH |
| $S_t$ | 1 | SENSITIVITY $= \frac{c_u}{\tau_r}$ |

### PHYSICAL PROPERTIES OF SOIL

| | | |
|---|---|---|
| $\rho_s$ | $kg/m^3$ | DENSITY OF SOLID PARTICLES |
| $\gamma_s$ | $kN/m^3$ | UNIT WEIGHT OF SOLID PARTICLES |
| $\rho_w$ | $kg/m^3$ | DENSITY OF WATER |
| $\gamma_w$ | $kN/m^3$ | UNIT WEIGHT OF WATER |
| $\rho$ | $kg/m^3$ | DENSITY OF SOIL |
| $\gamma$ | $kN/m^3$ | UNIT WEIGHT OF SOIL |
| $\rho_d$ | $kg/m^3$ | DENSITY OF DRY SOIL |
| $\gamma_d$ | $kN/m^3$ | UNIT WEIGHT OF DRY SOIL |
| $\rho_{sat}$ | $kg/m^3$ | DENSITY OF SATURATED SOIL |
| $\gamma_{sat}$ | $kN/m^3$ | UNIT WEIGHT OF SATURATED SOIL |
| $\rho'$ | $kg/m^3$ | DENSITY OF SUBMERGED SOIL |
| $\gamma'$ | $kN/m^3$ | UNIT WEIGHT OF SUBMERGED SOIL |
| e | 1, % | VOID RATIO |
| n | 1, % | POROSITY |
| w | 1, % | WATER CONTENT |
| $S_r$ | % | DEGREE OF SATURATION |
| $w_L$ | % | LIQUID LIMIT |
| $w_P$ | % | PLASTIC LIMIT |
| $w_S$ | % | SHRINKAGE LIMIT |
| $I_P$ | % | PLASTICITY INDEX $= w_L - w_P$ |
| $I_L$ | 1 | LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$ |
| $I_C$ | 1 | CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$ |
| $e_{max}$ | 1, % | VOID RATIO IN LOOSEST STATE |
| $e_{min}$ | 1, % | VOID RATIO IN DENSEST STATE |
| $I_D$ | 1 | DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$ |
| D | mm | GRAIN DIAMETER |
| $D_n$ | mm | n PERCENT - DIAMETER |
| $C_u$ | 1 | UNIFORMITY COEFFICIENT |
| h | m | HYDRAULIC HEAD OR POTENTIAL |
| q | $m^3/s$ | RATE OF DISCHARGE |
| v | m/s | DISCHARGE VELOCITY |
| i | 1 | HYDRAULIC GRADIENT |
| k | m/s | HYDRAULIC CONDUCTIVITY |
| j | $kN/m^3$ | SEEPAGE FORCE |

Foundation Investigation Report
for
Bear Creek Bridge #3 Replacement
Hwy. #21 1.8 km N of Petrolia
W. P. 82-84-02 Site: 14-195-97
District #1 (Chatham)

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## Introduction

This report contains the results of a foundation investigation carried out at the aforementioned site from 85 01 14 to 85 01 18. The fieldwork consisted of three sampled boreholes and three dynamic cone penetration tests adjacent to each boring. The borings were advanced by a continuous flight auger machine mounted on an all-terrain vehicle and equipped with 83 mm hollow stem augers and NXL rock coring equipment.

## Site Description

The site is located about 1.8 km north of Jct. Hwy. #21 and Lambton Rd. #4 on Hwy. #21 at Bear Creek. North of the structure site, Bear Creek runs parallel with Hwy. #21 in a north-sound direction and turns sharply towards south-west at the crossing. The width of the creek at the structure site is about 26 m (measured on a 45° skew). On the north side of the creek, the land is flat and marshy (El. 193 ±) while at the south side of the existing structure the ground surface rises quite sharply (El. 205 ±). The profile grade of the existing structure is at about El. 197 ± thus requiring both fills and cuts for the approaches. The level of the creek bed is at El. 190 ±. The observed high water level is at El. 194 ±.

Physiographically, the site is located in a region referred to as the St. Clair Clay Plains.

## Subsurface Conditions

### General

The native deposits at this location in general, were found to consist of heterogeneous mixture of silty clay, sand and gravel (glacial till) followed by weathered and sound shale bedrock. In addition, a relatively shallow, surficial layer of sand and organics mixture was also observed at the low lying areas. The material in the approach fills is silty clay, most probably taken out from the adjacent cuts.

The boundaries of the different strata, together with the obtained field and laboratory test results are shown on the Record of Borehole Sheets contained in the Appendix of this report. A stratigraphical profile is shown on Drawing No. 2 of the Contract Dwg's. A description of the different strata encountered is given below.

Silty Clay some Sand and Traces of Gravel
(Fill Material)

This stratum was encountered in B.H. #1 for a depth of about 2.9 m. It is believed however, that the thickness at a particular location (south approach) due to the original topography varies between 0.3 m and about 4 m. The fill material in the north approach is assumed to be similar to the south approach.

The material in the stratum consists mainly of silty clay, some sand and traces of gravel. The in-situ moisture content is in the order of 23%. Field vane tests carried out within the fill indicate that the undrained shear strength is over 100 kPa. The consistency may be classified as stiff to very stiff.

Mixture of Sand and Organics

This surficial deposit was observed to cover the low lying areas in the vicinity of the creek. The encountered maximum thickness at the boring locations (B.H. #2 & #4) is in the order of 1.4 m. The consistency may be described as soft to stiff.

Heterogeneous Mixture of Silty Clay,
Sand and Gravel (Glacial Till)

This stratum is the main deposit at this site. The lower boundary was found to vary between El. 178 ± and El. 182 ± at the boring locations. The material consists mainly of a heterogeneous mixture of silty clay, sand and gravel (glacial till). The matrix of this till is basically cohesive in nature - i.e. silty clay binding coarser particles. Atterberg Limit Tests carried out on samples obtained within this deposit, indicate that in general the material is inorganic and of low to medium plasticity. The corresponding natural moisture content in most cases being above the plastic limit. A plot of plasticity index versus liquid limit (Figure 1) shows the point to fall within the CL and CI zones.

Physical properties of the material as determined from laboratory tests are as follows:

| | |
|---|---|
| Natural Moisture Content (%) | 11 - 30 |
| Liquid Limit (%) | 33 - 41 |
| Plastic Limit (%) | 17 - 23 |

The results of the grain-size distribution tests are shown in an envelope form on Figure #2 of the Appendix.

Standard penetration tests 'N' values varied between 3 and 28 blows per 30 cm. In B.H. #2, an approx. 0.6 m thick bouldery zone was also encountered at El. 179±. Field vane tests carried out within the deposit gave Cu values (undrained shear strength) to range from about 20 kPa to over 100 kPa. In general, the undrained shear strength of the deposit increases with depth. The consistency of the overall deposit is classified as soft to very stiff.

## Bedrock

The bedrock was proven in B.H. #2 (El. 178.1) and in B.H. #4 (El. 181.9). The bedrock at this site consists of dark grey shale followed by limestone. The upper portion of the bedrock is weathered. For details, references should be made to 'Description of Rock Core' sheet appended to this report. The core description was carried out by Mr. E. R. Magni, M.T.C., Geologist.

## Groundwater Conditions

The following groundwater levels were observed during the time (Jan./85) of the field investigation:

| | |
|---|---|
| B.H. #1 | El.193.1 |
| B.H. #2 | El.191.5 |

Artesian conditions were encountered by B.H. #4 at the contact level between the glacial till and bedrock (El. 181-9). The water level has reached equilibrium at El. 191.6 some 1.6 m over the ground level at the boring location.

REGISTERED PROFESSIONAL ENGINEER
P. PAYER
PROVINCE OF ONTARIO

P. Payer
P. Payer, P. Eng.
Senior Foundations Eng.

Brian Ru...

for K.G. Selby, P. Eng.
Chief Foundations Eng.
(West)

# APPENDIX

Ministry of Transportation and Communications
Ontario

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 1 & 1A — METRIC

W P 82-84-02 — LOCATION Sta. 25 + 354.2 O/S 6.8 m Lt. Hwy. 21 — ORIGINATED BY MJK

DIST 1 HWY 21 — BOREHOLE TYPE H.S. Auger and Dynamic Cone — COMPILED BY MJK

DATUM Geodetic — DATE 85 01 14 — CHECKED BY

## SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 197.8 | Ground Level |
| 0.0 | Silty Clay some Sand Trace of Gravel Stiff to Very Stiff Fill Material |
| 194.9 / 2.9 | Heterogeneous Mixture of Silty Clay Sand and Gravel. Stiff to Very Stiff. Glacial Till |
| 179.4 / 18.4 | End of Borehole. Weathered Shale Bedrock |

## SAMPLES

| NUMBER | TYPE | 'N' VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|
| 1 | SS | 11 | 0 12 53 35 |
| 2 | SS | 12 | |
| 3 | SS | 19 | |
| 4 | SS | 13 | |
| 5 | SS | 11 | 0 2 38 60 |
| 6 | SS | 10 | |
| 7 | SS | 12 | |
| 8 | SS | 12 | |
| 9 | SS | 12 | 0 3 47 50 |
| 10 | SS | 14 | |
| 11 | SS | 16 | |
| 12 | SS | 24 | |
| 13 | SS | 21 | 2 8 62 28 |
| 14 | SS | 24 | |
| 15 | SS | 100/17 cm | |

GROUND WATER CONDITIONS

ELEVATION SCALE: 196, 194, 192, 190, 188, 186, 184, 182, 180, 178

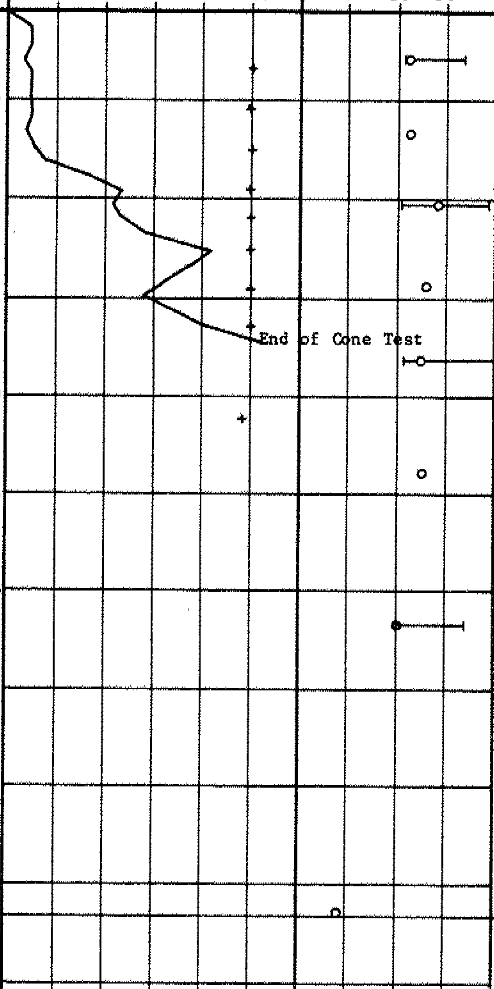
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED ✚ FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT $W_P$ — NATURAL MOISTURE CONTENT W — LIQUID LIMIT $W_L$

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ




$+^3$, $\times^5$ : Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

Ontario Ministry of Transportation and Communications

OFFICE REPORT ON SOIL EXPLORATION

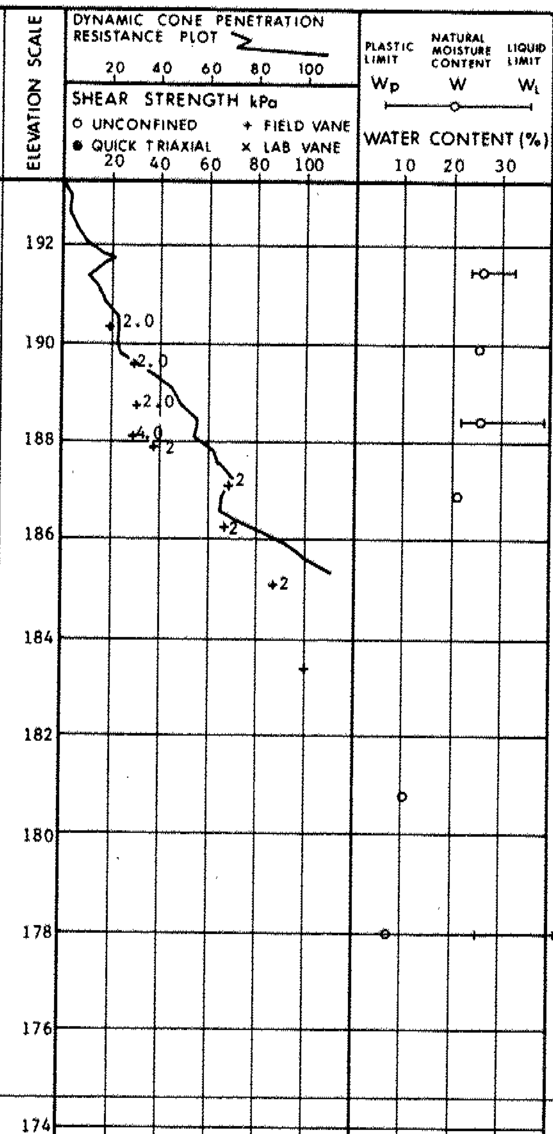
# RECORD OF BOREHOLE No 2 — METRIC

W P 82-84-02 | LOCATION Sta. 25 + 411.8 O/S 10 m Lt. Hwy. 21 | ORIGINATED BY MJK

DIST 1 HWY 21 | BOREHOLE TYPE H.S. Auger, NX Core and Dynamic Cone | COMPILED BY MJK

DATUM Geodetic | DATE 85 01 15 - 16 | CHECKED BY [signature]

| ELEV DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 193.3 | Ground Level | | | | | |
| 192.7 / 0.0 | Organic Material Soft | | | | | |
| 0.6 | Heterogeneous Mixture of Silty Clay Sand and Gravel; Soft to Very Stiff; Glacial Till | 1 | SS | 8 | | |
| | | 2 | SS | 6 | ▼ | 7 34 44 15 |
| | | 3 | SS | 6 | | |
| | | 4 | SS | 8 | | |
| | | 5 | SS | 5 | | |
| | | 6 | TW | PH | | 0 3 42 55 |
| | | 7 | SS | 11 | | |
| | | 8 | SS | 12 | | |
| | | 9 | SS | 16 | | |
| | | 10 | TW | PH | | |
| | | 11 | SS | 23 | | |
| | | 12 | SS | 28 | | |
| | Limestone Boulders | 13 | SS | 61 | | |
| 178.1 / 15.2 | Weathered / Unweathered Shale | 14 | SS | 100/15 cm | | 7 10 58 25 |
| | | 15 | RC NXL | REC 100% | | RQD: 69% |
| 174.6 | Limestone Bedrock | 16 | RC NXL | REC 93% | | RQD: 68% |
| 18.7 | End of Borehole | | | | | |




UNIT WEIGHT γ

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

# RECORD OF BOREHOLE No 4 — METRIC

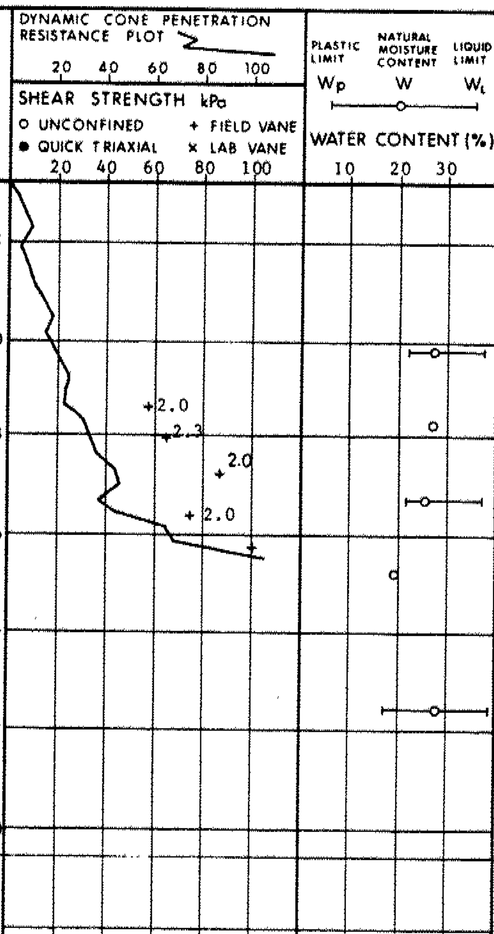
W P 82-84-02 — LOCATION Sta. 25 + 443.8 O/S 14 m Rt. Hwy. 21 — ORIGINATED BY MJK

DIST 1 HWY 21 — BOREHOLE TYPE H.S. Auger, NX Core and Dynamic Cone — COMPILED BY MJK

