

NEW BRIDGE

HAVELOCK ST.,

LUCKNOW

40P13 - 8



# DOMINION SOIL INVESTIGATION LIMITED

CONSULTING SOIL & FOUNDATION ENGINEERS

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CONSULTING ENGINEERS

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40P13-8

GEOCRE No.

Report On  
SOIL INVESTIGATION  
for  
PROPOSED NEW BRIDGE  
HAVELOCK STREET  
LUCKNOW, ONTARIO

by

Dominion Soil Investigation Limited  
1220 Trafalgar Street  
London Ontario

Ref: 73-4-L4  
July 25, 1973

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73-4-L4

I

INTRODUCTION

In accordance with authorization from B.M. Ross & Associates Limited, Consulting Engineers, a soil investigation has been carried out on Havelock Street in the Village of Lucknow, where it is proposed to replace and possibly realign to the south the existing road bridge over the east branch of the Lucknow River.

The requirements of the project were discussed with Mr. K. G. Dunn, P.Eng. who supplied the foregoing information.

The purpose of the investigation was to reveal the subsurface soil and groundwater conditions at the existing bridge site, and to determine the relevant soil properties for the design 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 April 6, 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 laboratory for classification and testing.

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 ground surface elevations. These were referred to the top of a hydrant located on the southeast corner of Havelock and Elgin Streets. The benchmark was established by the client and taken as Geodetic El. 892.77 feet.



### III SUBSURFACE CONDITIONS

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

Both boreholes encountered fill layers of silty sand with gravel and silty clay pockets. The fill is associated with the construction of the approaches to the existing bridge, and extends to depths of 6.2 and 6.3 feet in boreholes 1 and 2 respectively. A layer of topsoil was penetrated beneath the fill layers and found to range in thickness from 8 to 10 inches.

The natural subsoil underlying the fill and topsoil layers was found to consist of sand and gravel and gravelly sand with a trace to some silt. At borehole 2 location this stratum extends to a depth of 12.0 feet below the ground surface, while borehole 1 was terminated in this stratum at a depth of 20 feet.



Grading analyses were performed on five samples of the sand and gravel, and the results are shown as grain size distribution curves on Enclosure 4. The grading curves indicate that the sand and gravel contains from 5% to 11% silt and that the sample from borehole 2 has a predominant sand fraction. The relative density of the sand and gravel stratum is described as 'dense' based on 'N' values ranging from 53 to 65 blows per foot.

A grey silt layer was penetrated in borehole 2 between depths of 12 and 14½ feet, and the natural moisture content of the silt was found to be 14%. The relative density of the silt is described as 'dense' based on an 'N' value of 34 blows per foot.

A grey clayey silt stratum was encountered below the silt layer in borehole 2, and it extends to the limit of the borehole at a depth of 21½ feet. The natural moisture content of the clayey silt ranges from 9% to 14%. Atterberg Limits tests were also performed on a representative sample of the clayey silt giving values of Liquid Limit of 22%, Plastic Limit of 16%, and Plasticity Index of 6%. These results show that

the clayey silt has a low plasticity and compressibility. The consistency of the clayey silt is described as 'very stiff' based on 'N' values of 51 and 81 blows per foot.

#### IV GROUNDWATER CONDITIONS

The groundwater in borehole 2 was observed at El. 872.8, and no level was observed in borehole 1 due to the hole caving in after removal of the augers. The water level in the adjacent stream was observed at El. 873.6 on April 6, 1973.

#### V DISCUSSION AND RECOMMENDATIONS

The soil profile below the river bed consists of 'dense' sand and gravel, which is inherently capable of supporting normal spread footing foundations.

The river bed at the existing bridge site extends down to El. 871.0, therefore footings should be supported at or below El. 867 to provide sufficient protection against frost action. It is also recommended that a hydrology study be made to determine the maximum depth of scour and insure that the footings will have sufficient protection against undermining due to scour.



On the basis of the borehole results a maximum allowable soil pressure of 8000 p.s.f. is appropriate for the design of footings at or below El. 867, and this soil pressure incorporates a factor of safety of at least 3 against shear failure of the underlying soil. Total settlement of the footings is estimated to be 1 inch or less.

