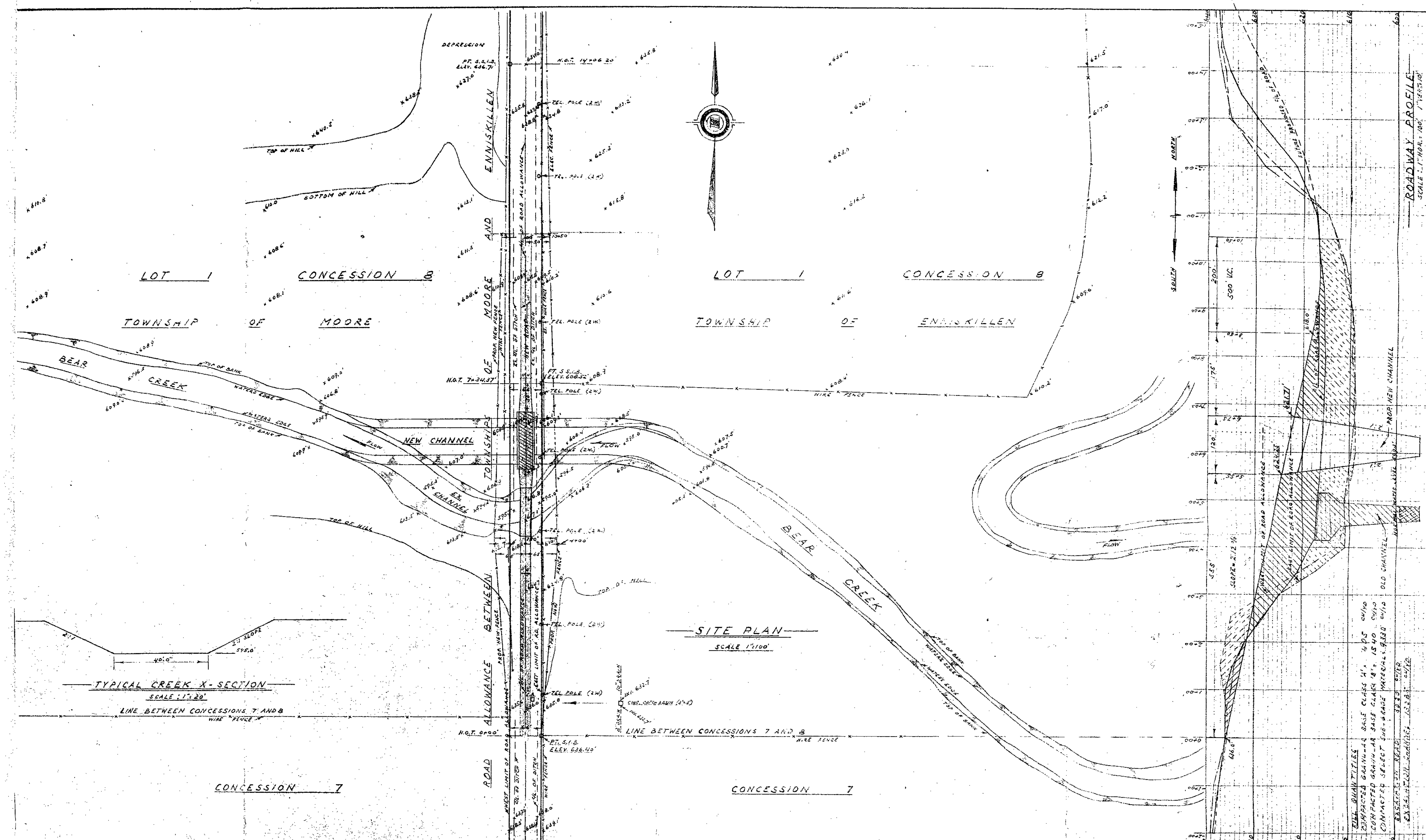
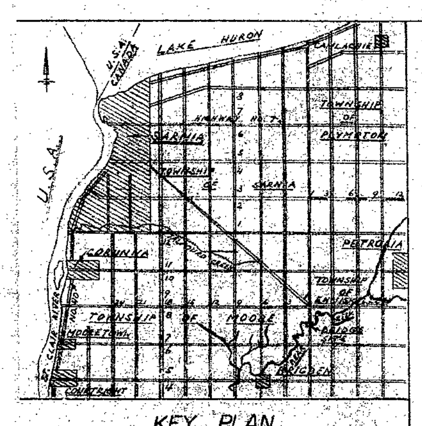
63-F-239M

MOORE-ENNISKILLEN T.L.

BEAR CREEK

LOT 1, CON. 8




FOLLOW SEPARATE INSTRUCTIONS FOR PREPARATION OF BRIDGE SITE PLAN, WHEN MAKING BRIDGE SURVEY.

## DATA

1. SPECIAL FEATURES: WATERFALLS, DAMS, EXCEPTIONAL FLOODS, ICE, DRIFTWOOD, SLIDING BANKS, ETC. DRIFTWOOD TENDS TO DAM CHANNEL 1000 FEET (PLUS) UPSTREAM OF BRIDGE SITE. ALSO DURING PERIODS OF HIGH RUNOFF THE GENERAL AREA IS FLOODED TO ELEVATION 610.0' O.D. (APPROX.)
2. (A) UPSTREAM & DOWNSTREAM BRIDGES (GIVE LOCATION, LENGTH, HEIGHT ABOVE N.H.W.L., NET CROSS-SECTIONAL AREA AT HIGH WATER & ESTIMATED AGE) D/S BRIDGE BETWEEN CON. 6 & 7, LOT 2, SPAN 120 FEET (3-40' SPANS), 3' ABOVE N.H.W.L., 800 SQ. FT., 20 YEARS OLD; U/S BRIDGES (2) BETWEEN LOT 3 & 4, CON. 8, 2-90 FOOT SPANS, 2' ABOVE N.H.W.L., 900 SQ. FT., 30 YEARS OLD.
   (B) REASONS WHY THESE BRIDGES ARE, OR ARE NOT, FAIR INDICATIONS OF 'F PROPOSED BRIDGE: D/S BRIDGE FAIR INDICATION; U/S BRIDGES LIE IN FLAT SWAMPY AREA.
3. REASONS FOR CHANGES IN HEIGHT OR LENGTH FROM THAT OF OLD BRIDGE: THE PROPOSED BRIDGE SHALL BE HIGHER IN ORDER TO PROVIDE ADEQUET ROAD APPROACHES AND ROAD SAG.