DATUM Geodetic — DATE 85 01 17-18 — CHECKED BY

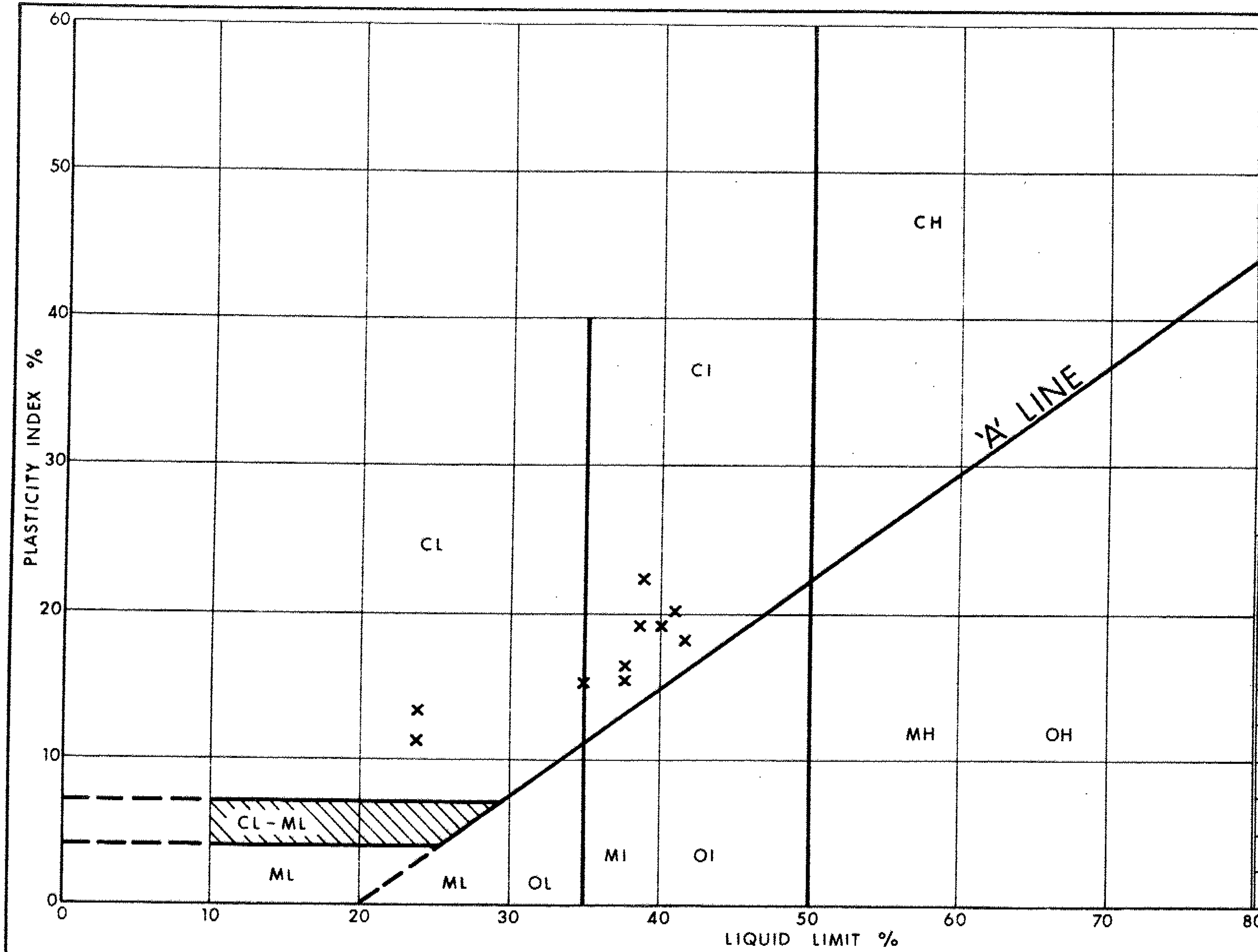
| ELEV DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | 'N' VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|
| 193.2 | Ground Level | | | | |
| 0.0 | Mixture of Sand and Organics Stiff | 1 | SS | 11 | |
| 191.8 / 1.4 | Heterogeneous Mixture of Silty Clay Sand and Gravel. Glacial Till. Firm to Very Stiff | 2 | SS | 4 | |
| | | 3 | SS | 3 | |
| | | 4 | SS | 4 | 2 27 45 26 |
| | | 5 | SS | 7 | |
| | | 6 | SS | 9 | |
| | | 7 | SS | 9 | |
| | | 8 | SS | 10 | 0 6 45 49 |
| | | 9 | SS | 20 | |
| | | 10 | SS | 21 | |
| | | 11 | SS | 27 | |
| 181.9 | | 12 | SS | 12 | 0 9 39 52 |
| 11.3 | Shale Bedrock Weathered | 14 | SS | 100/8 cm | |
| | | 15 | RC NXL | REC 100% | RQD: 29% |
| 179.5 | Shale Bedrock Unweathered | 16 | RC NXL | REC 100% | RQD: 33% |
| 13.7 | End of Borehole | | | | |

SOIL PROFILE — STRAT PLOT — SAMPLES — GROUND WATER CONDITIONS — ELEVATION SCALE — UNIT WEIGHT $\gamma$




$+^3$, $\times^5$ : Numbers refer to Sensitivity — 15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION




| LEGEND | | |
|---|---|---|
| BH | SAMPLE | SYMBOL |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |

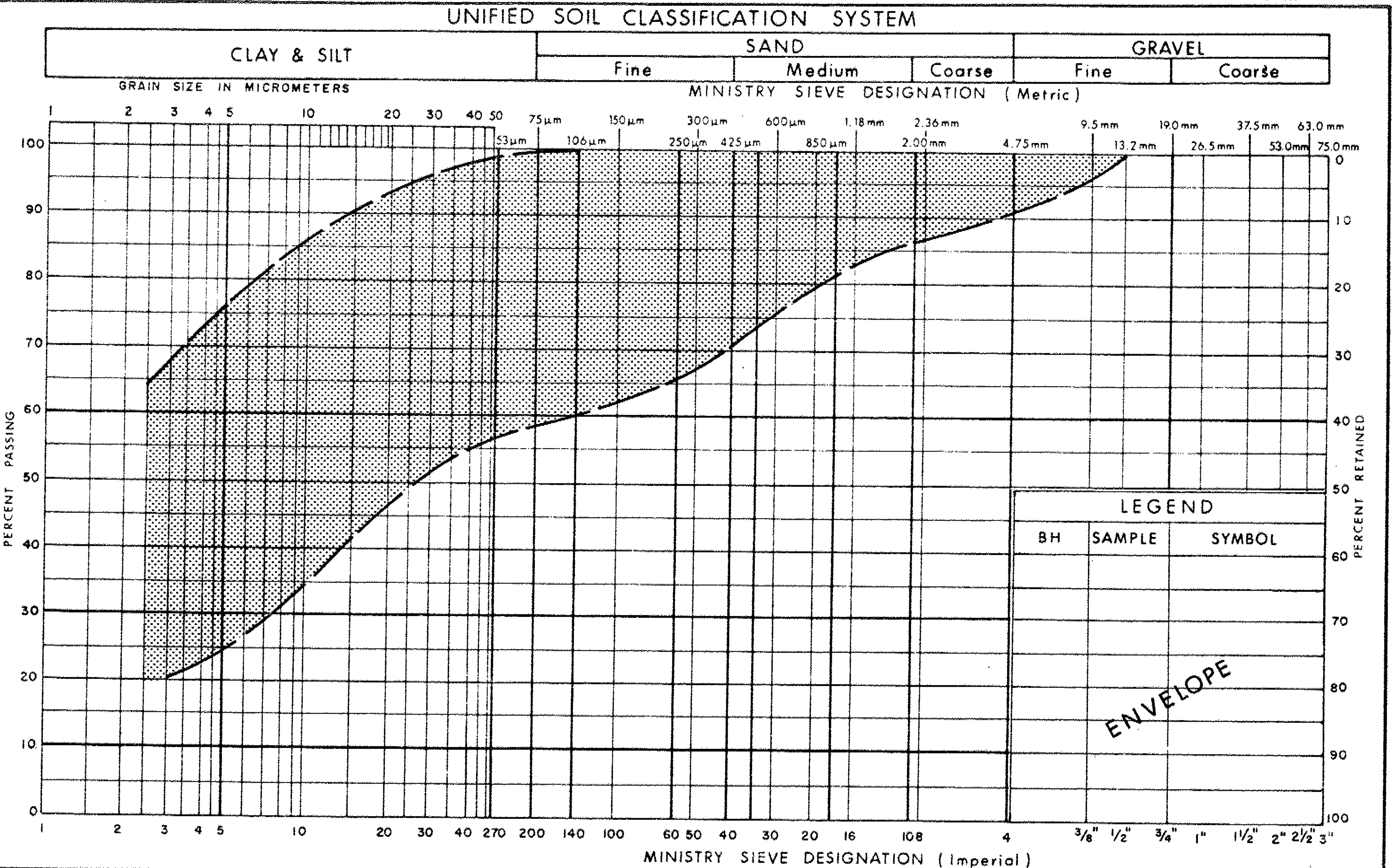
Ministry of Transportation and Communications
Ontario

# PLASTICITY CHART

HETEROGENEOUS MIXTURE OF SILTY CLAY, SAND & GRAVEL
( Glacial Till )

FIG No 1
W P 82-84-02

78 12 M

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# GRAIN SIZE DISTRIBUTION

HETEROGENEOUS MIXTURE OF SILTY CLAY, SAND & GRAVEL
( Glacial Till )

FIG No 2

W P 82 - 84 - 02

# DESCRIPTION OF ROCK CORE - W.P. 82-84-02

| BOREHOLE NUMBER | DEPTH (m) | % CR* | % RQD* | CORE DESCRIPTION DEPTH (m) | CORE DESCRIPTION DESCRIPTION |
|---|---|---|---|---|---|
| 2 | 15.85-17.17<br>17.17-18.69 | 100<br>93 | 69<br>68 | 15.85-17.09 | Shale, dark grey, unweathered, medium spaced joints, with occasional (2%) limestone layers |
| | | | | 17.09-17.96 | Shale (50%), dark grey, unweathered, medium spaced joints, and limestone (50%), unweathered, present as lenses |
| | | | | 17.96-18.69 | Limestone, grey, unweathered, medium spaced joints |
| 4 | 11.51-12.55<br>12.55-13.72 | 100<br>100 | 29<br>33 | 11.51-13.72 | Shale, dark grey, unweathered, closely to very closely spaced joints, with limestone, unweathered, from 13.25 m to 13.56 m, and shale, moderately weathered, from 13.56 m to 13.72 m |

* CR= CORE RECOVERY ; RQD= ROCK QUALITY DESIGNATION

# ENGINEERING MATERIALS OFFICE
## FOUNDATION DESIGN SECTION

85-70.

WP 82-84-02 DIST 1

HWY 21 STR SITE 14-195-97

Bear Creek Bridge #3 Replacement

## DISTRIBUTION

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GEOCRES 40J16-59 DATE

Foundation Investigation Report
for

Bear Creek Bridge #3 Replacement
Hwy. #21 1.8 km N of Petrolia
W.P. 82-84-02 Site: 14-195-97
District #1 (Chatham)

## Introduction

This report contains the results of a foundation investigation carried out at the aforementioned site from 85 01 14 to 85 01 18. The fieldwork consisted of three sampled boreholes and three dynamic cone penetration tests adjacent to each boring. The borings were advanced by a continuous flight auger machine mounted on an all-terrain vehicle and equipped with 83 mm hollow stem augers and NXL rock coring equipment.

## Site Description

The site is located about 1.8 km north of Jct. Hwy. #21 and Lambton Rd. #4 on Highway #21 at Bear Creek. North of the structure site, Bear Creek runs parallel with Hwy. #21 in a north-south direction and turns sharply towards south-west at the crossing. The width of the creek at the structure site is about 26 m (measured on a $45^{o}$ skew). On the north side of the creek, the land is flat and marshy (El. 193±) while at the south side of the existing structure the ground surface rises quite sharply (El. 205±). The profile grade of the existing structure is at about El. 197± thus requiring both fills and cuts for the approaches. The level of the creek bed is at El. 190±. The observed high water level is at El. 194±.

Physiographically, the site is located in a region referred to as the St. Clair Clay Plains.

## Subsurface Conditions

### General

The native deposits at this location in general, were found to consist of heterogeneous mixture of silty clay, sand and gravel (glacial till) followed by weathered and sound shale bedrock. In addition, a relatively shallow, surficial layer of sand and organics mixture was also observed at the low lying areas. The material in the approach fills is silty clay, most probably taken out from the adjacent cuts.

The boundaries of the different strata, together with the obtained field and laboratory test results are shown on the Record of Borehole Sheets contained in the Appendix of this report. A stratigraphical profile is shown on Drawing No. 828402-A. A description of the different strata encountered is given below.

Silty Clay some Sand and Traces of Gravel (Fill Material)

This stratum was encountered in B.H. #1 for a depth of about 2.9 m. It is believed however, that the thickness at a particular location (south approach) due to the original topography varies between 0.3 m and about 4 m. The fill material in the north approach is assumed to be similar to the south approach.

The material in the stratum consists mainly of silty clay, some sand and traces of gravel The in-situ moisture content is in the order of 23%. Field vane tests carried out within the fill indicate that the undrained shear strength is over 100 kPa. The consistency may be classified as stiff to very stiff.

Mixture of Sand and Organics

This surficial deposit was observed to cover the low lying areas in the vicinity of the creek. The encountered maximum thickness at the boring locations (B.H. #2 & #4) is in the order of 1.4 m. The consistency may be described as soft to stiff.

Heterogeneous Mixture of Silty Clay, Sand and Gravel (Glacial Till)

This stratum is the main deposit at this site. The lower boundary was found to vary between El. 178$\pm$ and El. 182$\pm$ at the boring locations. The material consists mainly of a heterogeneous mixture of silty clay, sand and gravel (glacial till). The matrix of this till is basically cohesive in nature - i.e. silty clay binding coarser particles. Atterberg Limit Tests carried out on samples obtained within this deposit, indicate that in general the material is inorganic and of low to medium plasticity. The corresponding natural moisture content in most cases being above the plastic limit. A plot of plasticity index versus liquid limit (Figure #1) shows the point to fall within the CL and CI zones.

Physical properties of the material as determined from laboratory tests are as follows:

| | Range |
|---|---|
| Natural Moisture Content (%) | 11 - 30 |
| Liquid Limit (%) | 33 - 41 |
| Plastic Limit (%) | 17 - 23 |

The results of the grain-size distribution tests are shown in an envelope form on Figure #2 of the Appendix.

Standard penetration tests 'N' values varied between 3 and 28 blows per 30 cm. In B.H. #2, an approx. 0.6 m thick bouldery zone was also encountered at El. 179$\pm$. Field vane tests carried out within the deposit gave Cu values (undrained shear strength) to range from about 20 kPa to over 100kPa. In general, the undrained shear strength of the deposit increases with depth. The consistency of the overall deposit is classified as soft to very stiff.

Bedrock

The bedrock was proven in B.H. #2 (El. 178.1) and in B.H. #4 (El. 181.9). The bedrock at this site consists of dark grey shale followed by limestone. The upper portion of the bedrock is weathered. For details, references should be made to 'Description of Rock Core' sheet appended to this report. The core description was carried out by Mr. E.R. Magni, M.T.C. Geologist.

Groundwater Conditions

The following groundwater levels were observed during the time (Jan./85) of the field investigation:

| | |
|---|---|
| B.H. #1 | El. 193.1 |
| B.H. #2 | El. 191.5 |

Artesian conditions were encountered in B.H. #4 at the contact level between the glacial till and the bedrock (El. 181.9). The water level has reached equalibrium at El. 191.6 some 1.6 m over the gound lvel at the boring location.

## DISCUSSION AND RECOMMENDATIONS

### General

It is proposed to replace the existing single span (30.48 m) concrete bow string bridge at the crossing of Hwy. #21 and Bear Creek #3. The existing structure was constructed under Cont. 29-60. The replacement structure will consist of either a single span (33.6 m long) or a three span (19.5 m - 19.5 m - 19.5 m) bridge. No horizontal or major vertical alignment changes are contemplated. It is assumed however, that the new structure will have additional width to accommodate the sidewalk or sidewalks.

### Structure Foundations

It is recommended that the replacement structure (both schemes) be founded on end-bearing steel 'H' driven to bedrock, using a safe design load of 1150 kN (HP 310 x 110) or 825 kN (HP 310 x 79) per pile. The pile tips should be reinforced with pile driving shoes. It is assumed that the piles will reach bedrock at approximately the following elevations:

| | Three Span | Single Span |
|---|---|---|
| South Abutment | El. 179.5 | El. 179.0 |
| South Pier | El. 179.0 | - |
| North Pier | El. 178.0 | - |
| North Abutment | El. 180.0 | El. 178.0 |

For purposes of the O.H.B.D.C. the following design values are recommended:

| | HP 310 x 110 | HP 310 x 79 |
|---|---|---|
| Factored Capacity at U.L.S.: | 1600 kN | 1150 kN |
| Capacity at S.L.S. Type II: | 1150 kN | 825 kN |

| | Granular 'A' | Granular 'B' |
|---|---|---|
| Angle of Internal Friction: | = $35^{o}$ | = $30^{o}$ |
| Unite Weight ($kN/m^3$) | = 22.8 | = 21.2 |

The earth pressures should be computed (assuming a non-yielding foundation and 'at rest' conditions) as per Subsection 6.6.1.2.2 of the code.

The frost protections requirement in this area is a minimum of 1.2 m of earth cover. The pile caps should be formed 'in the dry'. Therefore, a dewatering scheme will be required if the pile caps are formed below the prevailing water level.

Approach Embankments

The profile grade of the new structure proposed to be same or slightly (0.3 m) above the existing profile grade. The slopes of the approaches should not be steeper than 2:1. Should the existing approaches be widened, the removal of the surficial organic material is recommended. The fill material should not contain larger grain-sizes than 75 mm at locations where piles have to be driven.

No stability problems are anticipated.
The natural banks of the creek have shown signs of instability in the past, resulting in failures which have required remedial measures. It is recommended, therefore, that the slopes (natural) be flattened to 3:1 in the vicinity of the structure.

The creek bed and the slopes should be protected against erosion as per hydrology requirements.

Miscellaneous

The fieldwork for the project was supervised by Mr. M. Kelly, Trainee Engineer under the direction of Mr. P. Payer. The equipment used was owned and operated by Dominion Soil Investigation Inc. This report was prepared by Mr. P. Payer and approved by Mr. K.G. Selby.