The coefficient of friction between the footings and the sand and gravel stratum may be taken as 0.5, and the factor of safety against horizontal sliding of the abutments should be at least 1.5.

#### Construction

A major problem in constructing footings in the prevailing ground conditions will be to control the groundwater and to prevent 'sloughing-in' of the sides of the excavation and 'boiling' in the bottom of the excavation due to an out-of-balance hydrostatic pressure.

It is considered unlikely that the excavations can be dewatered by normal pumping procedures due to the high permeability of the sand and gravel, therefore it will be necessary to use sheeting to maintain the stability of the excavation. The sheeting should be driven to a depth below the footing grade equal to the height of the prevailing water table above the footing grade to prevent boiling occurring in the bottom of the excavation. Due to the presence of the impervious clayey silt stratum at the north abutment location there will be a smaller upward flow of water and the excavation may be dewatered by normal pumping procedures. At the south abutment location the flow of water may be too great to handle by pumping and it may be necessary to pour a tremie seal prior to dewatering the excavation. Particular care should be taken to insure that the thickness of the tremie seal is sufficient to prevent uplift of the slab due to hydrostatic pressure when the excavation is dewatered. After construction of the footings, the sheeting may be left in place as a positive means of scour protection.

All backfill behind retaining walls should be free-draining granular material to prevent an out of balance hydrostatic pressure being exerted on the wall by entrapped water. The backfill must also be compacted to at least 95% of the standard Proctor dry density to preclude settlement of the fill and damage to the finished road surface.



Yours very truly,  
DOMINION SOIL INVESTIGATION LTD.

A handwritten signature in cursive script, appearing to read "D. G. McLean".

D.G.McLean, P. Eng.



A handwritten signature in cursive script, appearing to read "C. J. W. Atkinson".

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

# LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

## SOIL COMPONENTS AND GROUND WATER CONDITIONS.

GOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY	ORGANICS		BEDROCK	GROUND WATER LEVEL	DEPTH OF CAVE-IN
$\phi$	> 8"	3"	3/4"	4.76mm	2.0	0.42							

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.

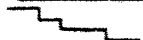
DYNAMIC PENETRATION RESISTANCE : to drive a 2"  $\phi$ , 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

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

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

SYMBOL :



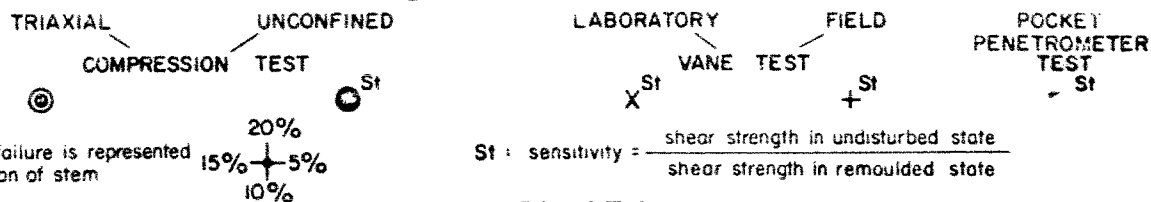
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## SOIL PROPERTIES.