## DATA (contd.)

4. IS DITCH, STREAM, OR RIVER GRADIENT LIABLE TO BE LOWERED? IMPROBABLE
5. NAVIGATION CLEARANCES REQUIRED, IF ANY: NOT REQUIRED
6. RAILWAY CLEARANCE REQUIRED, IF ANY: NOT REQUIRED
7. IF STRUCTURE IS OVER OR UNDER A RAILWAY HAS APPROVAL BEEN OBTAINED?
   (A) FROM RAILWAY CO.: NOT REQUIRED
   (B) FROM BOARD OF TRANSPORT COMMISSIONERS: NOT REQUIRED
8. HAS APPROVAL BEEN OBTAINED UNDER NAVIGABLE WATERS PROTECTION ACT? NOT REQUIRED
9. IS A TEMPORARY DETOUR REQUIRED? YES
   WHO WILL BUILD IT? GENERAL CONTRACTOR
   WHO WILL MAINTAIN IT? GENERAL CONTRACTOR
10. INFORMATION AND EVIDENCE OF EXTREME FLOODING WAS OBTAINED FROM FIELD OBSERVATIONS AND ELEVATIONS AND REFLECTS HIGHEST WATER ELEVATION IN THE AREA OF THIS CONSTRUCTION TO BE 610.0' O.D. AND THE LOWEST WATER ELEVATION TO BE 598.0' O.D.
11. ROAD DESIGN INFORMATION.
    ESTIMATED A.D.T.: 400-1000 CARS PER DAY
    DESIGN SPEED: 60 M.P.H.
    STOPPING SIGHT DISTANCE: 475 STRAIGHT ROAD

## STRUCTURE DATA.

1. NET SPAN LENGTH AND TYPE OF BRIDGE: 3-40' SPANS OF CONC. DECK, PIERS AND ABUTS.
2. ROADWAY WIDTH ON BRIDGE: 28'
3. NUMBER & WIDTH OF SIDEWALKS: NIL
4. SKEW ANGLE: NIL
5. TOTAL LENGTH & TYPE OF PILING
6. APPROX VOLUME OF CONCRETE: CU. YDS.
7. APPROX. WEIGHT OF STR. STEEL: TONS.
8. APPROX. WEIGHT OF REINFORCEMENT: TONS.
9. APPROX VOLUME OF APPROACH FILL 100' EACH SIDE OF STRUCTURE: SEE PROFILE CU. YDS.
10. DRAINAGE AREA: 144 SQ. MI.

FIELD INVESTIGATION MADE: APRIL 1963

BY: J. D. NISBET
SURVEY ENGINEER.

JAMES D. NISBET
CONSULTING ENGINEERS
SARNIA ONTARIO

# PROPOSED BRIDGE OVER BEAR CREEK

OWNER: CO. OF LAMBTON
MUNICIPAL DIST. No. 1
Co. OF LAMBTON
ROAD No. TOWN LINE
TWP. OF MOORE / OF ENNISKILLEN
LOT 1
CON. 8 / 8

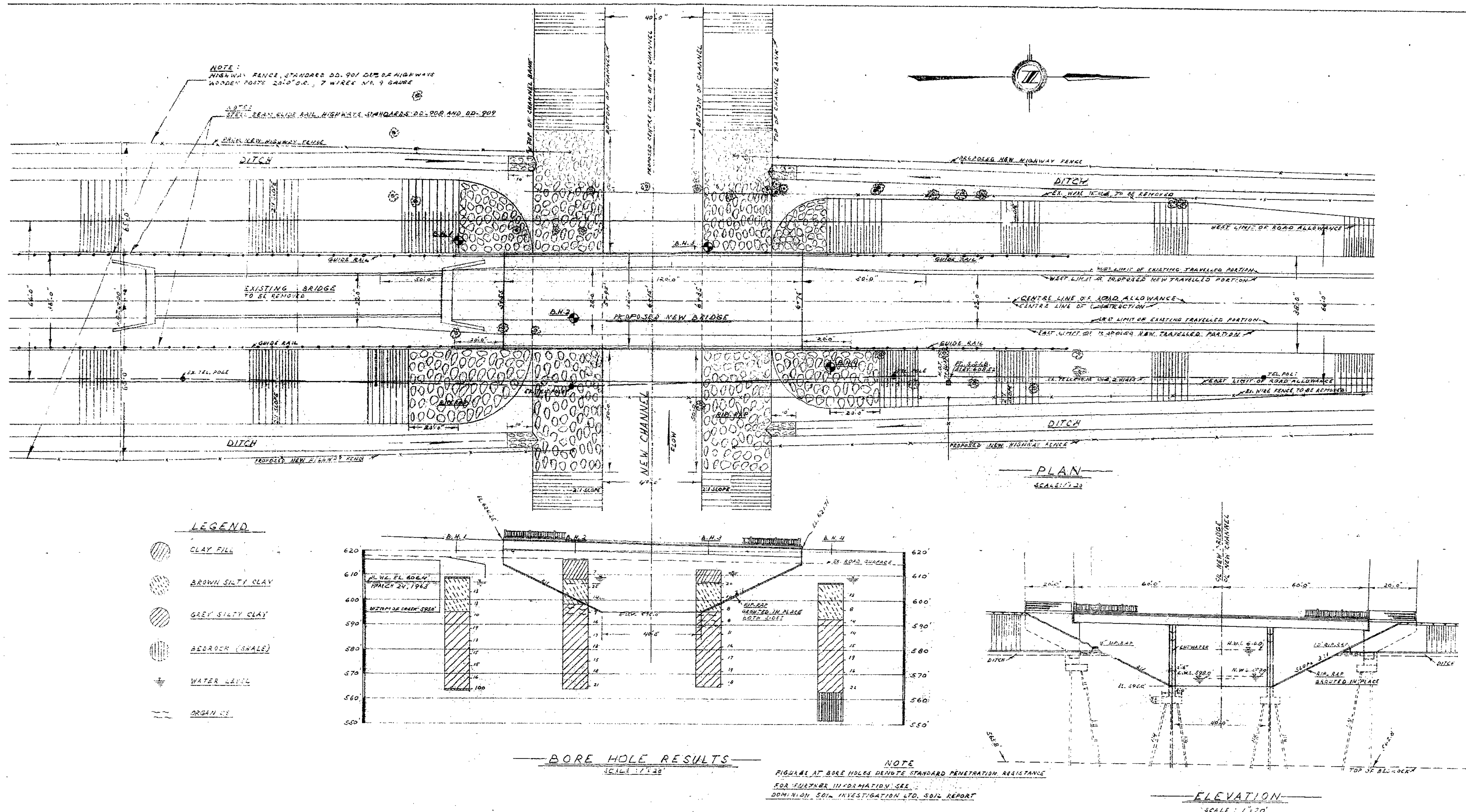
# SITE PLAN.