P. Payer

P. Payer, P. Eng.
Foundations Engineer

REGISTERED PROFESSIONAL ENGINEER
P. PAYER
PROVINCE OF ONTARIO

K.G. Selby

K.G. Selby, P. Eng.
Chief Foundations Engineer
(West)

# APPENDIX

# EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ($c_u$) AS FOLLOWS:

| $c_u$ (kPa) | 0 - 12 | 12 - 25 | 25 - 50 | 50 - 100 | 100 - 200 | >200 |
|---|---|---|---|---|---|---|
| | *VERY SOFT* | *SOFT* | *FIRM* | *STIFF* | *VERY STIFF* | *HARD* |

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

| N (BLOWS/0.3 m) | 0 - 5 | 5 - 10 | 10 - 30 | 30 - 50 | >50 |
|---|---|---|---|---|---|
| | *VERY LOOSE* | *LOOSE* | *COMPACT* | *DENSE* | *VERY DENSE* |

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

| RQD (%) | 0 - 25 | 25 - 50 | 50 - 75 | 75 - 90 | 90 - 100 |
|---|---|---|---|---|---|
| | *VERY POOR* | *POOR* | *FAIR* | *GOOD* | *EXCELLENT* |

**JOINTING AND BEDDING:**

| SPACING | 50mm | 50 - 300mm | 0.3m - 1m | 1m - 3m | >3m |
|---|---|---|---|---|---|
| JOINTING | *VERY CLOSE* | *CLOSE* | *MOD. CLOSE* | *WIDE* | *VERY WIDE* |
| BEDDING | *VERY THIN* | *THIN* | *MEDIUM* | *THICK* | *VERY THICK* |

# ABBREVIATIONS AND SYMBOLS

## FIELD SAMPLING

| | | | |
|---|---|---|---|
| S S | SPLIT SPOON | T P | THINWALL PISTON |
| W S | WASH SAMPLE | O S | OSTERBERG SAMPLE |
| S T | SLOTTED TUBE SAMPLE | R C | ROCK CORE |
| B S | BLOCK SAMPLE | P H | T W ADVANCED HYDRAULICALLY |
| C S | CHUNK SAMPLE | P M | T W ADVANCED MANUALLY |
| T W | THINWALL OPEN | F S | FOIL SAMPLE |

## STRESS AND STRAIN

| | | |
|---|---|---|
| $u_w$ | kPa | PORE WATER PRESSURE |
| $r_u$ | 1 | PORE PRESSURE RATIO |
| $\sigma$ | kPa | TOTAL NORMAL STRESS |
| $\sigma'$ | kPa | EFFECTIVE NORMAL STRESS |
| $\tau$ | kPa | SHEAR STRESS |
| $\sigma_1, \sigma_2, \sigma_3$ | kPa | PRINCIPAL STRESSES |
| $\epsilon$ | % | LINEAR STRAIN |
| $\epsilon_1, \epsilon_2, \epsilon_3$ | % | PRINCIPAL STRAINS |
| E | kPa | MODULUS OF LINEAR DEFORMATION |
| G | kPa | MODULUS OF SHEAR DEFORMATION |
| $\mu$ | 1 | COEFFICIENT OF FRICTION |

## MECHANICAL PROPERTIES OF SOIL

| | | |
|---|---|---|
| $m_v$ | $kPa^{-1}$ | COEFFICIENT OF VOLUME CHANGE |
| $C_c$ | 1 | COMPRESSION INDEX |
| $C_s$ | 1 | SWELLING INDEX |
| $C_\alpha$ | 1 | RATE OF SECONDARY CONSOLIDATION |
| $c_v$ | $m^2/s$ | COEFFICIENT OF CONSOLIDATION |
| H | m | DRAINAGE PATH |
| $T_v$ | 1 | TIME FACTOR |
| U | % | DEGREE OF CONSOLIDATION |
| $\sigma'_{vo}$ | kPa | EFFECTIVE OVERBURDEN PRESSURE |
| $\sigma'_p$ | kPa | PRECONSOLIDATION PRESSURE |
| $\tau_f$ | kPa | SHEAR STRENGTH |
| $c'$ | kPa | EFFECTIVE COHESION INTERCEPT |
| $\phi'$ | -° | EFFECTIVE ANGLE OF INTERNAL FRICTION |
| $c_u$ | kPa | APPARENT COHESION INTERCEPT |
| $\phi_u$ | -° | APPARENT ANGLE OF INTERNAL FRICTION |
| $\tau_R$ | kPa | RESIDUAL SHEAR STRENGTH |
| $\tau_r$ | kPa | REMOULDED SHEAR STRENGTH |
| $S_t$ | 1 | SENSITIVITY $= \frac{c_u}{\tau_r}$ |

## PHYSICAL PROPERTIES OF SOIL

| | | |
|---|---|---|
| $\rho_s$ | $kg/m^3$ | DENSITY OF SOLID PARTICLES |
| $\gamma_s$ | $kN/m^3$ | UNIT WEIGHT OF SOLID PARTICLES |
| $\rho_w$ | $kg/m^3$ | DENSITY OF WATER |
| $\gamma_w$ | $kN/m^3$ | UNIT WEIGHT OF WATER |
| $\rho$ | $kg/m^3$ | DENSITY OF SOIL |
| $\gamma$ | $kN/m^3$ | UNIT WEIGHT OF SOIL |
| $\rho_d$ | $kg/m^3$ | DENSITY OF DRY SOIL |
| $\gamma_d$ | $kN/m^3$ | UNIT WEIGHT OF DRY SOIL |
| $\rho_{sat}$ | $kg/m^3$ | DENSITY OF SATURATED SOIL |
| $\gamma_{sat}$ | $kN/m^3$ | UNIT WEIGHT OF SATURATED SOIL |
| $\rho'$ | $kg/m^3$ | DENSITY OF SUBMERGED SOIL |
| $\gamma'$ | $kN/m^3$ | UNIT WEIGHT OF SUBMERGED SOIL |

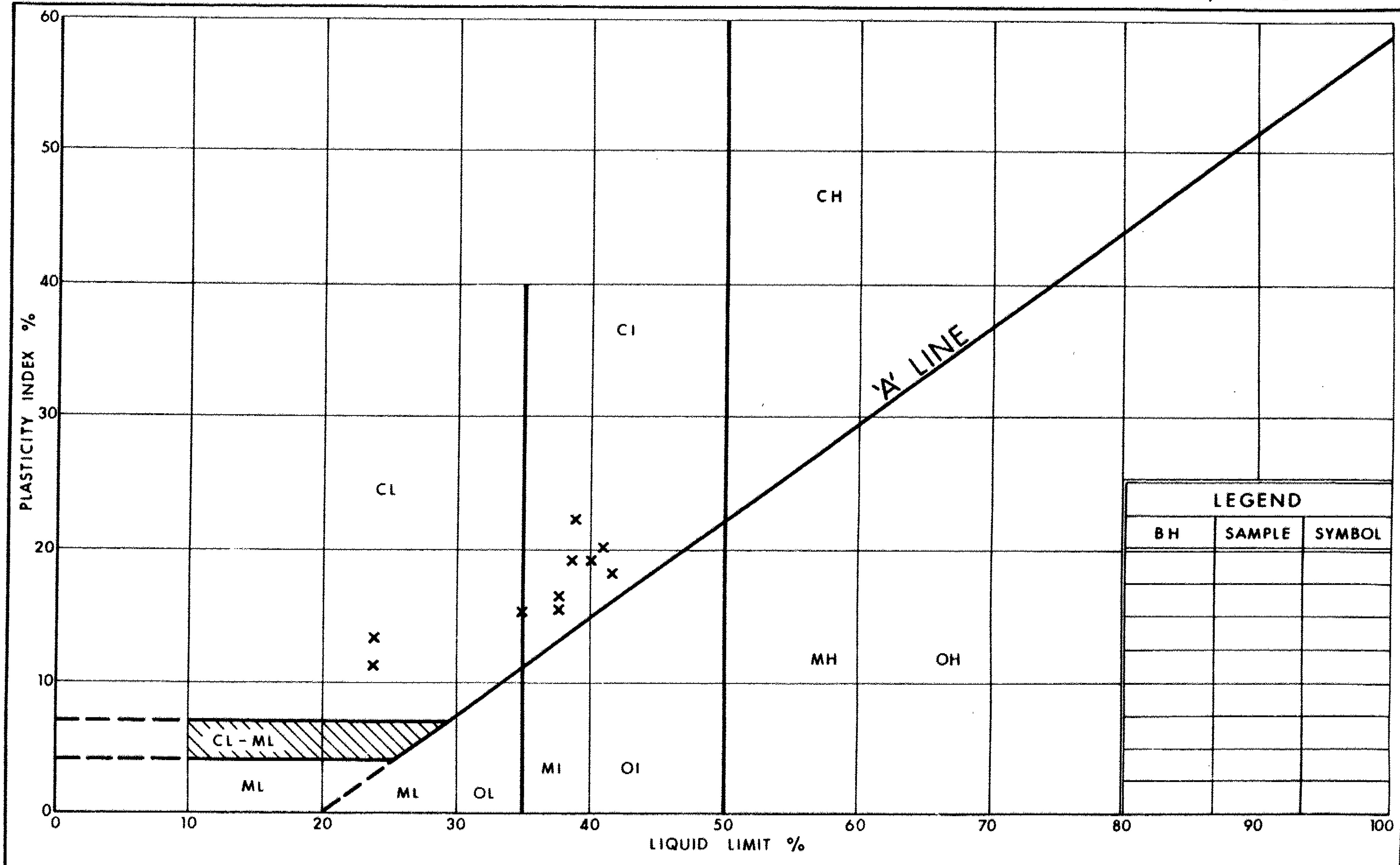
| | | |
|---|---|---|
| e | 1,% | VOID RATIO |
| n | 1,% | POROSITY |
| w | 1,% | WATER CONTENT |
| $S_r$ | % | DEGREE OF SATURATION |
| $w_L$ | % | LIQUID LIMIT |
| $w_P$ | % | PLASTIC LIMIT |
| $w_S$ | % | SHRINKAGE LIMIT |
| $I_P$ | % | PLASTICITY INDEX $= w_L - w_P$ |
| $I_L$ | 1 | LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$ |
| $I_C$ | 1 | CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$ |
| $e_{max}$ | 1,% | VOID RATIO IN LOOSEST STATE |

| | | |
|---|---|---|
| $e_{min}$ | 1,% | VOID RATIO IN DENSEST STATE |
| $I_D$ | 1 | DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$ |
| D | mm | GRAIN DIAMETER |
| $D_n$ | mm | n PERCENT - DIAMETER |
| $C_U$ | 1 | UNIFORMITY COEFFICIENT |
| h | m | HYDRAULIC HEAD OR POTENTIAL |
| q | $m^3/s$ | RATE OF DISCHARGE |
| v | m/s | DISCHARGE VELOCITY |
| i | 1 | HYDRAULIC GRADIENT |
| k | m/s | HYDRAULIC CONDUCTIVITY |
| j | $kN/m^3$ | SEEPAGE FORCE |

# DESCRIPTION OF ROCK CORE - W.P. 82-84-02

| BOREHOLE NUMBER | DEPTH (m) | % CR* | % RQD* | CORE DESCRIPTION DEPTH (m) | CORE DESCRIPTION DESCRIPTION |
|---|---|---|---|---|---|
| 2 | 15.85-17.17<br>17.17-18.69 | 100<br>93 | 69<br>68 | 15.85-17.09 | Shale, dark grey, unweathered, medium spaced joints, with occasional (2%) limestone layers |
| | | | | 17.09-17.96 | Shale (50%), dark grey, unweathered, medium spaced joints, and limestone (50%), unweathered, present as lenses |
| | | | | 17.96-18.69 | Limestone, grey, unweathered, medium spaced joints |
| 4 | 11.51-12.55<br>12.55-13.72 | 100<br>100 | 29<br>33 | 11.51-13.72 | Shale, dark grey, unweathered, closely to very closely spaced joints, with limestone, unweathered, from 13.25 m to 13.56 m, and shale, moderately weathered, from 13.56 m to 13.72 m |

* CR = CORE RECOVERY ; RQD = ROCK QUALITY DESIGNATION

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Ontario

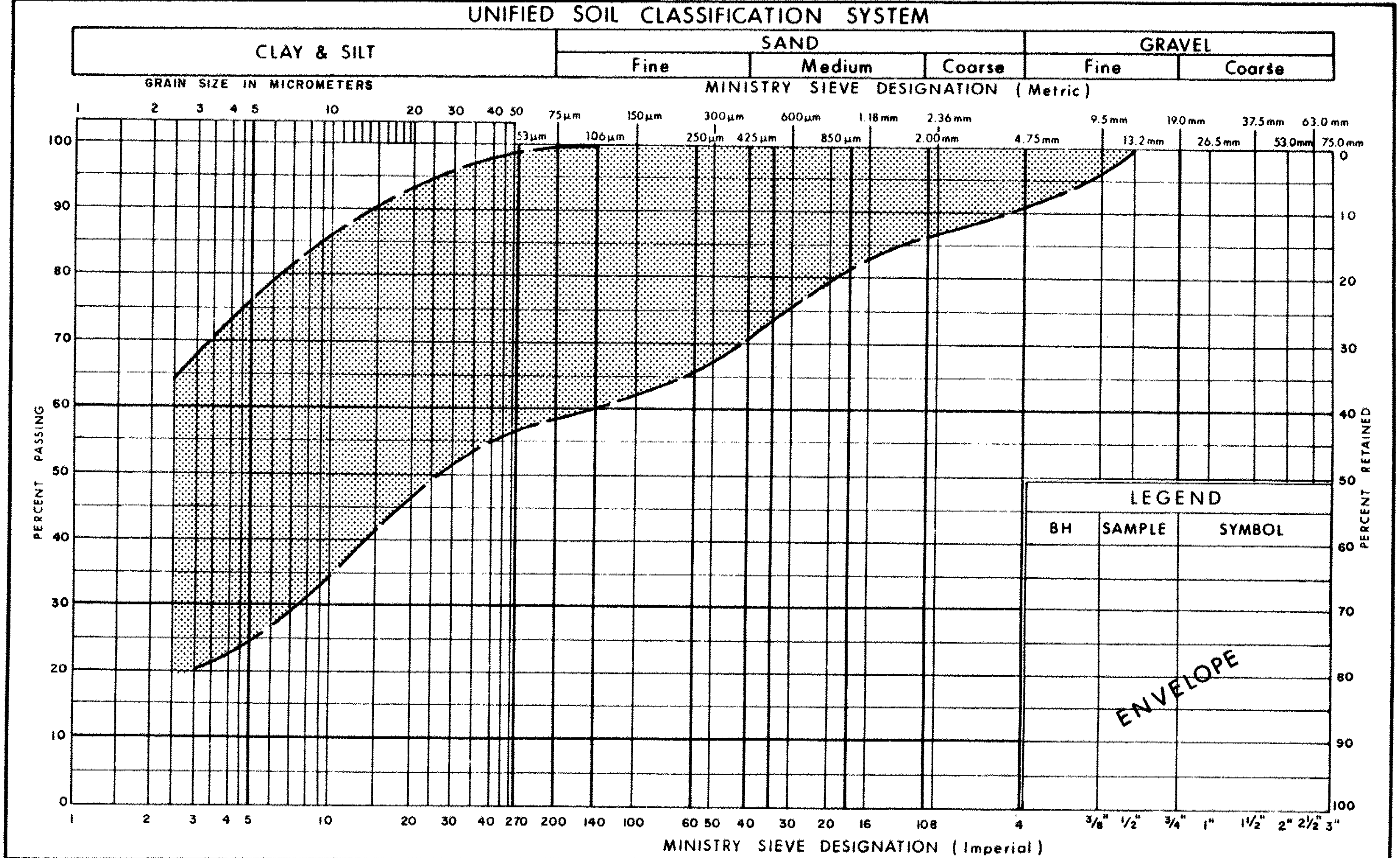
# PLASTICITY CHART

## HETEROGENEOUS MIXTURE OF SILTY CLAY, SAND & GRAVEL (Glacial Till)

FIG No 1

W P 82-84-02

78 12 M

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Ontario

# GRAIN SIZE DISTRIBUTION

HETEROGENEOUS MIXTURE OF SILTY CLAY, SAND & GRAVEL
(Glacial Till)

FIG No 2

W P 82-84-02

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# RECORD OF BOREHOLE No 1 & 1A

METRIC

W P 82-84-02 LOCATION Sta. 25 + 354.2 O/S 6.8 m Lt. Hwy. 21 ORIGINATED BY MJK

DIST 1 HWY 21 BOREHOLE TYPE H.S. Auger and Dynamic Cone COMPILED BY MJK

DATUM Geodetic DATE 85 01 14 CHECKED BY [signature]

| ELEV / DEPTH | SOIL PROFILE – DESCRIPTION | SAMPLE NUMBER | TYPE | 'N' VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|
| 197.8 | Ground Level | | | | |
| 0.0 | Silty Clay some Sand Trace of Gravel Stiff to Very Stiff Fill Material | 1 | SS | 11 | 0 12 53 35 |
| | | 2 | SS | 12 | |
| 194.9 | | 3 | SS | 19 | |
| 2.9 | Heterogeneous Mixture of Silty Clay Sand and Gravel Stiff to Very Stiff Glacial Till | 4 | SS | 13 | |
| | | 5 | SS | 11 | 0 2 38 60 |
| | | 6 | SS | 10 | |
| | | 7 | SS | 12 | |
| | | 8 | SS | 12 | |
| | | 9 | SS | 12 | 0 3 47 50 |
| | | 10 | SS | 14 | |
| | | 11 | SS | 16 | |
| | | 12 | SS | 24 | |
| | | 13 | SS | 21 | 2 8 62 28 |
| | | 14 | SS | 24 | |
| 179.4 | | 15 | SS | 100/17 cm | |
| 18.4 | End of Borehole Weathered Shale Bedrock | | | | |

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, 'N' VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT $W_p$, NATURAL MOISTURE CONTENT W, LIQUID LIMIT $W_L$, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Elevation scale: 196, 194, 192, 190, 188, 186, 184, 182, 180, 178

End of Cone Test

OFFICE REPORT ON SOIL EXPLORATION

$+^3, \times^5$ : Numbers refer to Sensitivity

15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

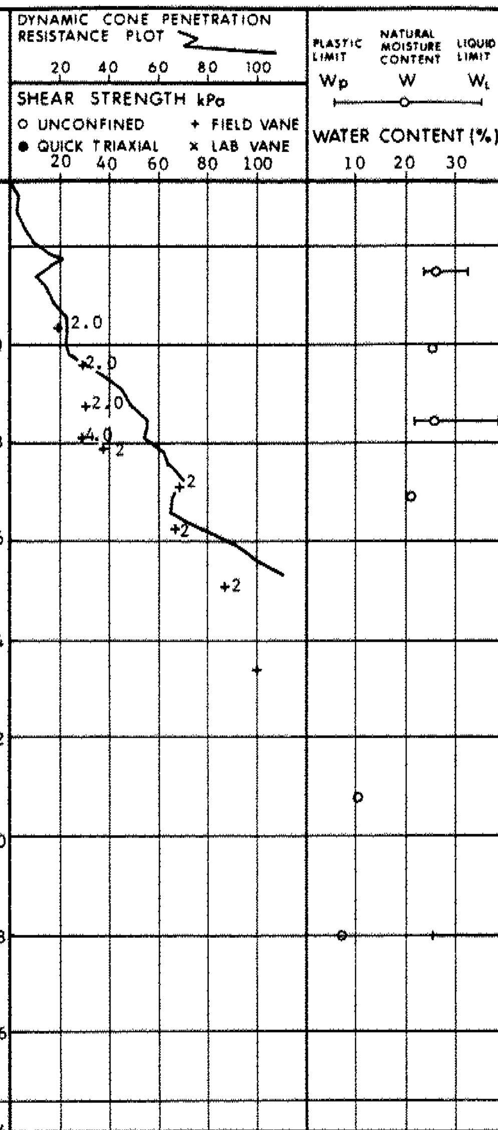
Ministry of Transportation and Communications
Ontario