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

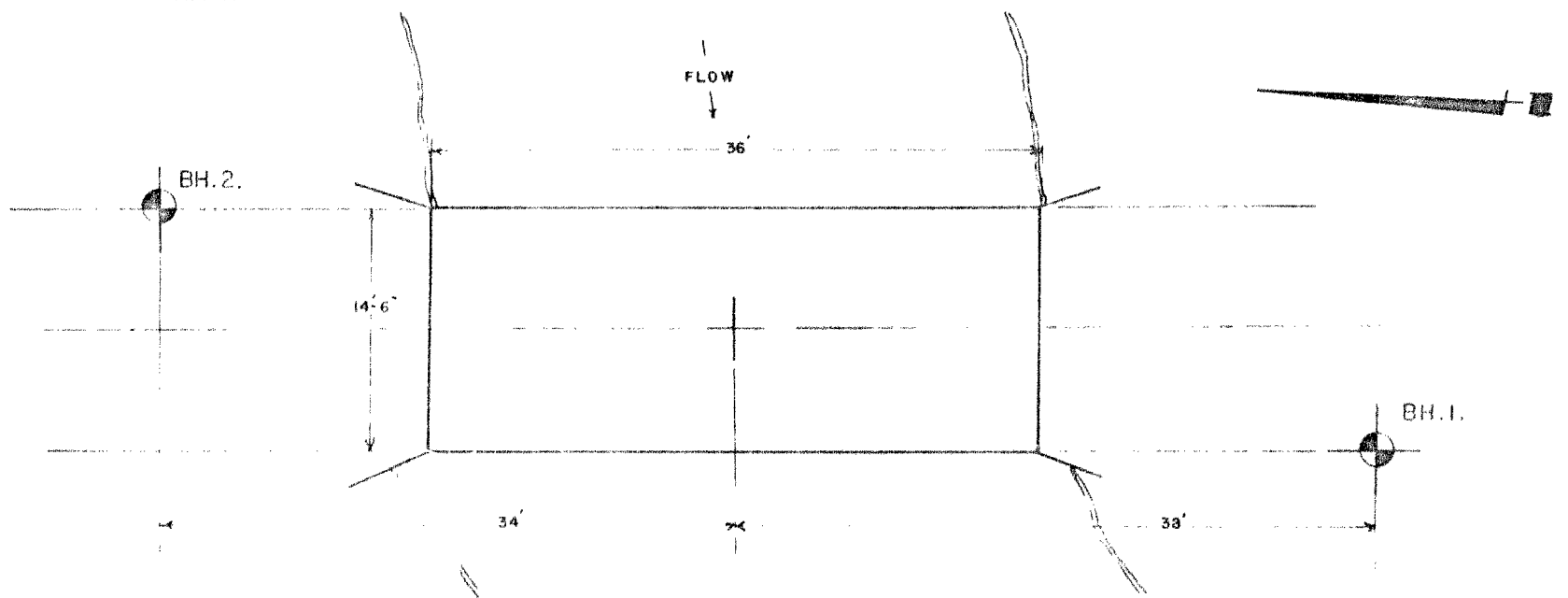
## UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