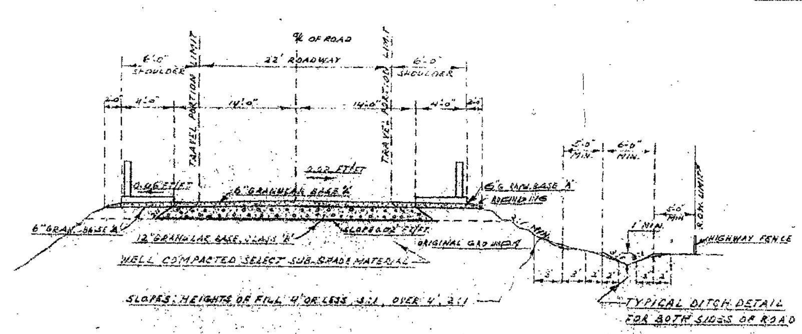
CIVIL
JAMES D. NISBET

| DATE | DESIGN ENGINEER. |
|---|---|
| DECEMBER 1963 | JAMES D. NISBET |

BRIDGE NAME: NONE

| LOADING. | BRIDGE No. | DWG. No. SH. NO. 1 |
|---|---|---|
| H 20-S 16 | TWP-1 | E-63-20 |




'A-A' TYPICAL ROADWAY X-SECTION BETWEEN CHAINAGE 9+00 AND 10+50

JAMES D. NISBET
CONSULTING ENGINEERS
SARNIA ONTARIO

PROPOSED BRIDGE OVER BEAR CREEK

OWNER: CO. OF LAMBTON — MUNICIPAL DIST. No. 1

CO. OF LAMBTON — ROAD No. TOWN LINE
TWP. OF MOORE LOT 1 — CON. B
TWP. OF ENNISKILLEN LOT 1 — CON. B

GENERAL LAYOUT

REGISTERED PROFESSIONAL ENGINEER — CIVIL — JAMES D. NISBET

DECEMBER 1963 — DATE
JAMES D. NISBET — DESIGN ENGINEER

BRIDGE NAME: NONE

| LOADING | BRIDGE NO. | DWG. NO. |
|---|---|---|
| H20-S16 | TWP-1 | E-63-20 |

BA 1758

MR. JAMES D. NISBET
CONSULTING ENGINEER
206 Water Street
SARNIA ONTARIO

Report on

SOIL INVESTIGATION

for

BEAR CREEK BRIDGE, TWP 1,

MOORE - ENNISKILLEN TOWNSHIP LINE

COUNTY OF LAMBTON

by

DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO

Reference No. 3-3-L4
March 1963

# CONTENTS



# ENCLOSURES

## SUMMARY

The strata are stiff cohesive glacial deposits, extending to sound bedrock at El. 563 feet, or approximately 50 feet below the present road surface.

In the prevailing soil conditions, the structure can either be supported on spread footings or on piles. Both types of foundation are technically satisfactory, and the choice will be governed by cost, time, availability of materials and other factors.

For spread footings a maximum design pressure of 3000 p.s.f. is recommended for footings with widths of 10 to 15 feet. The estimated consolidation settlement under a 10-foot wide footing is 1.4 inches.

For a piled structure, concrete-filled pipe piles, open-ended pipes, precast concrete piles or H-piles are all satisfactory. The maximum working load will depend on the material chosen, and the settlement of the structure will arise mainly from the elastic deflection of the piles.

No unusual construction problems are anticipated.

## I INTRODUCTION

Verbal authorization was received from Mr. R. Letham to carry out a soil investigation at the site of a proposed new bridge on the road allowance between the townships of Moore and Enniskillen. A visit was made to the site by Mr. Letham and the writer on the 12th of March 1963, and the requirements of the investigation were discussed.

The new bridge is intended to be a 3-span structure with an overall length of 120 to 150 feet. Its transverse centre-line is assumed to be 75 to 80 feet to the north of the north abutment of an existing single-span structure which presently carries the road across Bear Creek. The existing bridge will be demolished and the creek will be realigned to the new position. The total design loads for abutments and piers will be approximately 750 and 900 kips respectively for a 35-foot wide, freely-supported structure (assuming a spread footing design.).

The purpose of this investigation was to reveal the subsurface conditions and to determine the necessary soil properties for the design and construction of foundations.

## II PHYSIOGRAPHY

The site lies centrally in the Lambton Clay Plain which is part of the extensive physiographic region known as the St. Clair Clay Plains. In this part of the region the clay soil is generally classified as a "till" in that it is commonly a stiff, over-consolidated, glacial deposit. It is sometimes covered by thin deposits of normally-consolidated lacustrine clay.

The area is one of low surface relief. The creek is a branch of the Sydenham River which flows southward with a gentle gradient to join the main stream at Wallaceburg. The shallow valley in which the site is located is not classed as a spill-way by Chapman and Putnam, but nevertheless has apparently been occupied by a much larger stream. It is many times the width of the present stream, and traces of sandy and organic material were found in a seam some distance below the present ground surface.

## III FIELD WORK

Field work was carried out during the period 21st to 24th March 1963, and consisted of 4 boreholes at the locations shown on enclosure 2. One dynamic cone penetration test was performed adjacent to borehole 2, the first hole to be made (The holes were bored in the order 2 - 3 - 1 - 4.). The water in the creek rose rapidly during the night of 24th March and it became necessary to abandon this hole before the bedrock could be explored to a sufficient depth. The drill returned to this position approximately 10 days later, and further core samples were recovered.

The field tests consisted of Standard Penetration tests using a 2-inch O.D. split spoon, and insitu vane shear tests using a 2-inch diameter 4-bladed vane. Undisturbed samples were recovered in 2-inch diameter thin-walled tubes for examination in the laboratory.

The results of the field tests are shown on geotechnical data sheets comprising enclosures 3 to 6. Elevations have been referred to a local datum, viz. "top of northwest corner of northwest wingwall of existing bridge" which is assumed to have the geodetic elevation 615.90 feet.

## IV SUBSURFACE CONDITIONS

Details of the stratification at each borehole are shown on the data sheets, and a general picture of the subsurface conditions is given by the profile on enclosure 2.

Boreholes 2 and 3 both passed through some depth of embankment fill which consists of the local brown silty clay. This latter material, and the grey clay stratum below it, are both glacial deposits or "tills". They are dense, over-consolidated, and of stiff to very stiff consistency. They contain traces of fine granular particles.

A seam of material at the top of the grey clay layer in boreholes 2 and 3 was found to contain traces of sand and organic matter. This apparently marks an earlier surface elevation or, more probably, the bed of a previous water-course.

The bedrock is a brownish-black, horizontally laminated shale. In 11 feet of drilling at borehole 4, the recovery of core was better than 90%, indicating a sourd, fairly hard material. There is a difference of 1 foot in the elevations at which the bedrock was encountered at boreholes 1 and 4. Boreholes 2 and 3 did not reach the rock surface (whose proximity was not known at the time they were drilled), but apparently were terminated within 1 to 2 feet of it. It can reasonably be assumed that the rock surface is almost flat at El. 563±.