# RECORD OF BOREHOLE No 2

METRIC

| | | | |
|---|---|---|---|
| W P 82-84-02 | LOCATION Sta. 25 + 411.8 O/S 10 m Lt. Hwy. 21 | ORIGINATED BY MJK |
| DIST 1 HWY 21 | BOREHOLE TYPE H.S. Auger, NX Core and Dynamic Cone | COMPILED BY MJK |
| DATUM Geodetic | DATE 85 01 15 - 16 | CHECKED BY |

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLE NUMBER | TYPE | 'N' VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 193.3 | Ground Level | | | | | | |
| 0.0 | Organic Material Soft | | | | | | |
| 192.7 0.6 | Heterogeneous Mixture of Silty Clay Sand and Gravel<br>Soft to Very Stiff<br>Glacial Till | 1 | SS | 8 | | | |
| | | 2 | SS | 6 | ▼ | 192 | 7 34 44 15 |
| | | 3 | SS | 6 | | | |
| | | 4 | SS | 8 | | 190 | |
| | | 5 | SS | 5 | | | |
| | | 6 | TW | PH | | | 0 3 42 55 |
| | | 7 | SS | 11 | | 188 | |
| | | 8 | SS | 12 | | | |
| | | 9 | SS | 16 | | 186 | |
| | | 10 | TW | PH | | 184 | |
| | | 11 | SS | 23 | | 182 | |
| | | 12 | SS | 28 | | | |
| | Limestone Boulders | 13 | SS | 61 | | 180 | |
| 178.1 15.2 | Weathered | 14 | SS | 100/15 cm | | 178 | 7 10 58 25 |
| | Unweathered Shale | 15 | RC NXL | REC 100% | | | RQD: 69% |
| | Limestone Bedrock | 16 | RC NXL | REC 93% | | 176 | RQD: 68% |
| 174.6 18.7 | End of Borehole | | | | | 174 | |




$+^3$, $\times^5$ : Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20, 10)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

# RECORD OF BOREHOLE No 4 — METRIC

W P 82-84-02 LOCATION Sta. 25 + 443.8 O/S 14 m Rt. Hwy. 21 ORIGINATED BY MJK
DIST 1 HWY 21 BOREHOLE TYPE H.S. Auger, NX Core and Dynamic Cone COMPILED BY MJK
DATUM Geodetic DATE 85 01 17-18 CHECKED BY

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | 'N' VALUES | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 193.2 | Ground Level | | | | | |
| 0.0 | Mixture of Sand and Organics Stiff | 1 | SS | 11 | 192 | |
| 191.8 / 1.4 | Heterogeneous Mixture of Silty Clay Sand and Gravel; Glacial Till; Firm to Very Stiff | 2 | SS | 4 | | |
| | | 3 | SS | 3 | | |
| | | 4 | SS | 4 | 190 | 2 27 45 26 |
| | | 5 | SS | 7 | | |
| | | 6 | SS | 9 | 188 | |
| | | 7 | SS | 9 | | |
| | | 8 | SS | 10 | | 0 6 45 49 |
| | | 9 | SS | 20 | 186 | |
| | | 10 | SS | 21 | | |
| | | 11 | SS | 27 | 184 | |
| | | 12 | SS | 12 | | 0 9 39 52 |
| 181.9 / 11.3 | Shale Bedrock Weathered | 14 | SS | 100/8 cm | 182 | |
| | | 15 | RC NXL | REC 100% | | RQD: 29% |
| | Unweathered | 16 | RC NXL | REC 100% | 180 | RQD: 33% |
| 179.5 / 13.7 | End of Borehole | | | | 178 | |

Dynamic cone penetration resistance plot; shear strength (kPa) field vane values: +2.0, +2.3, +2.0, +2.0

$+^3, \times^5$ : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

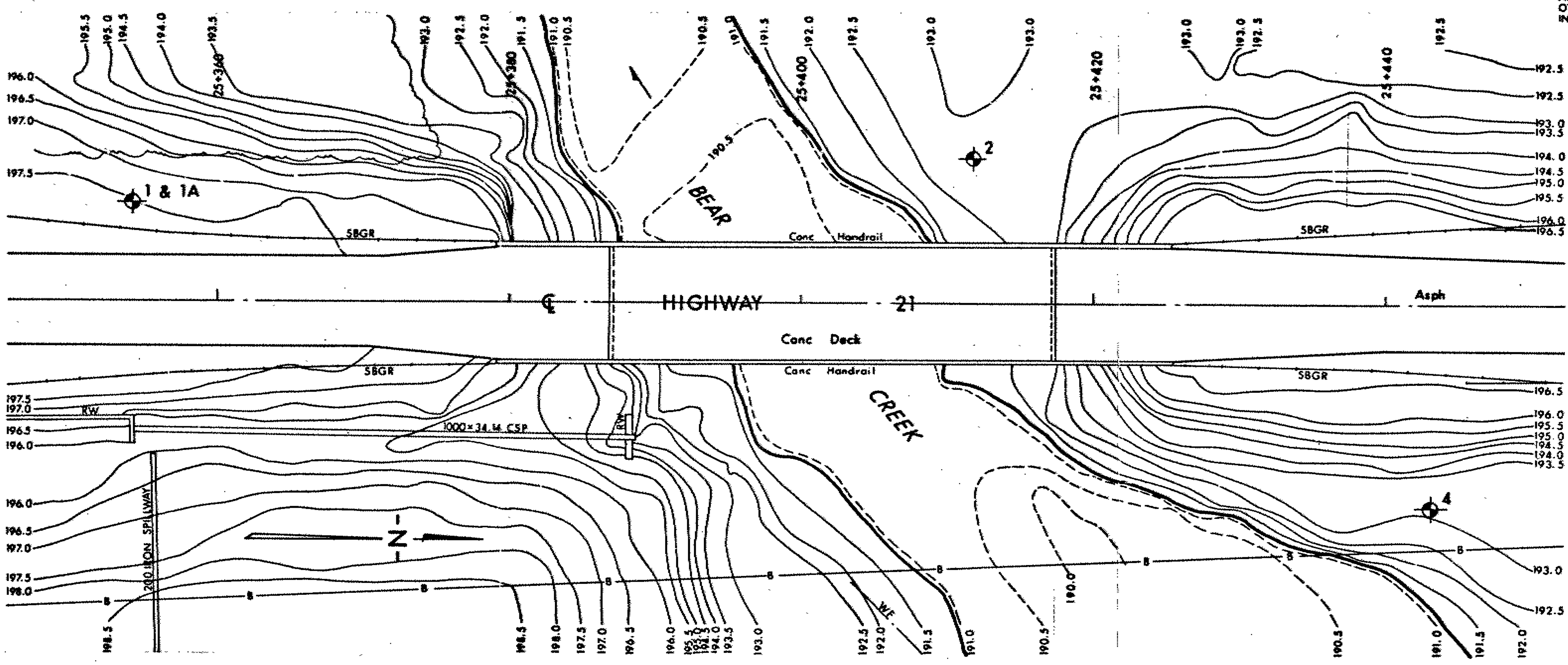
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO PH-D-207 82 04

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

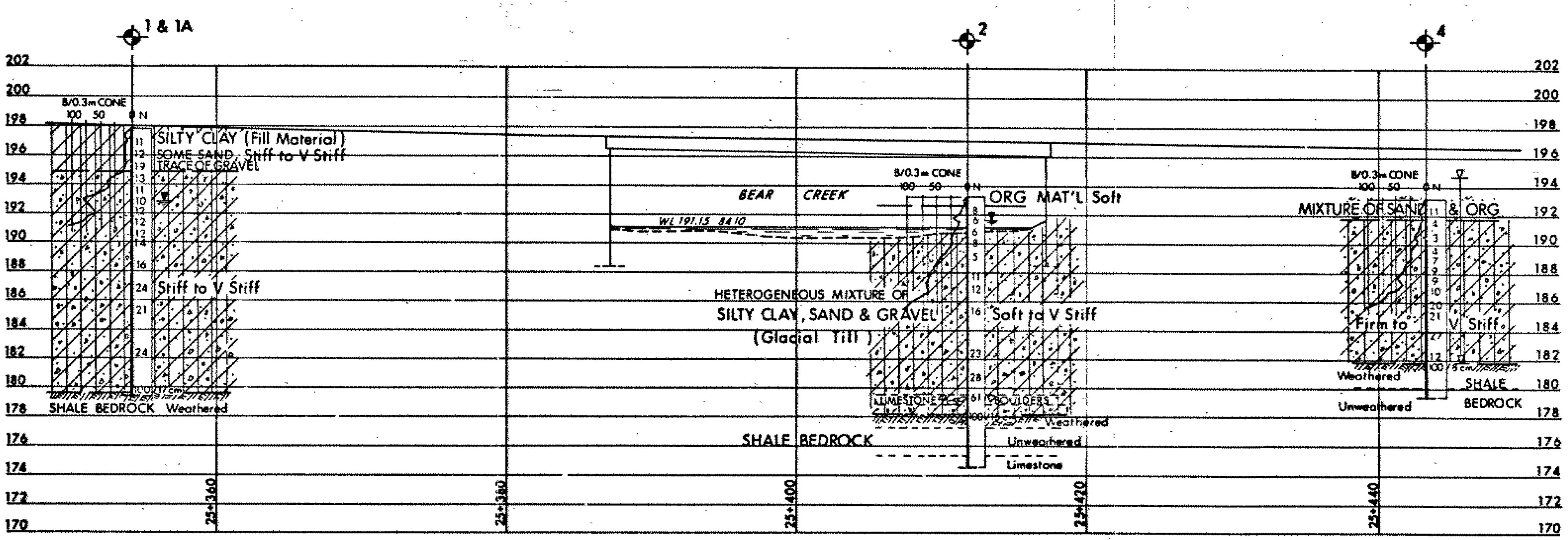
CONT No
WP No 82-84-02

SHEET

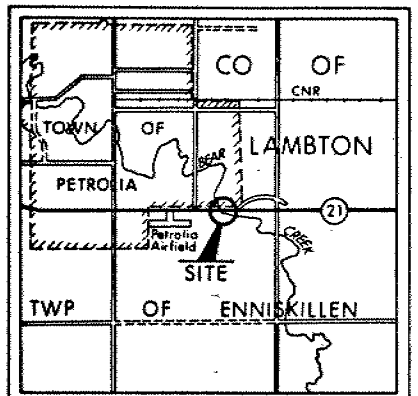
BEAR CREEK #3
BORE HOLE LOCATIONS & SOIL STRATA




## PLAN
SCALE 4m 2 0 4m




## ℄ PROFILE HWY 21
SCALE 4m 2 0 4m




KEY PLAN
SCALE 0.5 0 1km

## LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation 85 01
- Head ARTESIAN WATER Encountered

| No | ELEVATION | STATION | OFFSET |
|---|---|---|---|
| 1&1A | 197.8 | 25+354.2 | 6.8 m LT |
| 2 | 193.3 | 25+411.8 | 10.0 m LT |
| 4 | 193.2 | 25+443.8 | 14.0 m RT |

=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.

| REV. | DATE | BY | DESCRIPTION |
|---|---|---|---|

Geocres No 40J16-59

| | | | |
|---|---|---|---|
| HWY No 21 | | | DIST 1 |
| SUBM'D PP | CHECKED | DATE 85 03 15 | SITE 14-195-97 |
| DRAWN SO | CHECKED | APPROVED | DWG 828402-A |

REF NO E-10018-1 84 11

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

~~40 J - 159~~
GEOCRES No.

TO: Mr. J. G. Forster, (2)
Senior Soils Engineer,
Southwestern Region,
London, Ontario.

FROM: Foundations Office,
Design Services Branch,
West Bldg., Downsview.

ATTENTION:

DATE: December 31, 1973.

OUR FILE REF.

IN REPLY TO JAN - 4 1974

SUBJECT:

40J16-49
GEOCRES No.

*FOUNDATION INVESTIGATION REPORT*
*For*
*Bear Creek Bank Failure*
*Petrolia Bypass (Hwy. #21)*
*Twp. of Enniskillen, Co. of Lambton*
*District #1 (Chatham)*
*W.O. 73-11082*

Attached we are forwarding to you our foundation investigation report on the subsoil conditions existing at the above-mentioned site, the cause of the bank failure and the remedial measures to stabilize the slope.

We believe that the factual data and recommendations contained herein will prove adequate for your requirements. Should additional information be required, please do not hesitate to contact our Office.

A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

AP/ao
Attch.
c.c. W. Katarynczuk
A. P. Watt
A. Rutka
A. Wittenberg
J. R. Roy

Foundations Files
Documents

## TABLE OF CONTENTS

bank is the steepest. The failed area is about 135 ft. wide near the bottom of the slope and extends to a vertical height of approximately 25 ft.

## 3. *FIELD AND LABORATORY INVESTIGATION:*

A total of two sampled boreholes were carried out near the crest of the failed area. Disturbed samples were obtained in Borehole #1 by means of a standard 2 inch O.D. split spoon sampler; the energy in driving it conformed to the requirements of the Standard Penetration Test. Undisturbed samples were recovered in Borehole #2 using 2 inch I.D. Shelby tubes which were pushed into the soil hydraulically. Field vane tests were attempted 10 times from ground surface to 45 ft. depth, but it was not possible to turn the vane even once.

All boreholes and cross-sections were surveyed in the field by personnel from Chatham District Construction Surveys Section. The locations and elevations of the borings are shown on Drawing No. 73-11082A which accompanies this report.

All borehole samples were subjected to a careful visual examination and classification in the field and subsequently in the laboratory. Following this inspection, laboratory tests were carried out on selected samples to determine the following physical properties:

Atterberg Limits
Moisture Content
Grain Size Distribution
Bulk Density
Unconfined Undrained Shear Strength
Effective Stress Parameters

The results of the field and laboratory tests are summarized on the Record of Borehole Sheets contained in the Appendix of this report.

....3

If the slope was perfectly dry (i.e. $r_u = 0$) it would have a factor of safety of 1.56. However, the bank has failed indicating that a factor of safety of 1.0 was reached. This is obtained by using an $r_u$ value of 0.32 in the Bishop and Morgenstern method. This points out that the instability of the slope was caused by a build up of pore pressures in the soil mass as a result of ground water seepage.

Another contributing factor to the instability is the erosion of the toe. This removes the support at the bottom and produces locally steep slopes, which are inherently unstable. This results in progressive failure of the slope. The most critical period for these failures is the spring thaw of every year and also during the period of heavy precipitation.

In order to stabilize the present failure and to prevent further failures the following remedial measures are suggested:

That portion of the slope which lies below the fence line should be built up to a 3 horizontal to 1 vertical slope using rock fill. The rock fill should extend into the creek as shown on Drawing 73-11082A. This procedure will reinforce the toe of the slope and prevent its erosion. This treatment should extend between stations 513 + 80 and 515 + 60 and should be tapered at both ends for a distance of about 25 ft.

The portion of the slope above the fence should be covered with 1 ft. thick granular blanket. The upper limit of the blanket should be 5 - 10 ft. beyond the top of the failed area. This will help carry surface run off into the creek and prevent its seepage into the underlying soil mass.

As a further precaution a cut-off drain about 8 ft. deep and 2 ft. wide should be installed on top of the slope about 20 ft. beyond the crown or crest of the present failed area. This drain should be backfilled with free-draining granular material and should discharge into the creek. This

....5

# *APPENDIX I*

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

# RECORD OF BOREHOLE Nº 1

JOB 73-11082
W.P.
DATUM Geodetic

LOCATION Sta. 514 + 69 228' Rt. ¢
BORING DATE September 28th to October 1st, 1973
BOREHOLE TYPE Auger

ORIGINATED BY PK
COMPILED BY PK
CHECKED BY W.J.

| ELEV. / DEPTH | SOIL PROFILE: DESCRIPTION | STRAT. PLOT | SAMPLES: NUMBER | TYPE | BLOWS/FOOT | ELEV. SCALE | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) | LIQUID LIMIT $W_L$, PLASTIC LIMIT $W_P$, WATER CONTENT W (WATER CONTENT % 20 40 60) | BULK DENSITY γ P.C.F. | REMARKS GR.SA.SI.CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 198.75<br>651.9 | Ground Level | | | | | | | | | |
| 0.0<br>649.9 | Silt with sand&clay | | | | | 650 | | | | |
| 2.0 | Silty clay, Traces of sand and gravel. Very stiff<br>Brown<br>Grey | | 1 | SS | 21 | Dry Hole | | | | 0 3 43 54 |
| | | | 2 | SS | 17 | | | | | |
| | | | 3 | SS | 17 | | | | | |
| | | | 4 | SS | 18 | | | | | |
| | | | 5 | SS | 18 | 640 | | | | |
| | | | 6 | SS | 10 | | | | | |
| | | | 7 | SS | 11 | | | | | 2 9 53 36 |
| | | | 8 | SS | 15 | | | | | |
| | | | 9 | SS | 12 | | | | | |
| | | | 10 | SS | 14 | 630 | | | | |
| | | | 11 | SS | 20 | | | | | |
| | | | 12 | SS | 15 | | | | | |
| | | | 13 | SS | 15 | | | | | 0 4 43 53 |
| | | | 14 | SS | 14 | | | | | |
| | | | 15 | SS | 16 | 620 | | | | |
| | | | 16 | SS | 26 | | | | | |
| | | | 17 | SS | 21 | | | | | 0 10 60 30 |
| | | | 18 | SS | 17 | | | | | |
| | | | 19 | SS | 19 | 610 | | | | |
| | | | 20 | SS | 25 | | | | | |
| 181.8<br>596.4 | | | 21 | SS | 27 | 600 | | | | |
| 55.5<br>17m | End of Borehole | | | | | 590 | | | | WL not established |

OFFICE REPORT ON SOIL EXPLORATION

15 20 5 10 % STRAIN AT FAILURE

FD-9 (Rev. Jan.73)

## ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

### PENETRATION RESISTANCE

'N'= STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

| CONSISTENCY | c LB./SQ. FT. |
|---|---|
| VERY SOFT | 0 - 250 |
| SOFT | 250 - 500 |
| FIRM | 500 - 1000 |
| STIFF | 1000 - 2000 |
| VERY STIFF | 2000 - 4000 |
| HARD | > 4000 |

| DENSENESS | 'N' BLOWS / FT. |
|---|---|
| VERY LOOSE | 0 - 4 |
| LOOSE | 4 - 10 |
| COMPACT | 10 - 30 |
| DENSE | 30 - 50 |
| VERY DENSE | > 50 |

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10 % , SOME 10-25 % , WITH 25-40 % , > 40 % SILTY, SANDY, GRAVELLY, CLAYEY ETC.