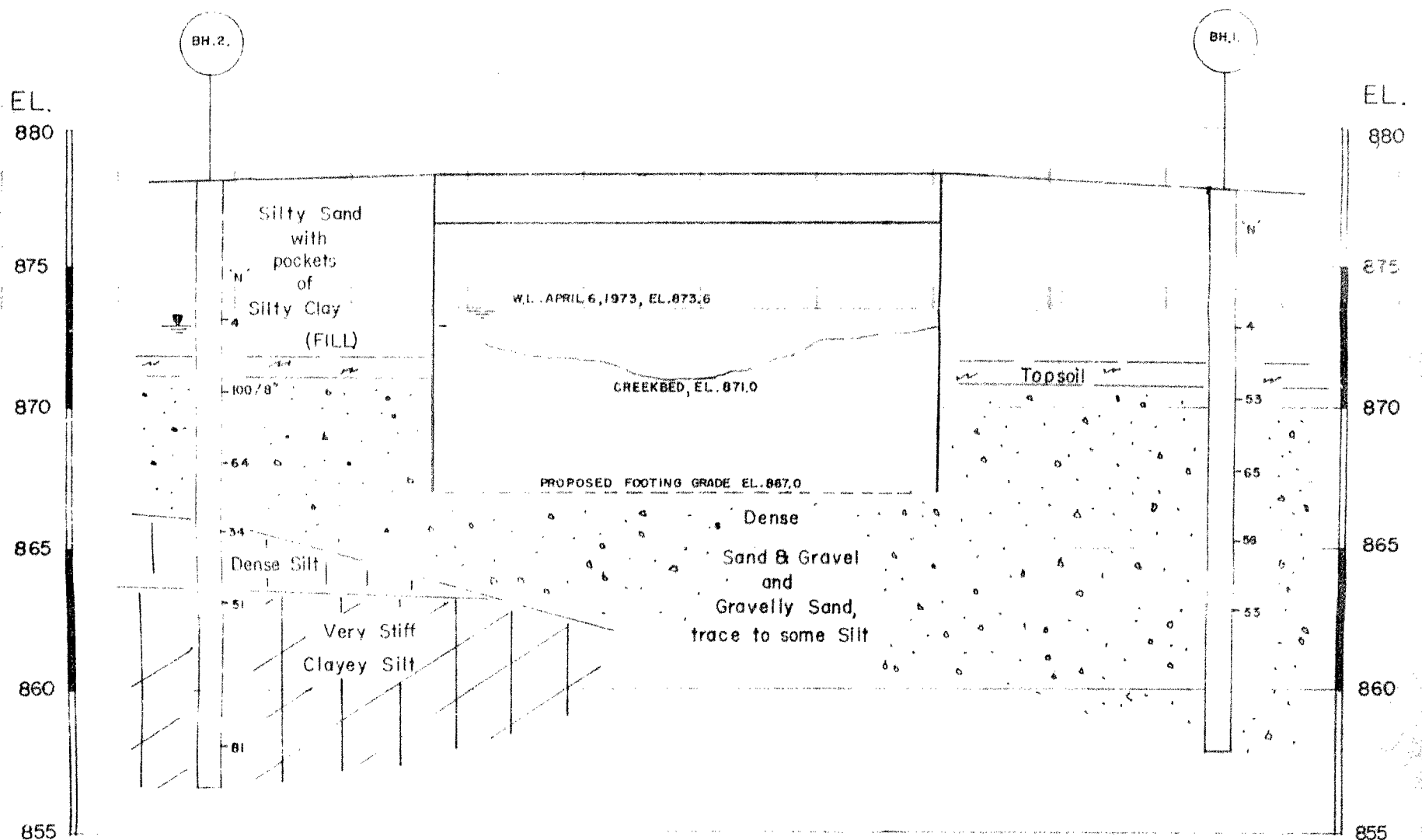


## SOIL DESCRIPTION.

COHESIONLESS SOILS :	RD :	COHESIVE SOILS	C lbs/sq ft
Very loose	0 - 15 %	Very soft	less than 250
Loose	15 - 35 %	Soft	250 - 500
Compact	35 - 65 %	Firm	500 - 1000
Dense	65 - 85 %	Stiff	1000 - 2000
Very dense	85 - 100 %	Very stiff	2000 - 4000
		Hard	over 4000



LOCATION OF BOREHOLES  
SCALE: 1"=10'



SUBSURFACE PROFILE  
SCALE: HOR.: 1"=10'  
VERT.: 1"=5'

40 P/3-8  
GEOCRE No.

# LOG OF BOREHOLE 1 & 2.....

Our Reference No 73-4-64.....

Enclosure No.....3

CLIENT: B.M. Ross & Associates Limited,  
PROJECT: Bridge No. 2,  
LOCATION: Lucknow, Ontario  
DATUM ELEVATION:

## DRILLING DATA

Method: Auger  
Diameter: 8-inch  
Date: April 6, 1973.

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE					WATER CONTENT %					REMARKS
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	IN' Blows / Foot	Blows / Foot					PLASTIC LIMIT W <sub>p</sub>	NATURAL W	LIQUID LIMIT W <sub>L</sub>		
								20	40	60	80	100					
								UNDRAINED SHEAR STRENGTH + FIELD VANE TEST								COMPRESSION TEST lbs/sq.ft.	
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VERTICAL SCALE: 1 inch to 5 feet