The water level in the creek varied by several feet during the period of the field work, and by flooding some of the borehole locations, masked the true equilibrium groundwater conditions. From such observations as were possible, it is deduced that the normal water table probably fluctuates between Els. 602 and 607 feet.

## V LABORATORY TESTS

A series of laboratory tests has been performed on samples of the grey silty clay stratum in which footings will bear if such a design is used. This enabled a much closer examination to be made of the physical properties of the soil than is possible in the field.

*Atterberg limits* were determined for 4 samples as a means of classification and as a guide to the probable behaviour of the soil.

One *consolidation test* was performed to determine the compressibility of the soil and enabling settlement calculations to be made. The test results are shown on enclosure 7.

*Unconfined compression tests* were performed on 4 samples in an attempt to confirm the field vane shear strength values. This test yielded very scattered results of 420, 863, 1190 and 1420 p.s.f., compared to an average value of 1980 p.s.f. for the vane results between Els. 590 and 580. The sample with the lowest value of 420 p.s.f. showed signs of slight disturbance and the other physical properties (void ratio, bulk density) are quite inconsistent with the low strength value. This value is therefore rejected. The remaining values from this test have an average of 1157 p.s.f. and the discrepancy between this and the average vane value of 1980 is believed to be the result of vertical fissures in the soil. These are not uncommon in stiff clays and are caused by dessication at some time in the earlier history of the soil deposit. When the confining pressures of the soil mass are removed, the fissures (which may be too small to be seen by the naked eye) can open, and thereby give a misleadingly low result in an unconfined compression test.

To examine this hypothesis further, *triaxial compression tests* were performed on undrained unconsolidated samples. The tests were performed at confining pressures of 15 p.s.i. (the natural overburden pressure), 20 p.s.i. and 25 p.s.i. and the results are shown on enclosure 8. The material has a small friction angle as shown by the increase in shear strength with confining pressure. For the purpose of bearing capacity calculations it will be assumed that the clay is a frictionless material with a shear strength of *1600 p.s.f.*

The test series, together with the field results, have shown the material to be a stiff, lean, overconsolidated, silty clay of intermediate plasticity. It has a sensitivity of approximately 2 and the liquidity index is 0.3 to 0.5.

A summary of the laboratory test results is given on enclosure 9.

## VI FOUNDATIONS

The present level of the bed of the creek is at approximately El. 596 suggesting a footing level of 589 to allow for scour. At this elevation the clay is stiff and can provide adequate support for a spread footing design. Also, sound bedrock was encountered at elevation 563 (±), only 26 feet below the proposed footing level, so that the use of relatively short end-bearing piles is also worth consideration.

### (a) Footings

Assuming the shear strength of the soil to be 1600 p.s.f., the ultimate bearing capacity of a footing 10 feet x 35 feet is calculated by Meyerhof's theory to be 8800 p.s.f. A design pressure of 3000 p.s.f. would thus give a factor of safety of 2.9. *The value 3000 p.s.f. is recommended as the maximum gross design pressure for footings of 10 to 15 feet in width.*

The long-term consolidation settlement which will occur below a 10 foot x 35 foot footing carrying a dead load of 2500 p.s.f. is estimated to be 1.4 inches. The calculated time periods for 50% and 90% of this consolidation to occur are one year and 7 years respectively. The assumed loading condition is probably more severe than will be realized, and especially in view of the long time periods, such a settlement should be tolerable in a freely-supported structure. In the very uniform soil conditions the differential settlement between supports is not expected to exceed 0.5 inch.

(b) Piles

The use of driven end-bearing piles may have attractive advantages in saving time and cost. The length of piles can be predicted quite accurately and will be practically constant across the site. Also, because the deflection of a piled structure under load in these conditions will be negligible, a continuous beam design could be employed.

Timber piles are not the most suitable because the stiff clay will necessitate hard driving and the piles may become damaged.

Concrete-filled pipe piles, open-ended pipes, or precast concrete piles are all expected to set between elevations 561 and 563. H-piles may go as far as El. 559. The working loads are limited only by the permissible working stresses in the materials, although for working loads of more than 60 tons per pile, the shells and driving equipment may be less easil. available.

Pipe piles could be used to act as columns at the pier supports, thus considerably simplifying the design. H-sections would be easier to drive because they cause less displacement, although details of their incorporation into the structure would be more complex.

The choice of type of pile, or the decision whether to use piles at all, will be largely influenced by factors such as cost, time, design arrangement and the availability of materials and equipment, which are beyond the scope of this project. Both spread-footing and pile designs will be satisfactory in the prevailing conditions. The stiff impervious soil will permit excavations to be made without special bracing or dewatering procedures. Seepage should be accumulated in sumps dug in the bottom of the excavation. In the case of a footing design it is recommended that the grade should be covered with a concrete blanket to prevent disturbance as soon as it has been inspected and approved. Disturbed material at the grade level should be replaced with lean concrete.

## VII REFERENCES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2. Procedures for Testing Soils, ASTM, April 1958, pp. 186 to 198 (Unified Soil Classification System, by A.A. Wagner).

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4. The Ultimate Bearing Capacity of Foundations, by G.G. Meyerhof, Geotechnique, Vol. II, 1950 and 1951.

5. A Contribution to the Settlement Analysis of Foundations on Clay, by A.W. Skempton and L. Bjerrum, Geotechnique VII (1957) and Amendment Thereto by A.M. Muir Wood (Correspondence Geotechnique, Vol. IX).

6. The Measurement of Soil Properties in the Triaxial Test, by Bishop and Henkel, London, 1957.


DOMINION SOIL INVESTIGATION LIMITED

James Park

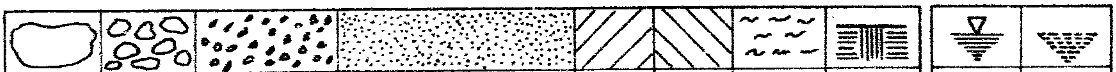
James Park, M.Sc., P.Eng.

CIVIL
J. PARK
PROVINCE OF ONTARIO

Encl.
JP/mc

# 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 | RC | Rock core | TP | Piston, thin walled tube sample |
| CS | Sample from casing | % | Recovery | TW | Open, thin walled tube sample |
| ChS | Chunk sample | SS | Split spoon 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 | $\gamma$ | Natural bulk density (unit weight) | k | Coeff. of permeability |
| LL % | Liquid limit | e | Void ratio | C | Shear strength — in terms of total stress |
| PL % | Plastic limit | RD | Relative density | $\phi$ | Angle of int. friction — in terms of total stress |
| PI % | Plasticity index | $C_v$ | Coeff. of consolidation | C' | Cohesion — in terms of effective stress |
| LI | Liquidity index | $m_v$ | Coeff. of volume compressibility | $\phi'$ | Angle of int. friction — in terms of effective stress |

## UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL / UNCONFINED COMPRESSION TEST; LABORATORY / FIELD VANE TEST; POCKET PENETROMETER TEST

St St St 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.

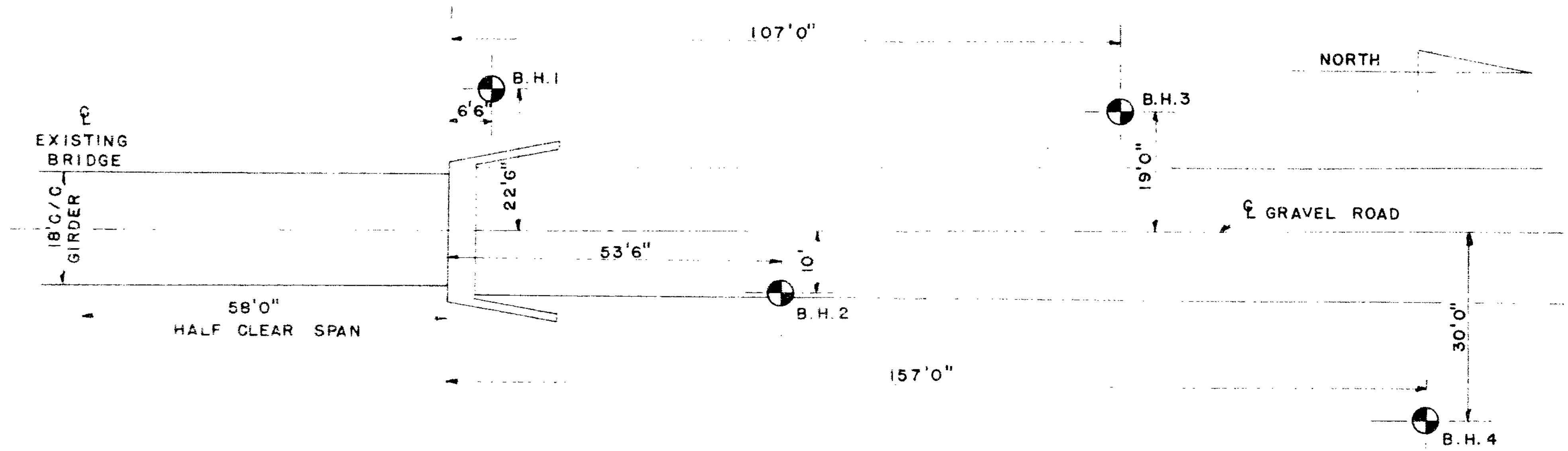
| COHESIONLESS SOILS : | RD : |
|---|---|
| Very loose | 0 – 15 % |
| Loose | 15 – 35 % |
| Compact | 35 – 65 % |
| Dense | 65 – 85 % |
| Very dense | 85 – 100 % |

| 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 |

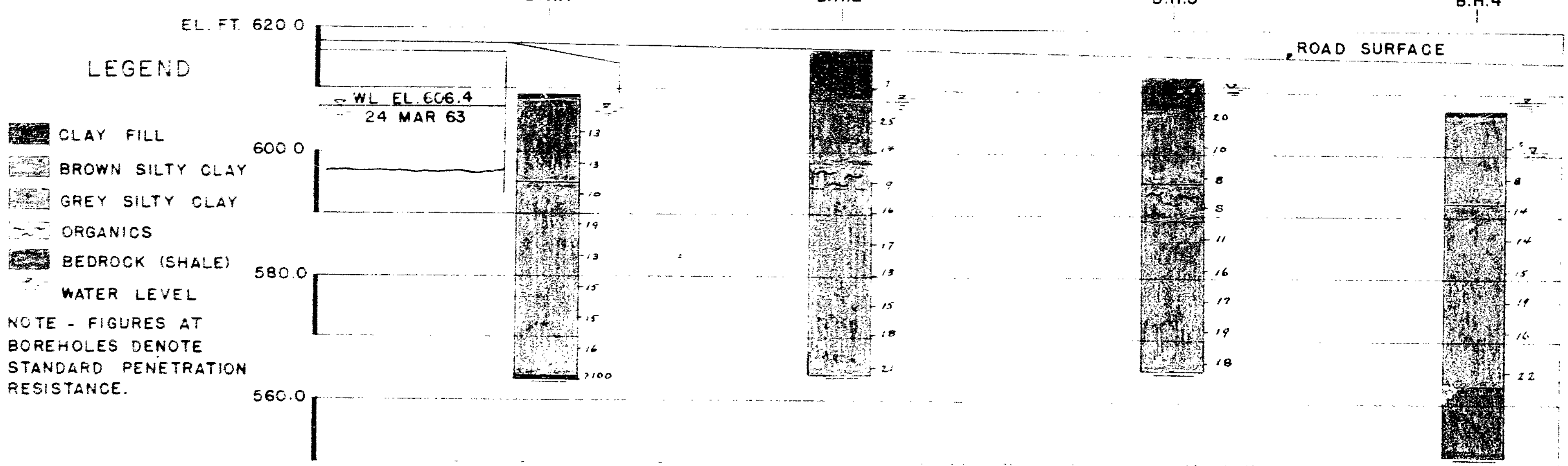
DOMINION SOIL INVESTIGATION LIMITED

JOB NO 3-3-L4
PREP. BY M.C.

ENCLOSURE 2




LOCATION OF BOREHOLES
SCALE - 1 INCH TO 20 FEET




SUBSURFACE PROFILE
SCALE - 1 INCH TO 20 FEET

DOMINION SOIL INVESTIGATION LIMITED

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 1

JOB REFERENCE NO. 3-3-L4

CLIENT Mr. J.D. Nisbet
PROJECT Bear Creek Bridge TWP 1
LOCATION Moore - Enniskillen Twp. Line
DATUM ELEVATION Geodetic

METHOD OF BORING Washboring
DIAMETER OF BOREHOLE Bx (3-inch)
DATE 23 Mar 63

ENCLOSURE NO. 3

Penetration resistance (blows/ft): 20 40 60 80 100
Shear strength (lbs/sq ft): 1000 2000 3000 4000 5000
Consistency (water content %): PL W LL — 10 20 30 40

| Elevation ft | Depth ft | Stratification Description | Sample Number | Type | Blows/ft | Remarks |
|---|---|---|---|---|---|---|
| 608.8 | | Ground surface | | | | |
| | 0 | Topsoil | | | | WL El. 606.8 24 Mar 63 |
| | 5 | Stiff brown silty clay | 1 | SS | 13 | |
| | 10 | | 2 | SS | 13 | |
| 594.8 | 15 | Grey silty clay (stiff to very stiff) | 3 | SS | 10 | |
| | | | | vane | | St=2.2 |
| | | | 4 | TW | p | |
| | 20 | | 5 | SS | 19 | |
| | | | | vane | | St=3.0 |
| | | | 6 | TW | p | |
| | 25 | | 7 | SS | 13 | |
| | | | | vane | | St=2.5 |
| | 30 | | 8 | TW | p | * |
| | | | 9 | SS | 15 | |
| | | | | vane | | St=1.7 |
| | 35 | | 10 | TW | p | |
| | | | 11 | SS | 15 | |
| | | | | vane | | St=2.0 |
| | 40 | | 12 | SS | 16 | |
| | | | | vane | | St=1.7 |
| 563.8 | 45 | Soft brown shale | 13 | SS | 176 | |
| 563.3 | | End of borehole | | | | |

*Triaxial test sample

Details of Extrapolated N-value

Sa.#13 88/6"

VERTICAL SCALE: 1 IN. TO 5 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE MC CH'D: JP

# GEOTECHNICAL DATA SHEET FOR BOREHOLE ..2..

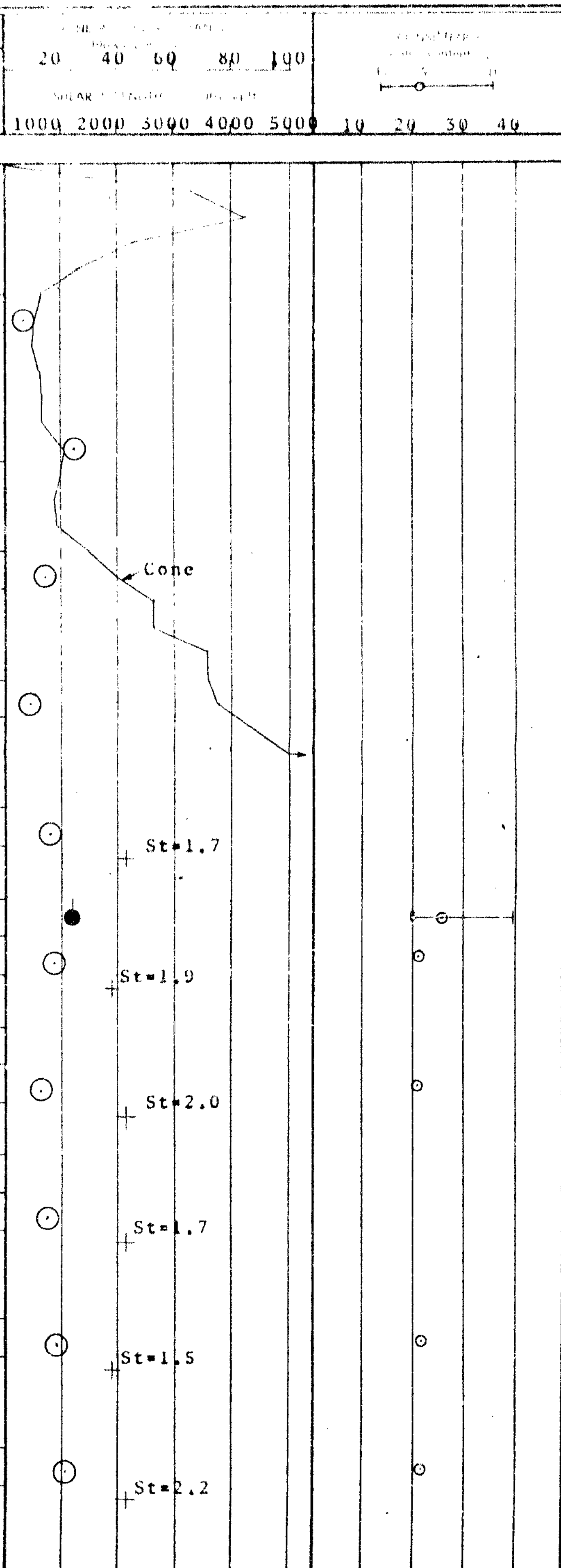
[illegible] 3-5-14

[illegible] Mr. J.D. Nisbet
[illegible] Bear Creek Bridge TSP 1
[illegible] Moore - Enniskillen Twp. Line
[illegible] Geodetic

[illegible] Washboring
[illegible] Bx (3-inch)
[illegible] 20/21 Mar 63

[illegible] 4

| Elevation | Depth | Description | Sample No. | Type | Blows | Remarks |
|---|---|---|---|---|---|---|
| 615.9 | 0 | Ground surface | | | | |
| | | Clay embankment | | | | |
| | 5 | | 1 | SS | 7 | WL El. 608.9 24 Mar 63 |
| 607.9 | | | | | | |
| | 10 | Stiff brown silty clay | 2 | SS | 25 | |
| | 15 | | 3 | SS | 14 | |
| 597.9 | | | | | | |
| | 20 | sandy, trace of organics — firm | 4 | SS | 9 | |
| 593.9 | | very stiff | | | | |
| | 25 | Grey silty clay | 5 | SS | 16 | |
| | | | | vane | | |
| | | | 6 | TW | p | |
| | 30 | | 7 | SS | 17 | |
| | | | | vane | | |
| | | | 8 | TW | p | |
| | 35 | | 9 | SS | 13 | |
| | | | | vane | | |
| | | | 10 | TW | p | |
| | 40 | | 11 | SS | 15 | |
| | | | | vane | | |
| | 45 | | 12 | SS | 18 | |
| | | | | vane | | |
| | 50 | | 13 | SS | 21 | |
| | | | | vane | | |
| 563.4 | | End of borehole | | | | |




VERTICAL SCALE: 1 IN. TO 5 FT. **DOMINION SOIL INVESTIGATION LIMITED** MADE: MC CH'D: JP

# GEOTECHNICAL DATA SHEET FOR BOREHOLE . .5 . . .

OUR REFERENCE No. 3-3-L4

CLIENT Mr. J.D. Nisbet
PROJECT Bear Creek Bridge TWP 1
LOCATION Moore - Enniskillen Twp. Line
DATUM ELEVATION Geodetic

METHOD OF BORING Washboring
DIAMETER OF BOREHOLE Bx (3-inch)
DATE 22 Mar 63

ENCLOSURE No. 5

| ELEVATION ft | DEPTH ft | STRATIGRAPHIC DESCRIPTION | SAMPLE No. | TYPE | BLOWS/FT | PENETRATION RESISTANCE / SHEAR STRENGTH | CONSISTENCY (Water content %) | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 612.2 | 0 | Ground surface | | | | | | WL El.610.9 24 Mar 63 |
| | | Clay embankment | | | | | | |
| 607.2 | 5 | Brown silty clay (stiff to very stiff) | 1 | SS | 20 | | | |
| | 10 | | 2 | SS | 10 | | | |
| 598.2 | | sandy | | | | | | |
| | 15 | | 3 | SS | 8 | | | |
| 595.2 | | Stiff grey clay thin sand seams wood fragments | | | | | | |
| | 20 | | 4 | SS | 8 | | | |
| 589.2 | | Very stiff grey silty clay | | | | | | |
| | 25 | | 5 | SS | 11 | | | |
| | | | | vane | | St=2.3 | | |
| | | | 6 | TW | p ** | | | **Consolidation test sample |
| | 30 | | 7 | SS | 16 | | | |
| | | | | vane | | St=1.9 | | |
| | | | 8 | TW | p * | | | * Triaxial test sample |
| | 35 | | 9 | SS | 17 | | | |
| | | | | vane | | St=2.0 | | |
| | | | 10 | TW | p | | | |
| | 40 | | 11 | SS | 19 | | | |
| | | | | vane | | St=1.9 | | |
| | 45 | | 12 | SS | 18 | | | |
| | | | | vane | | St=2.0 | | |
| 564.7 | | End of borehole | | | | | | |
| | 50 | | | | | | | |

Penetration resistance scale: 20 40 60 80 100; Shear strength scale: 1000 2000 3000 4000 5000; Consistency scale: 10 20 30 40 (PL, W, LL)

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: MC CH'D: JP

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 4

3-3-14

Mr. J.D. Nisbet
Bear Creek Bridge TWP 1
Moore - Enniskillen Twp. Line
Geodetic

Washboring
Bx (3-inch)
24 Mar 65 and
4 and 5 Apr 65

6

| Elevation | Depth | Stratification Description | Sample No. | Type | Blows | Remarks |
|---|---|---|---|---|---|---|
| 607.1 | 0 | Ground surface | | | | |
| | | Topsoil | | | | WL 2 feet above hole - 25 Mar 65 |
| | 5 | Brown silty clay (stiff to very stiff) | 1 | SS | 13 | WL El. 600.8 5 Apr 63 |
| | 10 | | 2 | SS | 8 | |
| 592.1 | 15 | Very stiff grey silty clay | 3 | SS | 14 | |
| | | | | vane | | St=2.0 |
| | | | 4 | TW | p | |
| | 20 | | 5 | SS | 14 | |
| | | | | vane | | St=2.0 |
| | 25 | | 6 | TW | p | |
| | | | 7 | SS | 15 | |
| | | | | vane | | St=2.0 |
| | 30 | | 8 | TW | p | |
| | | | 9 | SS | 19 | |
| | | | | vane | | St=1.8 |
| | 35 | | 10 | SS | 16 | |
| | | | | vane | | St=2.0 |
| | 40 | | 11 | TW | p | |
| | | | 12 | SS | 22 | |
| | | | | vane | | St=1.8 |
| 562.6 | 45 | Brown shale | 13 | RC Bxt | 100 % | |
| | 50 | | 14 | RC Bxt | 75 % * | * 6 inches of Sa.#14 lost in the hole, recovered with Sa.#15. |
| | | | 15 | RC Bxt | 110 % * | |
| 551.6 | 55 | End of borehole | | | | |

Penetration resistance: 20 40 60 80 100

Shear strength (lbs/sq ft): 1000 2000 3000 4000 5000

Consistency (water content %): PL W LL — 10 20 30 40

REMARKS

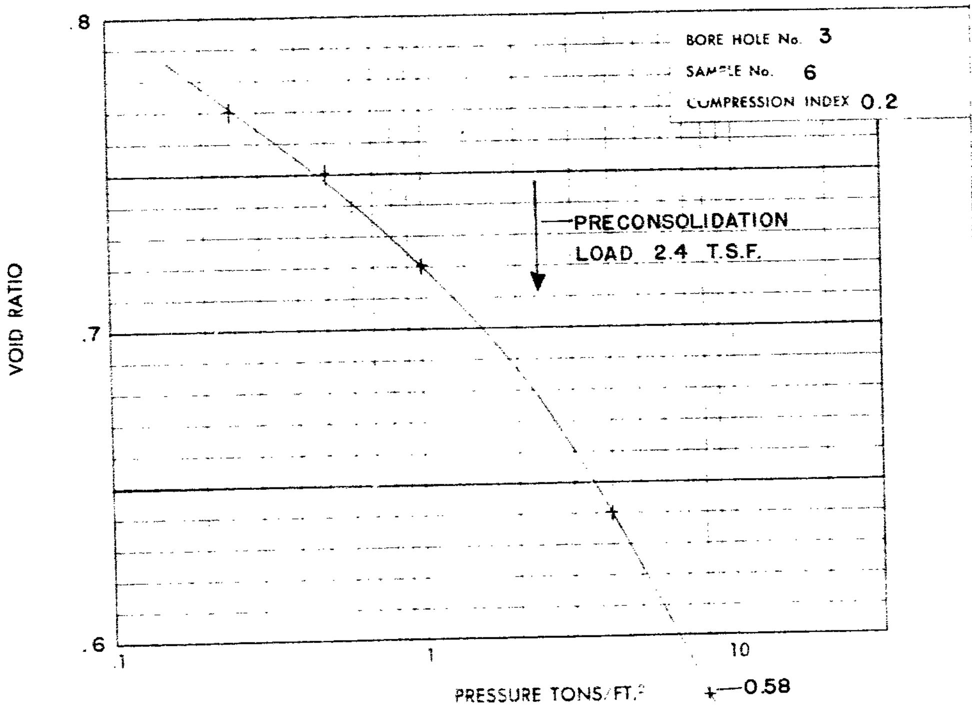
VERTICAL SCALE: 1 IN TO 5 FT.
Scale changes at 50 feet

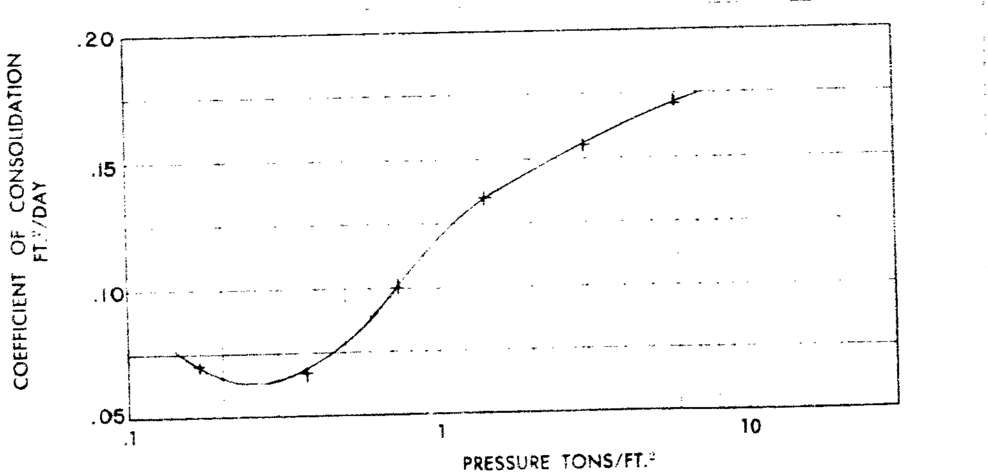
**DOMINION SOIL INVESTIGATION LIMITED**

MADE: MC CH'D: JP

# Dominion Soil Investigation Ltd.

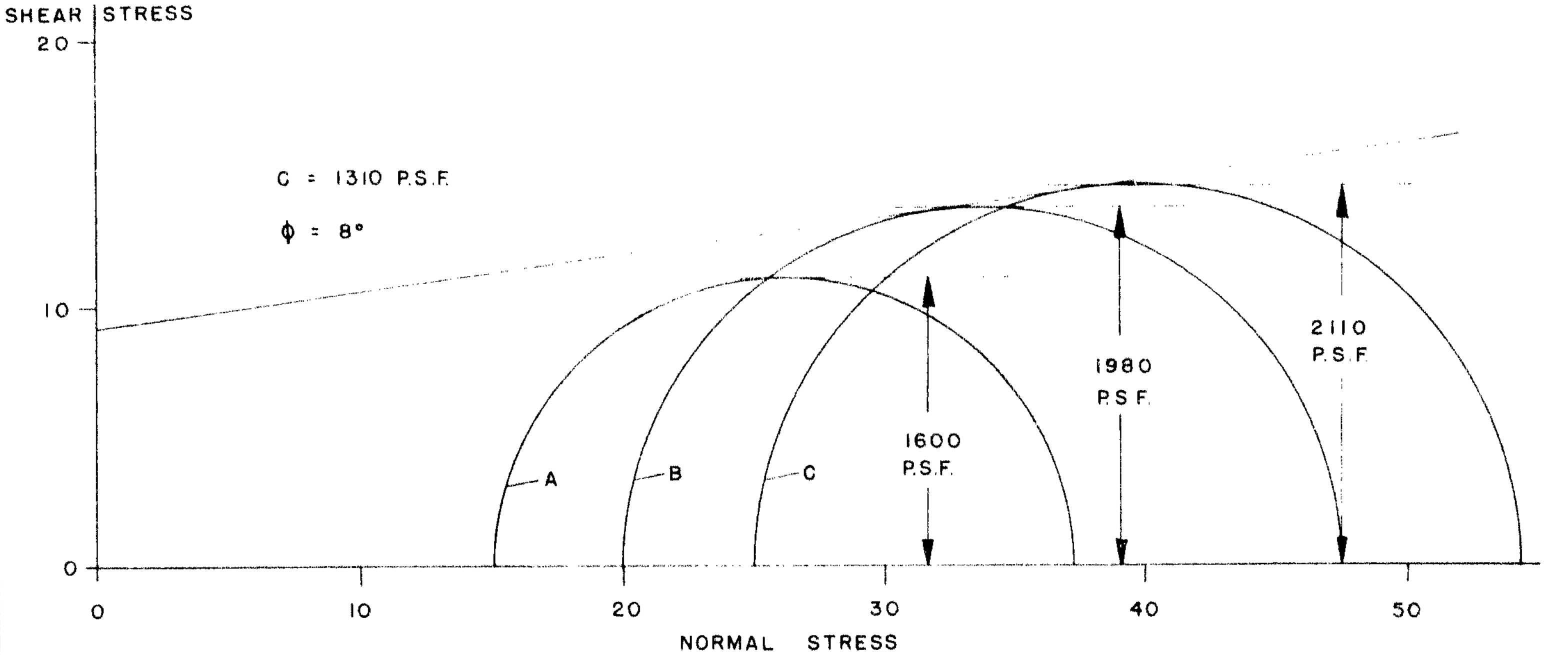
## CONSOLIDATION TEST



JOB NO. 3-3-L4
PREP BY M.C.

ENCLOSURE 8




# UNDRAINED TRIAXIAL TEST RESULTS

MATERIAL: GREY FISSURED SILTY CLAY

LL = 40% PL = 20% PI = 20%

LI = 0.3 TO 0.5

TEST DATA:

| | | | |
|---|---|---|---|
| CIRCLE DESIGNATION | A | B | C |
| BOREHOLE NO./SAMPLE NO. | 1/8 | 3/8 | 3/8 |
| SPECIMEN LENGTH X DIA. | 3" X 1.5" | 3" X 1.5" | 3" X 1.5" |
| UNIT WEIGHT P.C.F. | 122 | 123 | 124 |
| WATER CONTENT % | 28 | 27 | 23 |
| VOID RATIO | 0.79 | 0.73 | 0.68 |
| DEGREE OF SATURATION | 96.0 | 98.5 | 94.0 |
| CONFINING PRESSURE P.S.I. | 15 | 20 | 25 |
| MAX. TOTAL DEVIATOR STRESS P.S.I. | 22.3 | 27.5 | 29.3 |
| STRAIN AT FAILURE % | 20 | 20 | 20 |
| SHEAR STRENGTH AT FAILURE P.S.F. | 1600 | 1980 | 2110 |

DOMINION SOIL INVESTIGATION LIMITED

## SUMMARY OF LABORATORY TEST DATA

| | | | | | | |
|---|---|---|---|---|---|---|
| Borehole No. | 1 | 2 | 3 | 4 | 1 | 3 |
| Sample No. | 4 | 4 | 4 | 4 | 8 | 8 |
| Liquid limit (%) | 40.3 | 39.8 | 40.0 | 42.7 | - | - |
| Plastic limit (%) | 20.8 | 20.0 | 20.0 | 19.9 | - | - |
| Plasticity index (%) | 19.5 | 19.8 | 20.0 | 22.8 | - | - |
| Liquidity index | 0.4 | 0.3 | 0.5 | 0.4 | - | - |
| Natural moisture (%) | 28.5 | 26.2 | 29.4 | 29.0 | 28 | 25 |
| Void ratio | 0.78 | 0.69 | 0.74/0.80 | 0.72 | 0.79 | 0.70 |
| Bulk density (p.c.f.) | 120 | 124 | 123 | 124.8 | 122 | 124 |
| Cohesion (from unconfined compression test) p.s.f. | 863 | 1190 | 1420 | 420 | - | - |
| Compression index | - | - | 0.2 | - | - | - |
| Coefficient of consolidation (square feet per day) | - | - | 0.07-0.17 | - | - | - |
| Group Symbol | CI | CI | CI | CI | - | - |
| % Saturation | | | | | 96 | 96 |

| Borehole | Sample | Natural Moisture (%) |
|---|---|---|
| 1 | 4 | 28.5 |
| | 8 | 28.0 |
| 2 | 4 | 26.2 |
| | 7 | 21.2 |
| | 9 | 21.0 |
| | 12 | 21.7 |
| | 13 | 21.6 |
| 3 | 4 | 29.4 |
| | 7 | 22.7 |
| | 8 | 25.0 |
| | 9 | 21.5 |
| | 11 | 21.8 |
| | 12 | 20.3 |
| 4 | 4 | 29.0 |