### TYPE OF SAMPLE

| | | | |
|---|---|---|---|
| S.S. | SPLIT SPOON | T.W. | THINWALL OPEN |
| W.S. | WASHED SAMPLE | T.P. | THINWALL PISTON |
| S.T. | SLOTTED TUBE SAMPLE | O.S. | OESTERBERG SAMPLE |
| A.S. | AUGER SAMPLE | F.S. | FOIL SAMPLE |
| C.S. | CHUNK SAMPLE | R.C. | ROCK CORE |

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

### SOIL TESTS

| | | | |
|---|---|---|---|
| U | UNCONFINED COMPRESSION | L.V. | LABORATORY VANE |
| UU | UNCONSOLIDATED UNDRAINED TRIAXIAL | F.V. | FIELD VANE |
| CIU | CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL | C | CONSOLIDATION |
| CID | " " DRAINED " | S | SENSITIVITY |
| CAU | " ANISOTROPIC UNDRAINED " | | |
| CAD | " " DRAINED " | | |

FD-9b (Rev. Jan. 73)

# ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

## SOIL PROPERTIES

| | |
|---|---|
| $\gamma$ | UNIT WEIGHT OF SOIL (BULK DENSITY) |
| $\gamma_s$ | UNIT WEIGHT OF SOLID PARTICLES |
| $\gamma_w$ | UNIT WEIGHT OF WATER |
| $\gamma_d$ | UNIT DRY WEIGHT OF SOIL (DRY DENSITY) |
| $\gamma'$ | UNIT WEIGHT OF SUBMERGED SOIL |
| G | SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$ |
| e | VOID RATIO |
| n | POROSITY |
| w | WATER CONTENT |
| $S_r$ | DEGREE OF SATURATION |
| $w_L$ | LIQUID LIMIT |
| $w_P$ | PLASTIC LIMIT |
| $I_P$ | PLASTICITY INDEX |
| $w_S$ | SHRINKAGE LIMIT |
| $I_L$ | LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$ |
| $I_C$ | CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$ |
| $e_{max}$ | VOID RATIO IN LOOSEST STATE |
| $e_{min}$ | VOID RATIO IN DENSEST STATE |
| ~~$I_D$~~ | ~~DENSITY INDEX~~ $= \frac{e_{max} - e}{e_{max} - e_{min}}$ |
| | RELATIVE DENSITY $D_r$ IS ALSO USED |
| h | HYDRAULIC HEAD OR POTENTIAL |
| q | RATE OF DISCHARGE |
| v | VELOCITY OF FLOW |
| i | HYDRAULIC GRADIENT |
| k | COEFFICIENT OF PERMEABILITY |
| j | SEEPAGE FORCE PER UNIT VOLUME |
| $m_v$ | COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma'}$ |
| $c_v$ | COEFFICIENT OF CONSOLIDATION |
| $C_c$ | COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma'}$ |
| $T_v$ | TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH) |
| U | DEGREE OF CONSOLIDATION |
| $\tau_f$ | SHEAR STRENGTH |
| $c'$ | EFFECTIVE COHESION INTERCEPT |
| $\phi'$ | EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION |
| | } IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$ |
| $c_u$ | APPARENT COHESION |
| $\phi_u$ | APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION |
| | } IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$ |
| $\mu$ | COEFFICIENT OF FRICTION |
| $S_t$ | SENSITIVITY |

## GENERAL

| | |
|---|---|
| $\pi$ | $= 3.1416$ |
| e | BASE OF NATURAL LOGARITHMS 2.7183 |
| $\log_e a$ OR $\ln a$ | NATURAL LOGARITHM OF $a$ |
| $\log_{10} a$ OR $\log a$ | LOGARITHM OF $a$ TO BASE 10 |
| t | TIME |
| g | ACCELERATION DUE TO GRAVITY |
| V | VOLUME |
| W | WEIGHT |
| M | MOMENT |
| F | FACTOR OF SAFETY |

## STRESS AND STRAIN

| | |
|---|---|
| u | PORE PRESSURE |
| $\sigma$ | NORMAL STRESS |
| $\sigma'$ | NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED) |
| $\tau$ | SHEAR STRESS |
| $\varepsilon$ | LINEAR STRAIN |
| $\gamma$ | SHEAR STRAIN |
| $\nu$ | POISSON'S RATIO ($\mu$ IS ALSO USED) |
| E | MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS) |
| G | MODULUS OF SHEAR DEFORMATION |
| K | MODULUS OF COMPRESSIBILITY |
| $\eta$ | COEFFICIENT OF VISCOSITY |

## EARTH PRESSURE

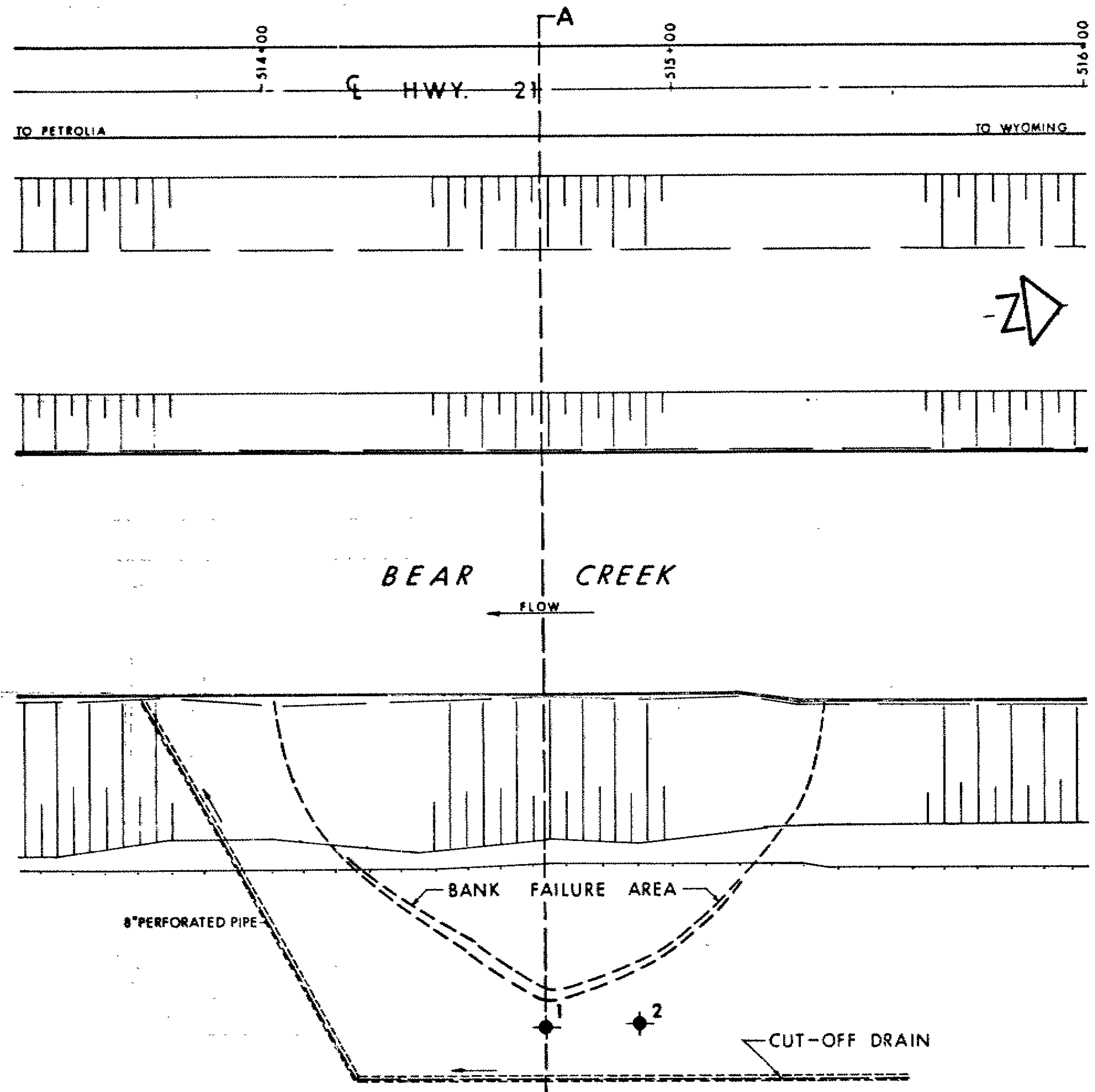
| | |
|---|---|
| d | DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE |
| $\delta$ | ANGLE OF WALL FRICTION |
| K | DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS |
| $K_o$ | COEFFICIENT OF EARTH PRESSURE AT REST |

## FOUNDATIONS

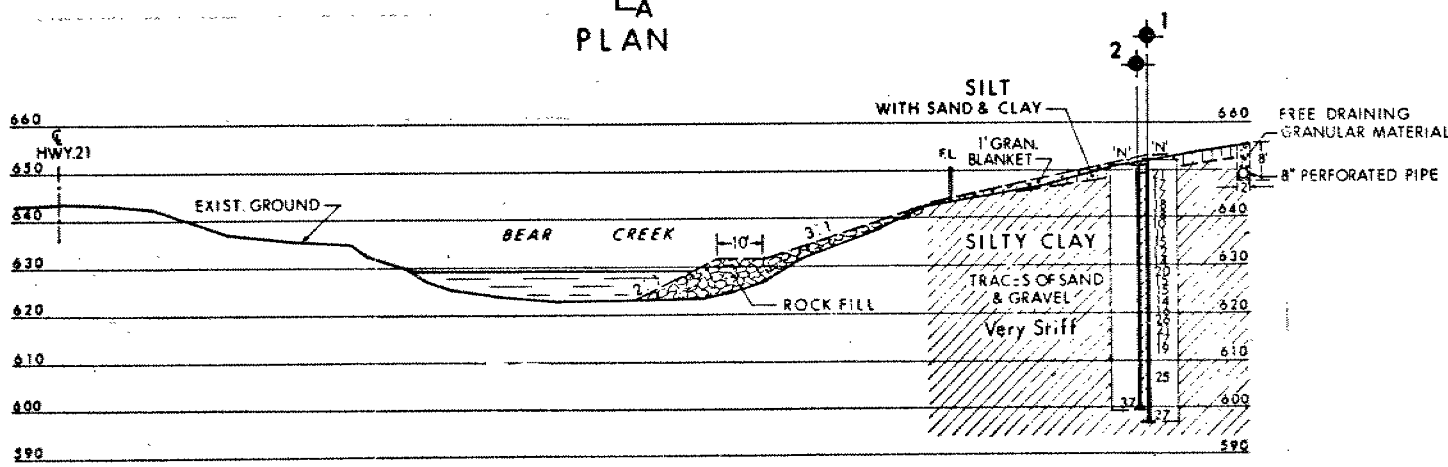
| | |
|---|---|
| B | BREADTH OF FOUNDATION |
| L | LENGTH OF FOUNDATION |
| D | DEPTH OF FOUNDATION BENEATH GROUND |
| N | DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY |
| $k_s$ | MODULUS OF SUBGRADE REACTION |

## SLOPES

| | |
|---|---|
| H | VERTICAL HEIGHT OF SLOPE |
| D | DEPTH BELOW TOE OF SLOPE TO HARD STRATUM |
| $\beta$ | ANGLE OF SLOPE TO HORIZONTAL |

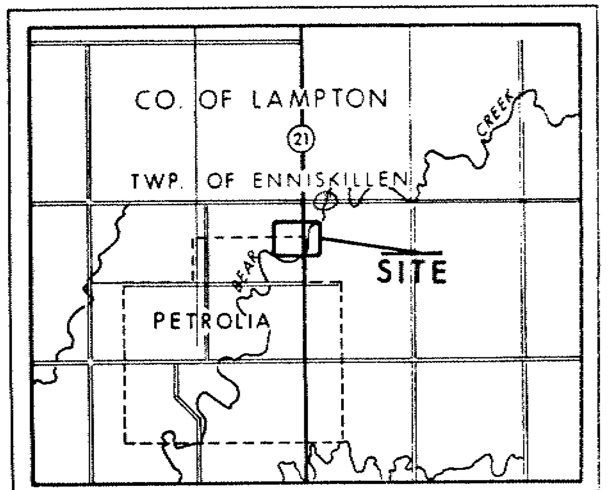



PLAN




SECTION A-A

20 10 0 20 FT.




KEY PLAN

SCALE IN MILES

0.5 0

## LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation.
- Water Levels Not established

| NO. | ELEVATION | STATION | OFFSET |
|---|---|---|---|
| 1 | 651·9 | 514+69 | 228 RT. |
| 2 | 651·0 | 514+92 | 227 RT. |

**NOTE:**
The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the CHATHAM District Office.

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

| REVISIONS | | |
|---|---|---|
| DATE | BY | DESCRIPTION |

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

# BEAR CREEK BANK FAILURE

HIGHWAY NO 21 DIST. NO. 1
CO. OF LAMPTON
TWP OF ENNISKILLEN LOT 15 & 16 CON XII

## BORE HOLE LOCATIONS & SOIL STRATA

| | | | |
|---|---|---|---|
| SUBM'D A.P. | CHECKED | W.P. NO | DRAWING NO 73-11082A |
| DRAWN OL J | CHECKED | W.O. NO 73-11082 | |
| DATE 28 DEC 1973 | | SITE NO | BRIDGE DRAWING NO |
| APPROVED | | CONT NO | |

VIEW OF THE SLIDE

73-F-223 /M

DOMINION SOIL INVESTIGATION LIMITED

CONSULTING ENGINEERS

TORONTO KITCHENER LONDON WINDSOR THUNDER BAY

DOMINION SOIL INVESTIGATION LIMITED
CONSULTING SOIL & FOUNDATION ENGINEERS
1220 TRAFALGAR ST., P.O. BOX 4033, STATION C, LONDON, ONT. (519) 455-7780

MONTEITH INGRAM ENGINEERING LTD.
CONSULTING ENGINEERS
PETROLIA ONTARIO

73-F-223 M

40 J-162
GEOCRES No.

40J16-51
GEOCRES No.

# Report On SOIL INVESTIGATION for BEAR CREEK BRIDGE LOT 16 CONCESSIONS 12/13 TOWNSHIP OF ENNISKILLEN

DIST. 1

RECEIVED
FEB 11 1974
MGR. GEOTECHNICAL OFFICE

by

Dominion Soil Investigation Limited
1220 Trafalgar Street
London Ontario

Ref: 73-8-L4
Oct. 19, 1973

STRUCTURE SITE No. 14-94

# C O N T E N T S



# E N C L O S U R E S

## I INTRODUCTION

In accordance with instructions from Monteith Ingram Engineering Limited, Consulting Engineers, a soil investigation has been carried out in the Township of Enniskillen, where it is proposed to replace an existing bridge with a new structure.

The existing 90 foot span bridge is located on the Concession 12/13 road in Lot 16 of the Township, where it crosses Bear Creek, and it is understood that the new structure will be centered on the existing structure. The requirements of the project were discussed with Mr. G. W. Ingram P.Eng., who supplied the foregoing information.

The purpose of the investigation was to reveal the subsurface conditions at the site and to determine the relevant soil properties for the design and construction of the new foundations.

## II FIELD WORK

The field work, consisting of two boreholes accompanied by two dynamic cone penetration tests, was carried out on August 16, 1973, at the locations shown on Enclosure 2.

The holes were advanced to the sampling depths by a self-propelled drilling machine which was equipped with hollow-stem augers for soil sampling.

Standard penetration tests were performed at frequent intervals of depth, as detailed in Appendix 'A', and the results are recorded on the borehole logs as 'N' values. The split-spoon samples were stored in air-tight containers and transferred to our London laboratory for testing and storage.

Insitu vane shear tests were performed in the cohesive subsoil to determine the undrained shear strength. The procedure followed in this test is outlined in Appendix 'B'.

The dynamic cone penetration tests were performed adjacent to the borehole locations to obtain an indication of soil density and strata changes with depth. The energy used to drive the cone was the same as was used for the standard penetration tests.

The field work was supervised by a soils engineer who also determined the pertinent ground surface elevations. These were referred to the top of an OWRC tablet at the southeast corner of the existing bridge, which has a known El. 643.25 feet.

## III SUBSURFACE CONDITIONS

Detailed descriptions of the strata, which were encountered in each borehole, are given on the borehole logs comprising Enclosures 3 and 4, and a general picture of the soil stratigraphy is presented in the form of a Subsurface Profile on Enclosure 2. The following notes are intended only to amplify this data.

Both boreholes encountered surface layers of weathered silty clay and clayey silt fill, which are associated with the construction of the approaches to the existing bridge. The fill extends to depths of 13 and 12 feet in boreholes 1 and 2 respectively.

Borehole 2 penetrated an alluvial deposit of silty clay which contains layers or pockets of sand and traces of wood, shells and gravel. The wood and shells indicate that it is a recent geological deposit and that it was probably laid down in shallow water. This could have occurred in a small lake or an old channel of the present creek system. Standard penetration test values within the alluvial material range from 4 to 18 blows per foot.