DOMINION SOIL INVESTIGATION LIMITED

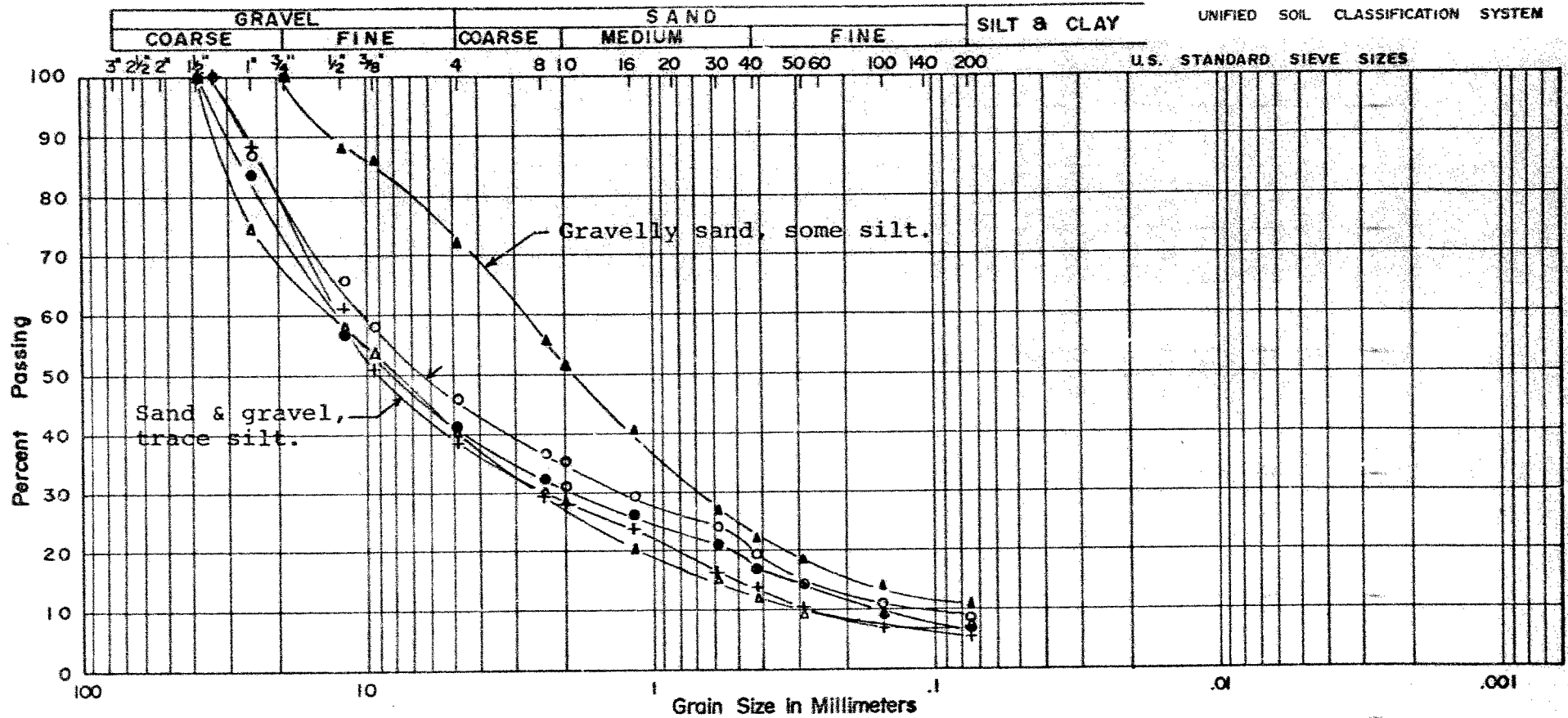
MADE:

CHECKED:

# DOMINION SOIL INVESTIGATION LIMITED

## GRAIN SIZE DISTRIBUTION

OUR REFERENCE N<sup>o</sup> 73-4-L4



PROJECT: Bridge No. 2.  
 LOCATION: Lucknow, Ontario  
 BOREHOLE N<sup>o</sup>: 1 1 1 1 2  
 SAMPLE N<sup>o</sup>: 2 3 4 5 3  
 DEPTH:  
 ELEVATION: o + • Δ ▲

COEFFICIENT OF UNIFORMITY:  
 COEFFICIENT OF CURVATURE:

Classification of Sample and Group Symbol:

PLASTIC PROPERTIES  
 LIQUID LIMIT % =  
 PLASTIC LIMIT % =  
 PLASTICITY INDEX % =  
 MOISTURE CONTENT % =