The predominant soil type is a glacial silty clay and it was encountered at El.631.1 and El. 621.3 in boreholes 1 and 2 respectively. The silty clay contains embedded sand, gravel, cobble and small boulders, and this type of material is commonly referred to as 'Glacial Till'. Due to the clay content, the till should be regarded as a plastic and cohesive material and the consistency is described as 'stiff' to 'very stiff' based on laboratory undrained shear strengths ranging from 900 to 2800 p.s.f., field vane shear strengths ranging from 2800 to 5300 p.s.f., and 'N' values ranging from 10 to 33 blows per foot. Atterberg Limit tests were performed on two samples of the silty clay till giving values of Liquid Limit of 30% and 37%, Plastic Limit of 20% and 21%, and Plasticity Index of 10% and 16%. These values indicated that the silty clay has a low plasticity and compressibility. The natural moisture content of the silty clay ranges from 17% to 25%, which is close to the Plastic Limit of the soil and confirms the generally 'stiff' to 'very stiff' consistency obtained from visual and tactile examination.

Shale bedrock was encountered at El. 604.6 in borehole 1 and it was cored for a depth of 5 feet. The 100% recovery shows that the shale is in a sound and unweathered condition. Refusal to further penetration by augering was also obtained at El. 604.3 in borehole 2, which is assumed to be the bedrock surface and indicates that the surface of the bedrock is relatively flat across the site. Compression tests were performed on three 1.6 inch diameter by 3.2 inch long core samples of the shale and these gave values of ultimate compressive stress ranging from 12.6 to 16.7 kips per square inch.

## IV GROUNDWATER CONDITIONS

Equilibrium water levels were observed at El. 630.1 and El. 628.3 in borehole 1 and 2 respectively, and the water level in the adjacent creek was observed at El. 619.9. The levels were recorded on August 16, 1973.

## V DISCUSSION AND RECOMMENDATIONS

The investigation has shown that the site is underlain by a considerable thickness of 'stiff' to 'very stiff' glacial silty clay, however at the east abutment location (borehole 2) there is evidence of a recent alluvial deposit which extends to a depth of about 5 feet below the present creek bed level. Spread footing foundations may be located in the silty clay till stratum or alternatively end-bearing piles can be used supported by shale bedrock at El. 604±.

### Spread Footing Foundations

The silty clay till stratum was encountered at El. 631 and El. 621 in boreholes 1 and 2 respectively, however it is recommended that all footings be located at or below El. 621 to provide sufficient protection against frost action. Also, it would be adviseable to establish the footing grade at least one foot below the surface of the silty clay stratum to allow for undulations in its surface, and the grade should therefore be lowered to El. 620.

On the basis of the borehole results a maximum allowable soil pressure of 4000 p.s.f. may be used for the design of the footings and this soil pressure incorporates a factor of safety of 3 against shear failure of the underlying soil.

Based on the results of consolidation tests on silty clay samples from nearby sites, the coefficient of volume compressibility of the stiff to very stiff silty clay has been assumed to be 0.012 sq. feet per ton, and the long-term consolidation settlement which will occur below a 12 foot wide footing carrying a dead load of 3000 p.s.f. is calculated to be 1½ inches. Applying Skempton and Bjerrum's lateral strain correction and using a value of 0.4 for pore pressure coefficient 'A', the value of the actual settlement is estimated to be 0.75 inch. In view of the uniformity of consistency, moisture content and thickness of the silty clay below the footing grade level, no appreciable differential settlement is anticipated.

The adhesion between the footings and the underlying silty clay may be taken as 1000 p.s.f., and the factor of safety against horizontal sliding of the abutments must be at least 1.5.

One problem in constructing the footings will be to control the flow of soil and water from the alluvial sand layers at the east abutment location (borehole 2), and in order to do this it is recommended that a sheet pile enclosure be used. The sheeting should be driven into the impervious silty clay subsoil and afterwards it may be left in place as a positive means of scour protection.

In the case of spread footing design it is recommended that the subgrade be covered with a lean concrete mat to prevent disturbance as soon as it has been inspected and approved. Disturbed material at the grade level should be removed and replaced with lean concrete.

## Pile Foundations

Due to proximity of the shale bedrock at a depth of about 22 feet below the creek bed, end-bearing piles should be considered. Any driven type of pile would be satisfactory, the most common types being concrete filled steel tube, and H-piles. The following working loads may be used for the design of piles driven to refusal in the bedrock.

| Pile Type | Working Load Tons |
|---|---|
| 8 BP 36 | 63 |
| 10 BP 42 | 74 |
| 10 BP 57 | 100 |
| 12 BP 53 | 93 |
| 10.75 inch Tube (4000 p.s.i. concrete) | 60 |
| 12.75 inch Tube (4000 p.s.i. concrete) | 100 |

The loadings for the H-piles are based on a stress of 12 kips per square inch over the area of the pile, which is less than the compressive strength of the shale bedrock.

The general rule for refusal of a pile is that 5 blows of an adequate hammer produce a total penetration of ¼ inch. Driving should then cease providing that the pile has not hit an obstruction, and has been driven to a depth at which refusal is expected. When piles are driven to refusal in rock, pile loading tests are not considered necessary.

REGISTERED PROFESSIONAL ENGINEER
C. J. W. ATKINSON
PROVINCE OF ONTARIO

Yours very truly,

DOMINION SOIL INVESTIGATION LTD.

C.J.W.Atkinson, M.Sc.,P.Eng.
Branch Manager

CJWA:eg

APPENDIX 'A'

## THE STANDARD PENETRATION TEST.

In order to determine the relative density of non-cohesive soils, such as sands and gravels, the standard penetration test has been adopted. The test also gives an indication of the consistency of cohesive soils.

A two inch external diameter thick-walled sample tube is driven into the ground at the bottom of the borehole by means of a 140 lb. hammer falling freely through 30-ins. The tube is first driven an initial 6-inches to allow for the presence of disturbed material at the bottom of the borehole. The number of standard blows (N) required to drive the sampler a further 12-in. is recorded. The sample tube is one originally developed by Raymond Concrete Pile Company in the United States, where a sufficient number of tests have been made in conjunction with field investigations to show that the results, although essentially empirical, may be applied to foundation design.

For Sands:-

| Values of 'N' | Density |
|---|---|
| Less than 10 | Loose |
| Between 10 and 30 | Compact |
| Between 30 and 50 | Dense |
| Greater than 50 | Very dense |

APPENDIX 'B'

## INSITU VANE SHEAR TEST

In soft to stiff clays, and particularly sensitive clay soils such as frequently occur in alluvial deposits, it is difficult to obtain reasonable undisturbed samples for the determination of the undrained shear strength. In order to overcome this difficulty, the vane test was developed as an in-situ method of measuring the shear strength.

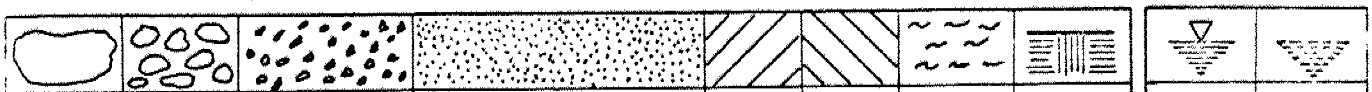
The apparatus consists of a 4-inch long by 2-inch wide rectangular 4-bladed rotating vane attached to a thin rod, which is pushed into the undisturbed soil below the bottom of the bore-hole to the depth at which the test is to be made.

A torque is then applied to the vane and the maximum torque when failure occurs is recorded. The vane is then rotated 10 times to remould the soil and after one minute the torque test is repeated. The shear strength of the soil can then be calculated from the torque and the dimensions of the vane, and the sensitivity of the material estimated from the ratio of the original torque to the final torque after remoulding.

Enclosure No. 1.

# LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

## SOIL COMPONENTS AND GROUND WATER CONDITIONS.



| BOULDER | COBBLE | GRAVEL | | SAND | | | SILT | CLAY | ORGANICS | BEDROCK | GROUND WATER LEVEL | DEPTH OF CAVE-IN |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | COARSE | FINE | COARSE | MEDIUM | FINE | | | | | | |
| Ø > 8" | 3" | 3/4" | 4.76mm | 2.0 | 0.42 | 0.074 | 0.002 | > | NO SIZE LIMIT | | | |

U.S. Standard Sieve Size: No.4 No.10 No.40 No.200

## SAMPLE TYPES.

AS Auger sample
CS Sample from casing
ChS Chunk sample

RC Rock core
% Recovery
SS Split spoon sample

TP Piston, thin walled tube sample
TW Open, thin walled tube sample
WS Wash sample

SAMPLER ADVANCED BY static weight : w
" pressure : p
" tapping : t

OBSERVATIONS MADE WHILE CORING: Steady pressure; No pressure; Intermittent pressure; Washwater returns; Washwater lost

## PENETRATION RESISTANCES.

SYMBOL:

DYNAMIC PENETRATION RESISTANCE : to drive a 2"ϕ, 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

STANDARD PENETRATION RESISTANCE, -N- : to drive a 2" outside dia, split spoon sampler 1 foot into the ground, expressed in blows per foot.

EXTRAPOLATED -N- VALUE

322

The energy for the penetration resistances is supplied by a 140 lb. hammer falling 30 inches

## SOIL PROPERTIES.

W % Water content
LL % Liquid limit
PL % Plastic limit
PI % Plasticity index
LI Liquidity index

$\gamma$ Natural bulk density (unit weight)
e Void ratio
RD Relative density
$C_v$ Coeff. of consolidation
$m_v$ Coeff. of volume compressibility

k Coeff. of permeability
C Shear strength — in terms of total stress
ϕ Angle of int. friction — in terms of total stress
C' Cohesion — in terms of effective stress
ϕ' Angle of int. friction — in terms of effective stress

## UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL COMPRESSION TEST ◎
UNCONFINED COMPRESSION TEST ●St

LABORATORY VANE TEST X St
FIELD VANE TEST + St

POCKET PENETROMETER TEST St

Strain at failure is represented by direction of stem: 20% 15% 5% 10%

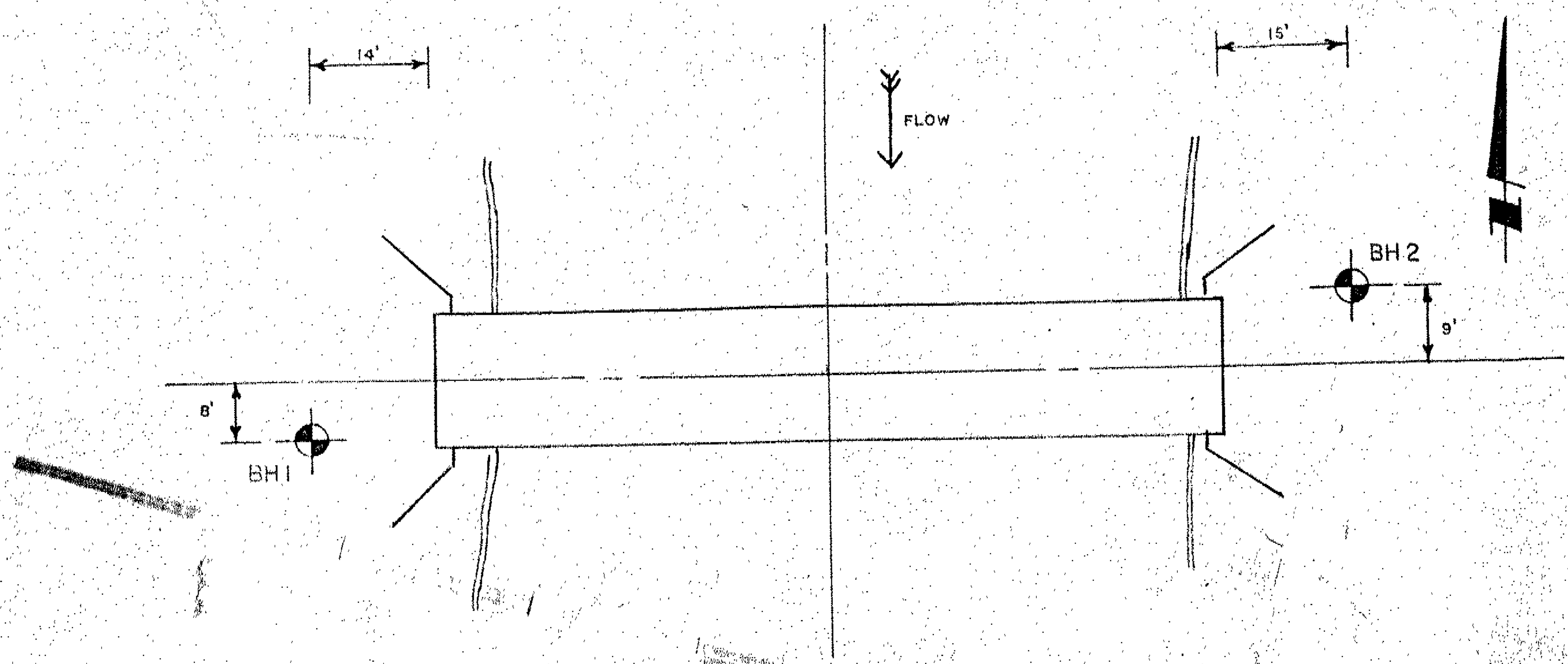
St : sensitivity = $\frac{\text{shear strength in undisturbed state}}{\text{shear strength in remoulded state}}$

## SOIL DESCRIPTION.

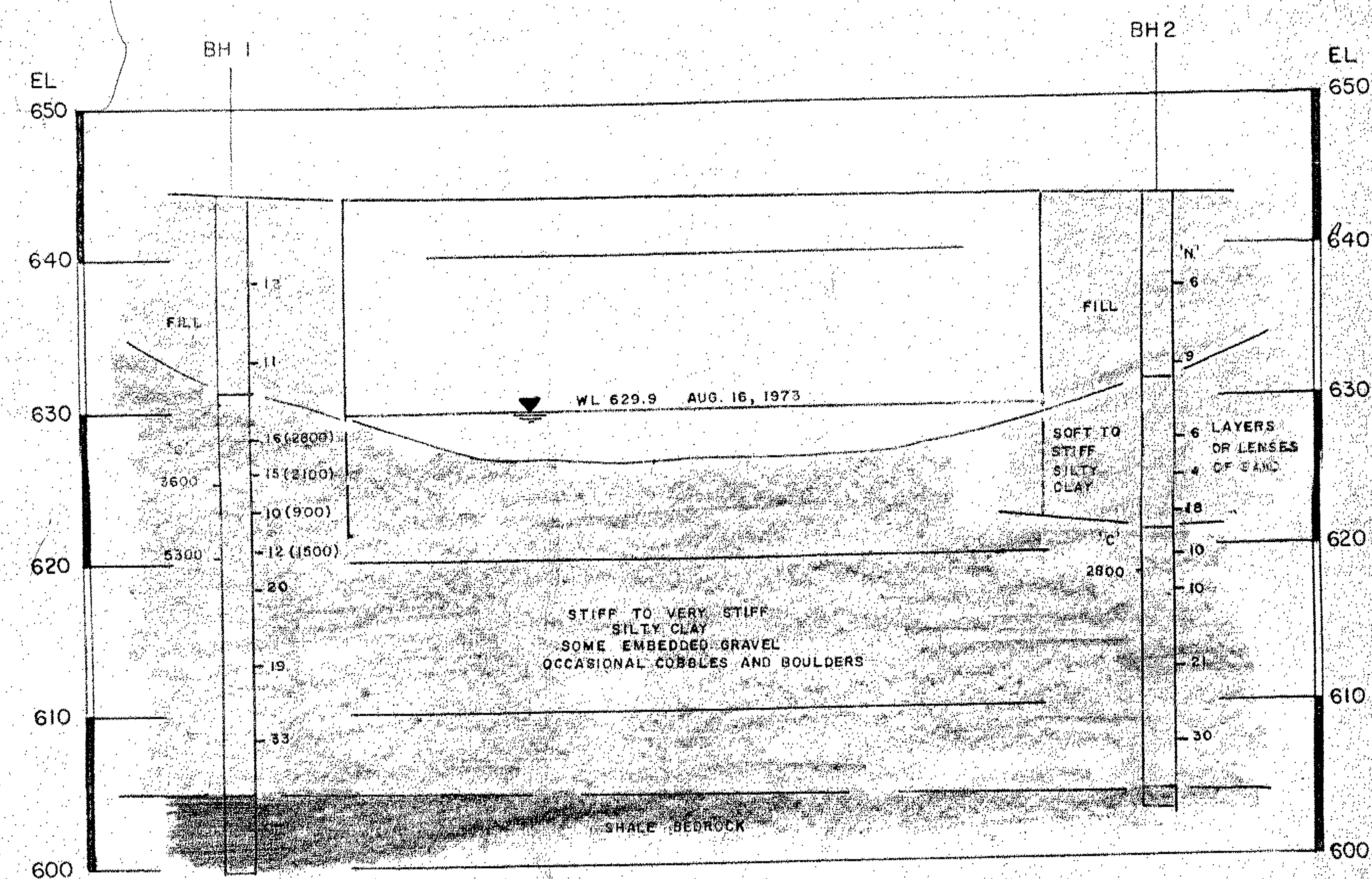
| COHESIONLESS SOILS : | RD : |
|---|---|
| Very loose | 0 – 15 % |
| Loose | 15 – 35 % |
| Compact | 35 – 65 % |
| Dense | 65 – 85 % |
| Very dense | 85 – 100 % |

| COHESIVE SOILS | C lbs/sq.ft. |
|---|---|
| Very soft | less than 250 |
| Soft | 250 – 500 |
| Firm | 500 – 1000 |
| Stiff | 1000 – 2000 |
| Very stiff | 2000 – 4000 |
| Hard | over 4000 |

DOMINION SOIL INVESTIGATION LIMITED




LOCATION OF BOREHOLES
SCALE 1" = 20'




SUBSURFACE PROFILE
VERT. SCALE 1" = 10'

OUR REF. No. 73 - 8 - L4

**DOMINION SOIL INVESTIGATION LIMITED**

ENCLOSURE No. 2

# LOG OF BOREHOLE 1

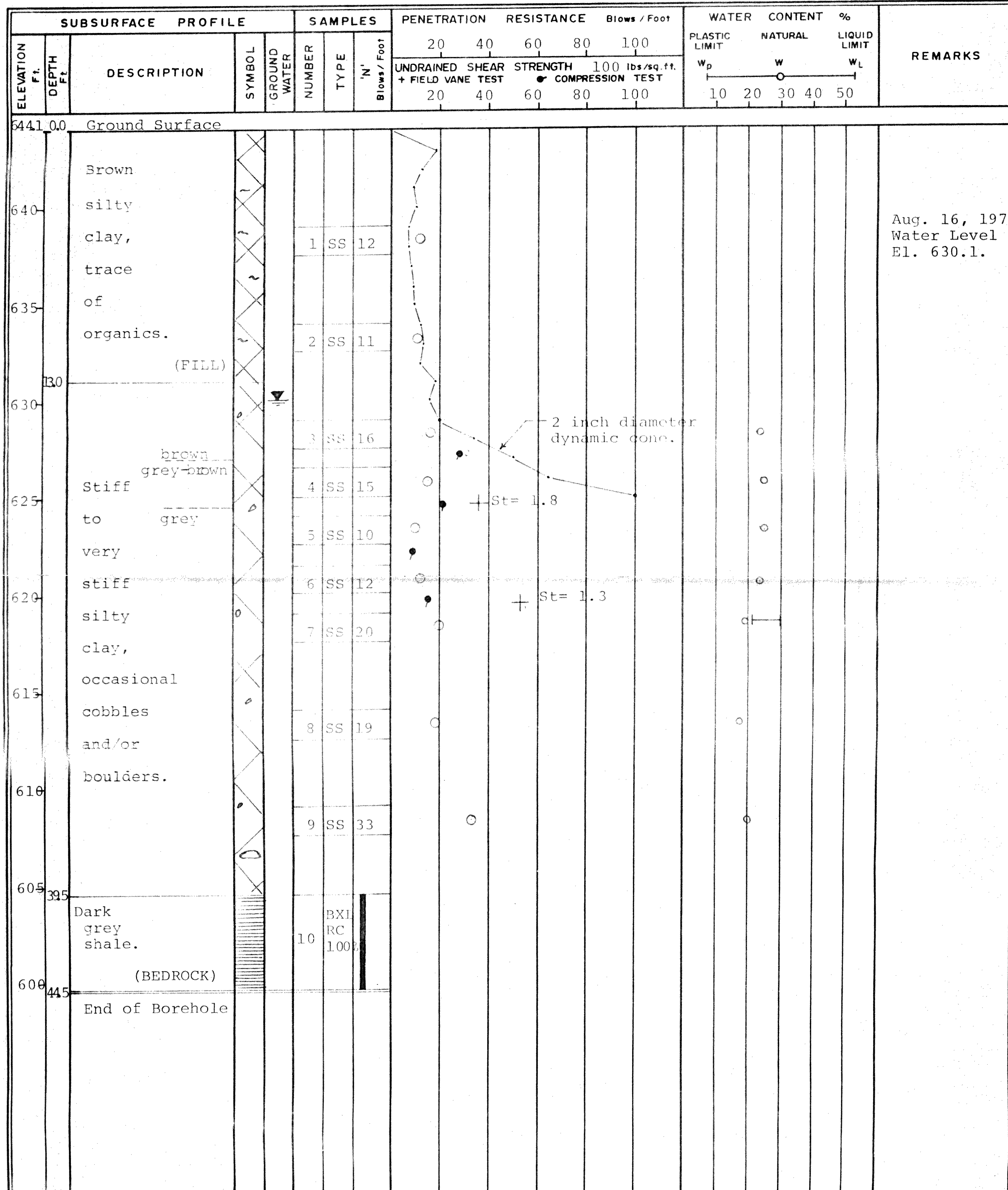
Our Reference Nº 73-8-L4

Enclosure Nº 3

CLIENT: Monteith Ingram Engineering Limited
PROJECT: Proposed Bridge (Bear Creek)
LOCATION: Township of Enniskillen
DATUM ELEVATION: Geodetic

**DRILLING DATA**
Method: Hollow-stem auger
Diameter: 8-inch
Date: August 16, 1973.

| Elevation Ft. | Depth Ft. | Description | Sample Number | Type | 'N' Blows/Foot | Remarks |
|---|---|---|---|---|---|---|
| 644.1 | 0.0 | Ground Surface | | | | |
| | | Brown silty clay, trace of organics. (FILL) | 1 | SS | 12 | Aug. 16, 1973 Water Level El. 630.1. |
| | | | 2 | SS | 11 | |
| | 13.0 | Stiff to very stiff brown grey-brown grey silty clay, occasional cobbles and/or boulders. | 3 | SS | 16 | |
| | | | 4 | SS | 15 | |
| | | | 5 | SS | 10 | |
| | | | 6 | SS | 12 | |
| | | | 7 | SS | 20 | |
| | | | 8 | SS | 19 | |
| | | | 9 | SS | 33 | |
| | 39.5 | Dark grey shale. (BEDROCK) | 10 | BXL RC 100% | | |
| | 44.5 | End of Borehole | | | | |




VERTICAL SCALE: 1 inch to 5 feet

MADE:

CHECKED:

DOMINION SOIL INVESTIGATION LIMITED

# LOG OF BOREHOLE 3

Our Reference No 73-8-L4

Enclosure No 4

CLIENT: Monteith Ingram Engineering Limited
PROJECT: Proposed Bridge (Bear Creek)
LOCATION: Township of Enniskillen
DATUM ELEVATION: Geodetic

**DRILLING DATA**

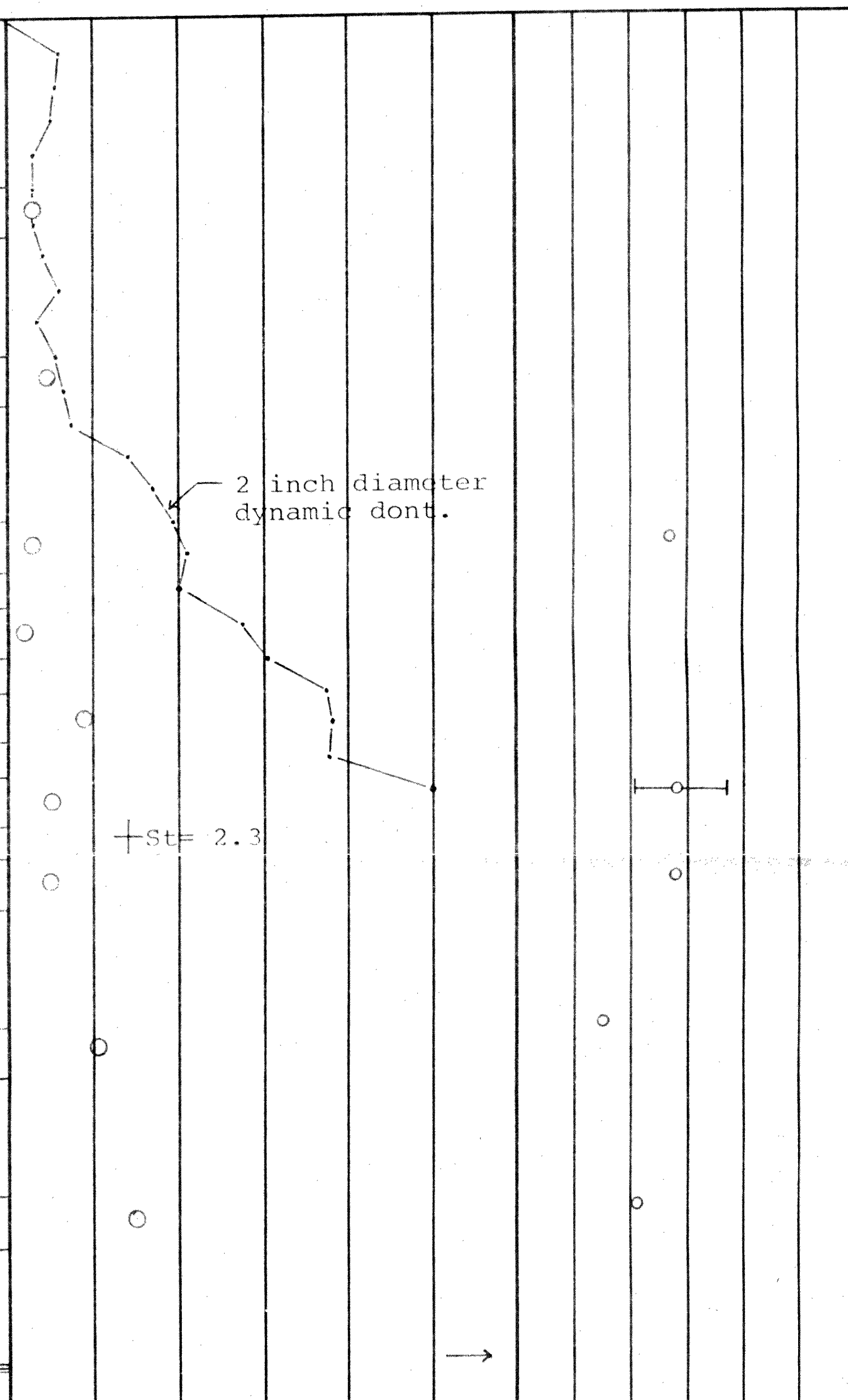
Method: Hollow-stem Auger
Diameter: 8-inch
Date: August 16, 1973.

| ELEVATION Ft. | DEPTH Ft. | DESCRIPTION | SYMBOL | GROUND WATER | NUMBER | TYPE | 'N' Blows/Foot | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 643.3 | 0.0 | Ground Surface | | | | | | |
| | | Brown silty clay & clayey silt, traces of gravel & organics. (FILL) | | | 1 | SS | 6 | Water Level Aug.16, 1973 EL. 628.3. |
| | | | | | 2 | SS | 9 | |
| | 12.0 | Soft to stiff silty clay with layers and/or pockets of sand & traces of wood, shells & gravel. brown / grey | | ▽ | 3 | SS | 6 | |
| | | | | | 4 | SS | 4 | |
| | | | | | 5 | SS | 18 | |
| | 22.0 | Stiff to very stiff grey silty clay. gravelly | | | 6 | SS | 10 | |
| | | | | | 7 | SS | 10 | |
| | | | | | 8 | SS | 21 | |
| | | | | | 9 | SS | 30 | |
| | 39.0 | Inferred Bedrock | | | | | | |
| | | Shale, Inferred Bedrock | | | 10 | SS | 50/1" | |
| | 40.1 | End of Borehole | | | | | | |

Elevation scale: 640, 635, 630, 625, 620, 615, 610, 605

PENETRATION RESISTANCE Blows / Foot: 20 40 60 80 100
UNDRAINED SHEAR STRENGTH lbs/sq.ft.: + FIELD VANE TEST ● COMPRESSION TEST: 20 40 60 80 100

WATER CONTENT %: PLASTIC LIMIT $W_p$, NATURAL $W$, LIQUID LIMIT $W_L$: 10 20 30 40 50




VERTICAL SCALE: 1 inch to 5 feet

MADE:

CHECKED:

DOMINION SOIL INVESTIGATION LIMITED

## Unified Soil Classification Plasticity Chart

ML Inorganic silts and very fine sands, silty or clayey fine sands, or clayey silts with slight plasticity.

CL Inorganic clays of low plasticity, gravelly clays, sandy clays, silty clays, lean clays.

OL Organic silts and organic silty clays of low plasticity.

MI Inorganic silts, clayey fine sands or clayey silts with medium plasticity.

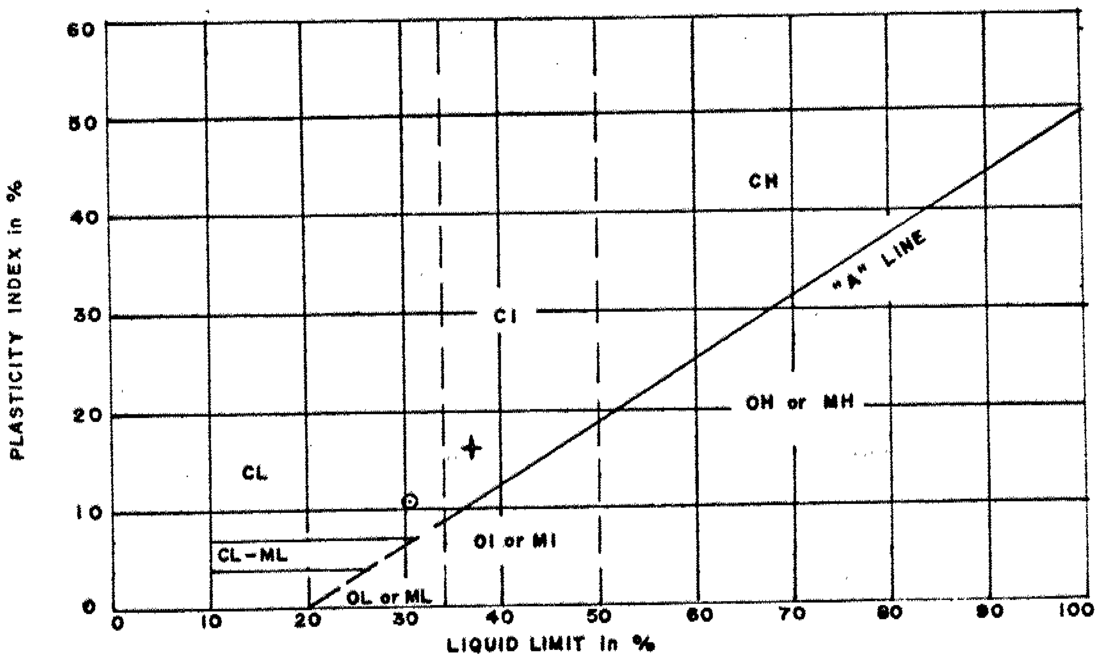
CI Inorganic clays with medium plasticity, sandy clays, silty clays.

OI Organic silts and organic silty clays of medium plasticity.

MH Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.

CH Inorganic clays of high plasticity, fat clays.

OH Organic silts and organic clays of high plasticity.




| | | |
|---|---|---|
| PROJECT: | Bear Creek Bridge, | |
| LOCATION: | Township of Enniskillen, | |
| BOREHOLE NO.: | 1 | 2 |
| SAMPLE NO.: | 7 | 6 |
| DEPTH OF SAMPLE; | 26' | 23' |
| ELEVATION OF SAMPLE: | 618' | 620' |
| LIQUID LIMIT: % | 30.4 | 36.9 |
| PLASTIC LIMIT: % | 19.8 | 21.0 |
| PLASTICITY INDEX: % | 10.6 | 15.9 |
| MOISTURE CONTENT: % | 19.0 | 28.3 |
| LIQUIDITY INDEX: | -.075 | 0.46 |
| LEGEND: | o | + |

DOMINION SOIL INVESTIGATION LIMITED
CONSULTING SOIL & FOUNDATION ENGINEERS
104 CROCKFORD BLVD., SCARBOROUGH, ONT. (416)751-6565 TELEX 02-21210 CABLES: DOMSOIL

P.O. Box 4033, Station 'C',
London, Ontario.

| | | | |
|---|---|---|---|
| Report On: | Shale Bedrock. | Ref. No: | 73-8-L4 |
| Client: | | File No: | |
| Type: | Bx core. | Clients No: | |
| | | Date: | 16 October, 1973. |
| | | Specification: | |
| Source: | | Date Received in Lab: | |
| Sampled by: | | On (date) | |

COMPRESSION:

| Lab. No. | | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|---|
| Mean area | sq ins | 2.06 | 2.06 | 2.06 | | |
| Maximum load | lb | 26,000 | 34,500 | 26,500 | | |
| Compressive strength | lb/sq inch | 12,620 | 16,750 | 12,860 | | |

AVERAGE COMPRESSIVE STRENGTH 14,080 lb/sq inch

DOMINION SOIL INVESTIGATION LIMITED,

P.H. Davies, P. Eng.

G.I.-30 SEPT. 1976

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 40J16-59

DIST. 1 REGION

W.P. No. 82-84-02

CONT. No. 85-70

W. O. No.

STR. SITE No. 14-195-97

HWY. No. 21

LOCATION Bear Creek Bridge

No of PAGES - —

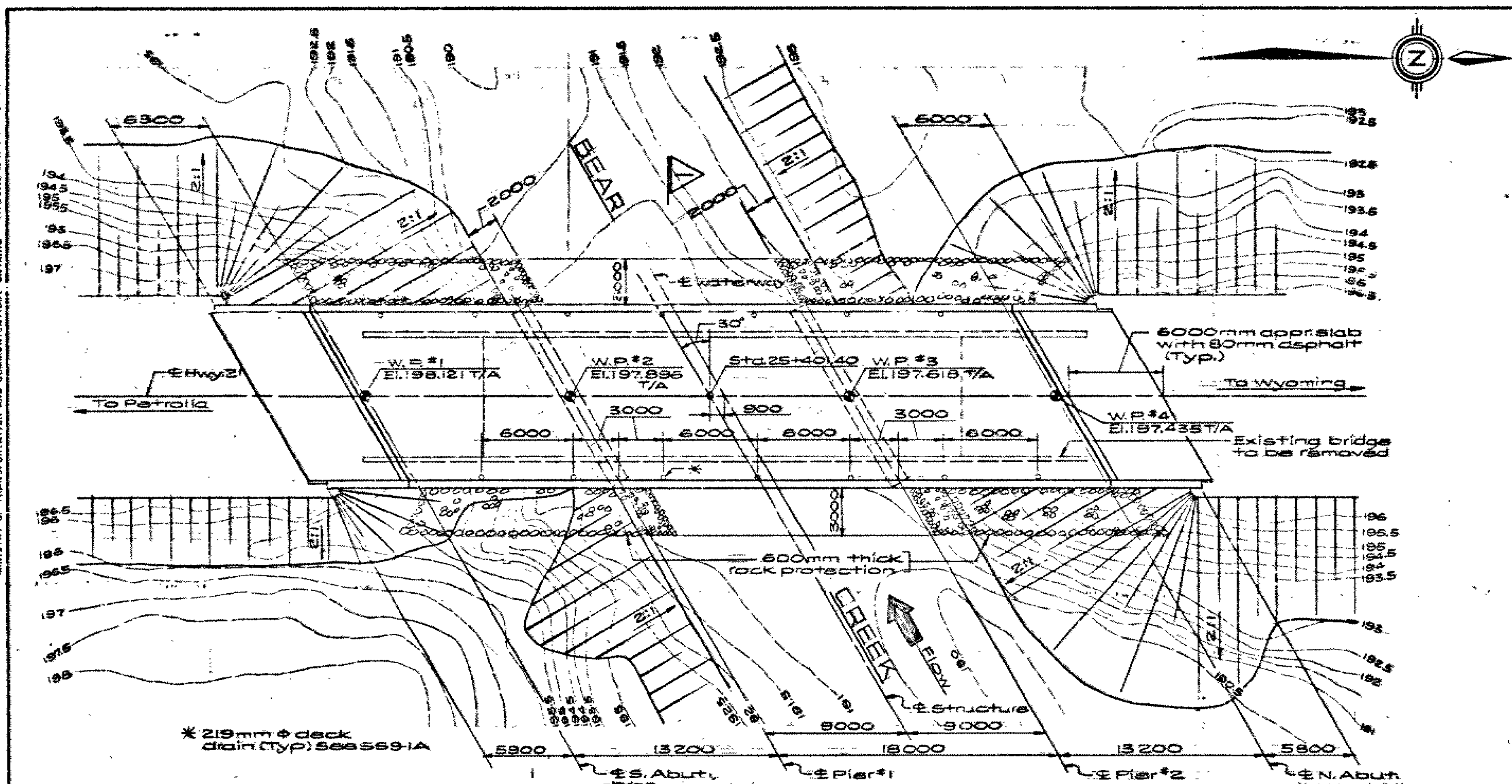
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

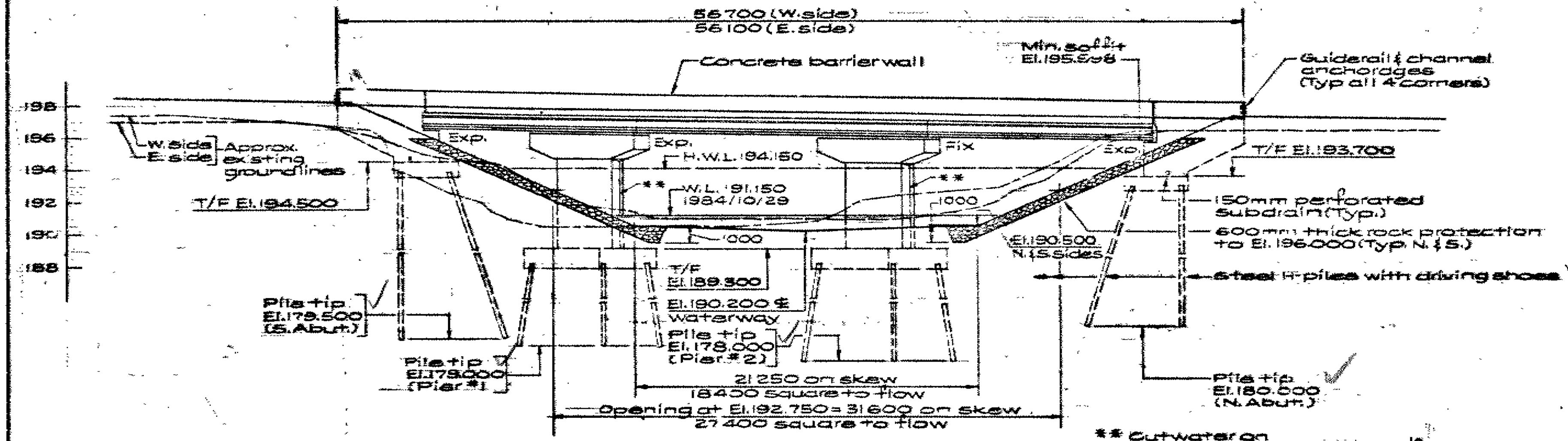
# METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

| | |
|---|---|
| DIST. 1 | |
| CONT No | |
| WP No 82-84-02 | |
| BEAR CREEK BRIDGE #3<br>1.6 Km north of Petrolia<br>GENERAL ARRANGEMENT | SHEET |

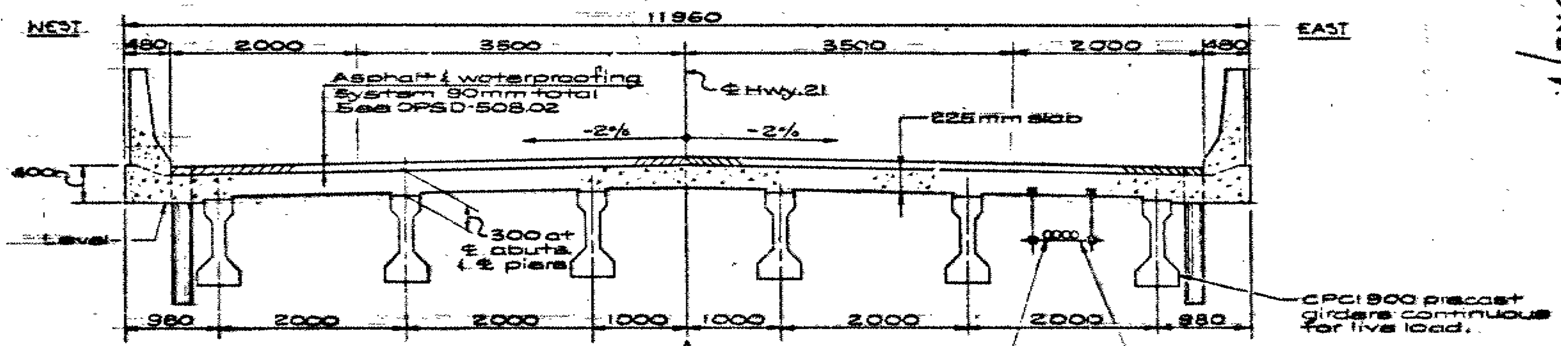



PLAN
1:200

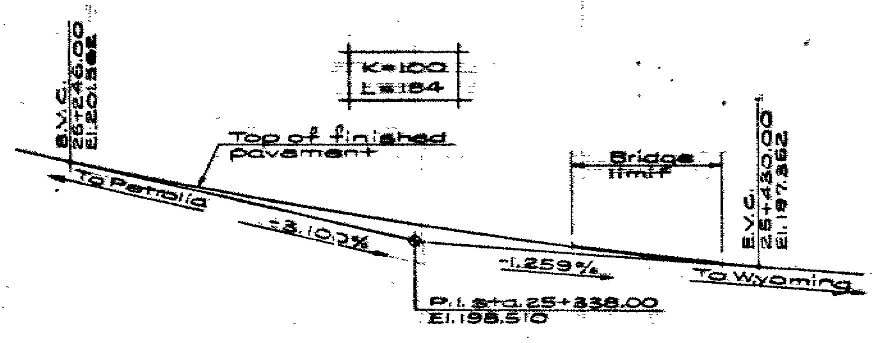



ELEVATION
1:200

B.M. 203.106
Cut cross on N.E. Corner DI 8.40 Rt. 25+179.8




1:40




PROFILE OF HWY. 21
N.T.S.

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

## NOTES

### CLASS OF CONCRETE

Footings ———— 70 MPa
Piers ———— 30 MPa
Abutments & wingwalls ———— 30 MPa
Precast girders ———— 35 MPa
Deck & Barrier walls ———— 30 MPa
And as noted.

### CLEAR COVER TO REINF. STEEL

Footings ———— 100±25
Abutments & wingwalls:
Front face ———— 80±20
Back face ———— 70±20
Piers ———— 80±20
Deck:
Top ———— 70±20
Bottom & sides ———— 40±10
Remainder
Unless otherwise noted 70±20

### REINFORCING STEEL

Reinforcing steel shall be grade 400 unless otherwise specified. Bar marks with suffix 'C' denote coated bars.

### CONSTRUCTION NOTES

The contractor shall finish the bearing seats dead level to the specified elevations to a tolerance of ±3mm.

## LIST OF DRAWINGS

14-195-97- 1 General Arrangement
" " " 2 Bore Hole Location & Soil Strata
" " " 3 Footing Layout
" " " 4 Abutment footing Reinforcing
" " " 5 South Abutment
" " " 6 North Abutment
" " " 7 Piers and Pier Footings
" " " 8 Prestressed Girders
" " " 9 Deck Layout & Screed Elevations
" " " 10 Deck Reinforcing
" " " 11 West Barrier Wall
" " 12 East Barrier Wall
" " " 13 6000mm Approach Slabs
" " " 14 As Constructed Elev. & Dim.
" " " 15 Bridge Data & Site Number Data
" " " 16 Joint Anchorage and Armouring
" " " 17 Standard Details
" " " 18 Pile Driving-Steam & Diesel Hammers
" " " 19 Quantities
" " " 20 Quantities

## LIST OF ABBREVIATIONS

W.P. = Working point
T/A = Top of asphalt
T/F Top of footing
F.F. Front face
B.F. Back face
E.F. Each face

REGISTERED PROFESSIONAL ENGINEER — CIVIL — K. Z. STOLARSKI — PROVINCE OF ONTARIO

| REVISIONS DATE | BY | DESCRIPTION | |
|---|---|---|---|
| DESIGN K.Z.S. | CHECK | LOADING OHBDC-A-83 | DATE Sept/85 |
| DRAWING D.C. | CHECK M.H.K.Z.S. | SITE No 14-195-97 | DWG 1 |

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO

# METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

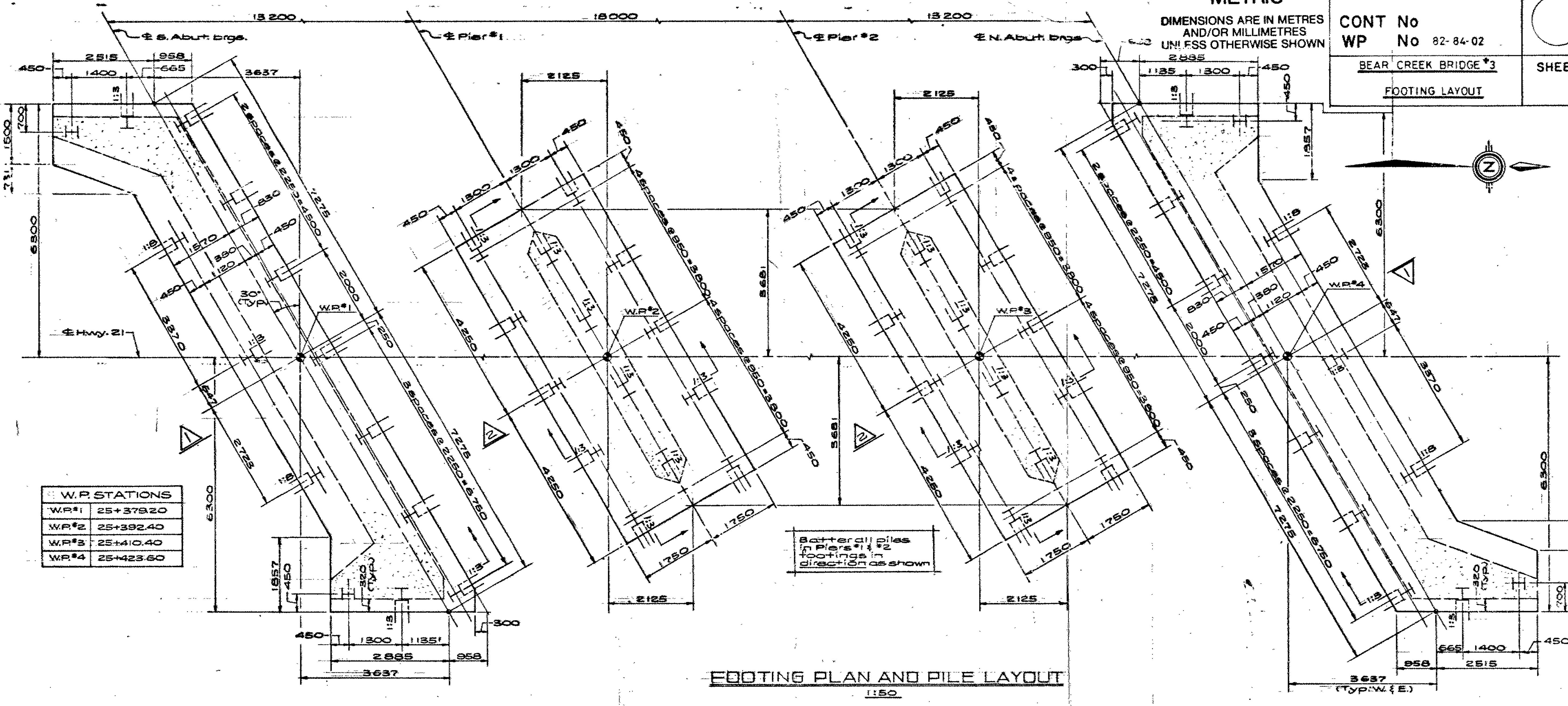
CONT No

WP No 82-84-02

BEAR CREEK BRIDGE #3

FOOTING LAYOUT

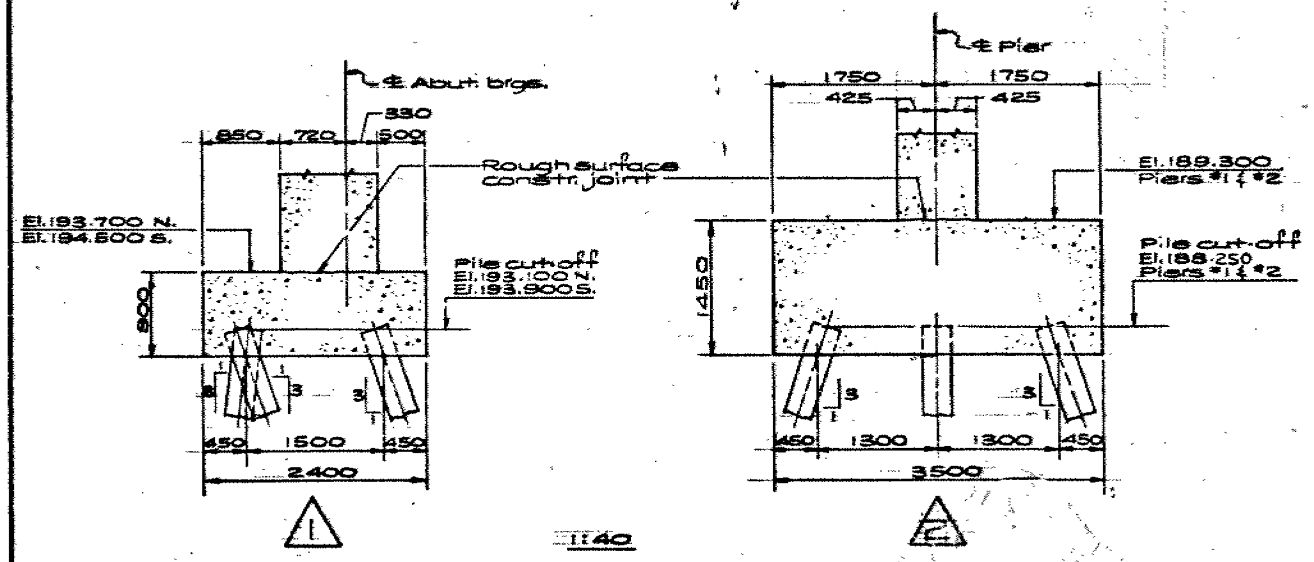
SHEET




## FOOTING PLAN AND PILE LAYOUT

1:50

| W.P. STATIONS | |
|---|---|
| W.P.#1 | 25+379.20 |
| W.P.#2 | 25+392.40 |
| W.P.#3 | 25+410.40 |
| W.P.#4 | 25+423.60 |




1:40

| PILE DATA | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Location | Row | No | Length | Batter | Location | Row | No | Length | Batter |
| S. Abut. | Front row | 7 | 15250 | 1:3 | Pier #2 | North side | 5 | 11 000 | 1:3 |
| | Back row | 2 | 14750 | 1:8 | | South side | 5 | 11 000 | 1:3 |
| | | 1 | 15250 | 1:3 | | Middle | 4 | 11 000 | 1:3 |
| | West end | 1 | 15250 | 1:3 | | | | | |
| | | 1 | 145 00 | Vert. | | | | | |
| | East end | 1 | 5250 | 1:3 | N. Abut. | Front row | 7 | 14000 | 1:3 |
| | | 1 | 145 00 | Vert. | | Back row | 2 | 13250 | 1:8 |
| | | | | | | | 1 | 14000 | 1:3 |
| Pier #1 | North side | 5 | 10000 | 1:3 | | West end | 1 | 14000 | 1:3 |
| | South side | 5 | 10000 | 1:3 | | | 1 | 13250 | Vert. |
| | Middle | 4 | 10 000 | 1:3 | | East end | 1 | 14000 | 1:3 |
| | | | | | | | 1 | 13250 | Vert. |
| | | | | | | | | | |

DRAWING NOT TO BE SCALED

100 mm ON ORIGINAL DRAWING

## NOTES

- All piles for abutments & piers to be HP310 X 79.
- Pile spacing to be measured at the underside of footings.
- Pile length shown on the drawing is the theoretical length below cut-off.
- Piles to be driven in accordance with Std. SS103-11
- Driving shoes to provided on all piles in accordance with Std. DD-3301.

## PILE DESIGN DATA

Factored capacity at U.L.S. = 1150 KN.

Capacity at SLS Type II = 825 KN.

CIVIL

K. Z. STOLAR

| REVISIONS | | |
|---|---|---|
| | | |
| | | |
| | | |
| DATE | BY | DESCRIPTION |

| | | | |
|---|---|---|---|
| DESIGN K.Z.S. | CHECK | LOADING OHBDC-A-83 | DATE Sept/85 |
| DRAWING | CHECK M.M. | SITE No 4-95-97 | DWG 3 |

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO