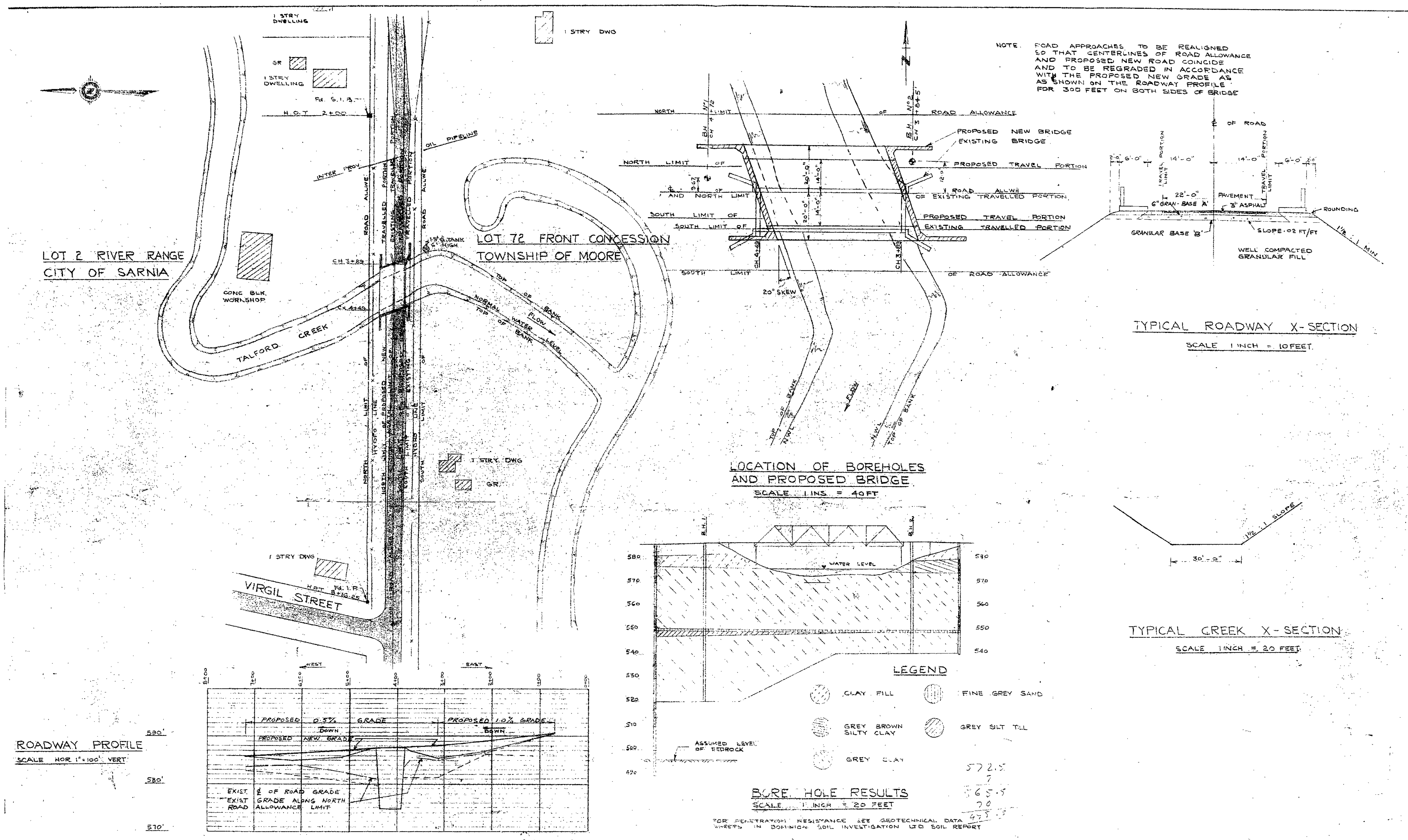
62-F-276M

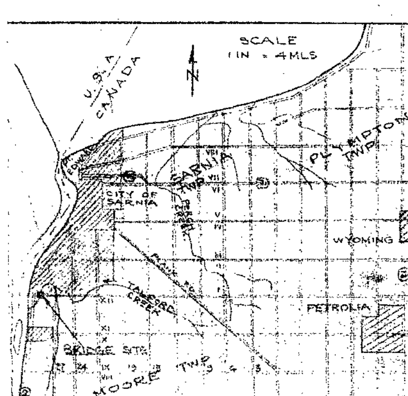
LOT #72 + #2

CON. FRONT &

RIVER RANGE

TALFORD CREEK





KEY PLAN.

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. LOW LYING LAND IN IMMEDIATE VICINITY OF BRIDGE SUBJECT TO FLOODING DURING PERIODS OF HIGH FLOW. HOWEVER THE EXISTING BRIDGE AND ROADWAY DO NOT APPEAR TO HAVE BEEN AFFECTED DUE TO THIER HIGHER ELEVATION
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 – HWY 40, SPAN 96 FT, 5.54 FT ABOVE N.H.W.L., 480 FT² X-AREA AT NHWL, 1 YEAR OLD; U/S BRIDGES (1) RD 14, LOT 27 CON XII TWP OF MOORE SPAN 20 FT, 4 FT ABOVE NHWL, 120 FT² X-AREA AT NHWL, 30 YEARS OLD (2) RD 14 LOT 26 CON XII TWP OF MOORE, SPAN 40 FT, 4 FT ABOVE NHWL, 320 FT² X-AREA, 50 YRS
   (B) REASONS WHY THESE BRIDGES ARE, OR ARE NOT, FAIR INDICATIONS OF SIZE OF PROPOSED BRIDGE: DOWNSTREAM BRIDGE WIDE DUE TO CLOSE PROXIMITY OF THE SAINT CLAIR RIVER (100 YARDS)
3. REASONS FOR CHANGES IN HEIGHT OR LENGTH FROM THAT OF OLD BRIDGE:

DATA (cont'd.)

4. IS DITCH, STREAM, OR RIVER GRADIENT LIABLE TO BE LOWERED? IMPROBABLE
5. NAVIGATION CLEARANCES REQUIRED, IF ANY: NONE
6. RAILWAY CLEARANCE REQUIRED, IF ANY: NONE
7. IF STRUCTURE IS OVER OR UNDER A RAILWAY HAS APPROVAL BEEN OBTAINED?
   (A) FROM RAILWAY CO. NOT REQ'D
   (B) FROM BOARD OF TRANSPORT COMMISSIONERS: NOT REQ'D
8. HAS APPROVAL BEEN OBTAINED UNDER NAVIGABLE WATERS PROTECTION ACT? NOT REQ'D
9. IS A TEMPORARY DETOUR REQUIRED? YES
   WHO WILL BUILD IT? CONTRACTOR – NOT YET KNOWN
   WHO WILL MAINTAIN IT? [illegible]
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 583.4' G.D. AND THE LOWEST WATER ELEVATION TO BE 576.5' G.D.
11. ROAD DESIGN INFORMATION:
    ESTIMATED A.D.T.: 400 [illegible] CARS PER DAY
    DESIGN SPEED: 60 [illegible]
    STOPPING SIGHT DISTANCE: 475 STRAIGHT ROAD

STRUCTURE DATA.

1. NET SPAN LENGTH AND TYPE OF BRIDGE: 60 FT. SPAN
2. ROADWAY WIDTH ON BRIDGE: 28 FEET
3. NUMBER & WIDTH OF SIDEWALKS: 2 – 6 FEET OVERALL WIDTH
4. SKEW ANGLE: 20°
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 ____ CU. YDS.
10. DRAINAGE AREA 38.0 SQ. MI.

FIELD INVESTIGATION MADE APRIL 1962

BY J. D. NISBET
SURVEY ENGINEER.

JAMES D NISBET
CONSULTING ENGINEERS
SARNIA ONTARIO

PROPOSED BRIDGE OVER
TALFORD CREEK

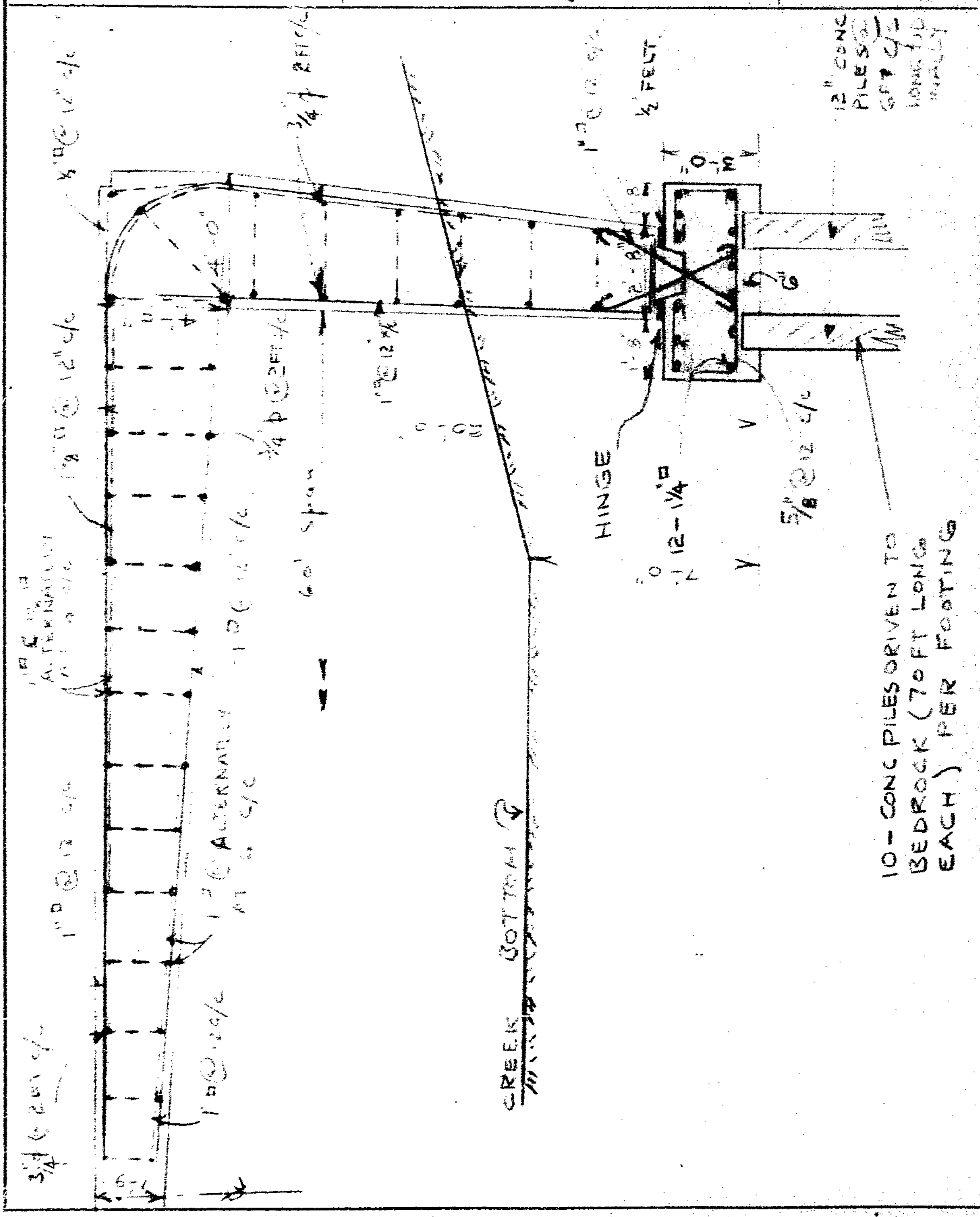
OWNER Co OF LAMBTON	MUNICIPAL DIST. No. 1
Co. LAMBTON	ROAD No. 14
TWP. OF MOORE / CITY OF SARNIA LOT 72 / 2	CON. FRONT / RIVER RANG.

— SITE PLAN. —

APRIL 1962 DATE.	JAMES D NISBET DESIGN ENGINEER.
BRIDGE NAME. NONE	

LOADING.	BRIDGE No.	DWG. No.
H-20-S16	TYP 15	E-62-45

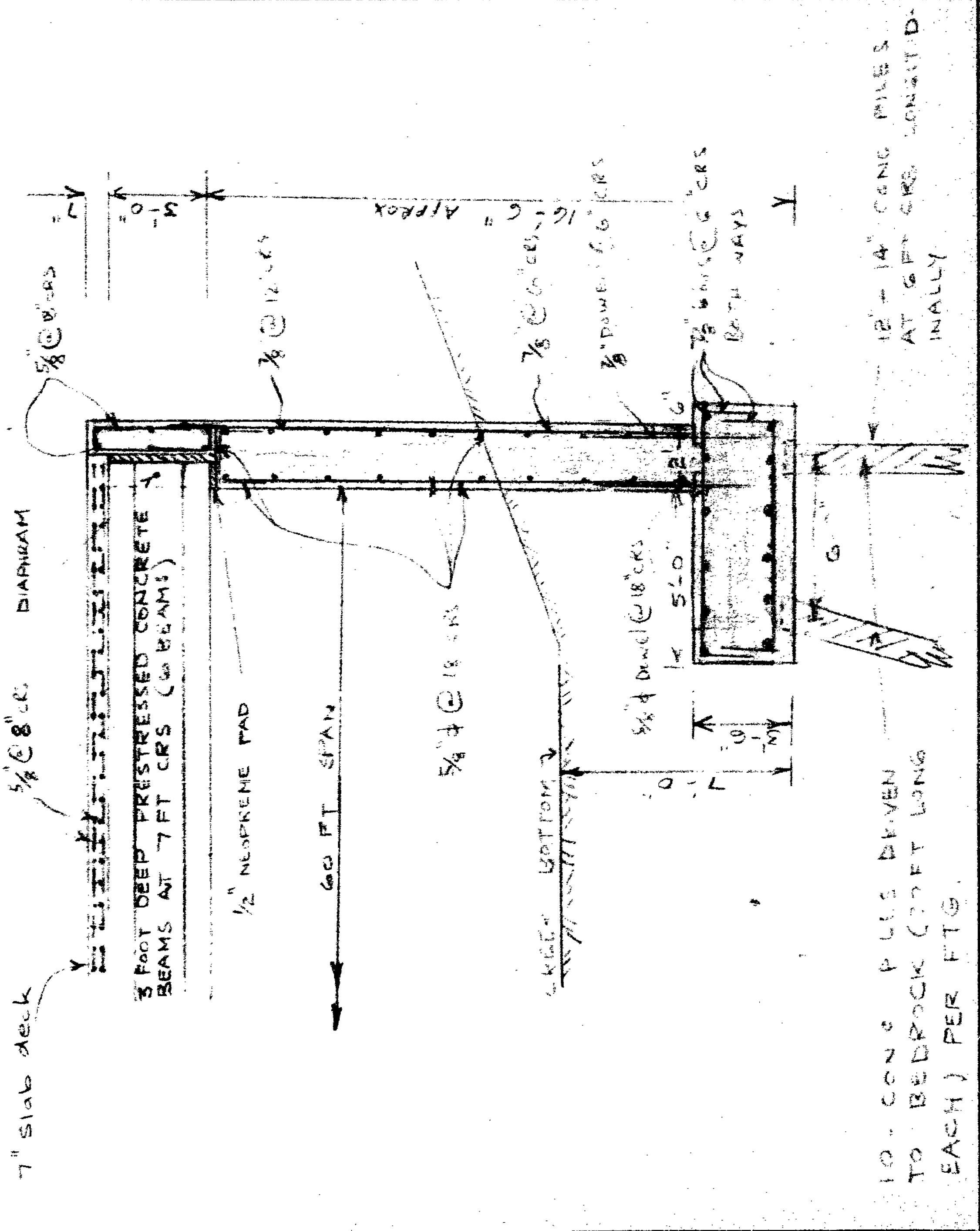
JAMES D. NISBET CONSULTING ENGINEER— ONTARIO LAND SURVEYOR SARNIA — ONTARIO	LAMBTON COUNTY ROAD BRIDGE OVER TALFORD CREEK TITLE PRELIMINARY DESIGN 'A'.	TWP # 13 No. E-62-45 Sheet 1 Of 3 By RDL Appd. JDN Date AUG 62 Scale N. T. S.



JAMES D. NISBET
CONSULTING ENGINEER
ONTARIO LAND SURVEYOR
SARNIA — ONTARIO

LAMBTON COUNTY ROAD TWP #13
BRIDGE OVER TALFORD CREEK
TITLE: PRELIMINARY DESIGN 'B'

No. E-62-45
Sheet 2 of 3
By RDL Appd. JDN
Date AUG 62
Scale N.T.S.



JAMES D. NISBET

CONSULTING ENGINEER—

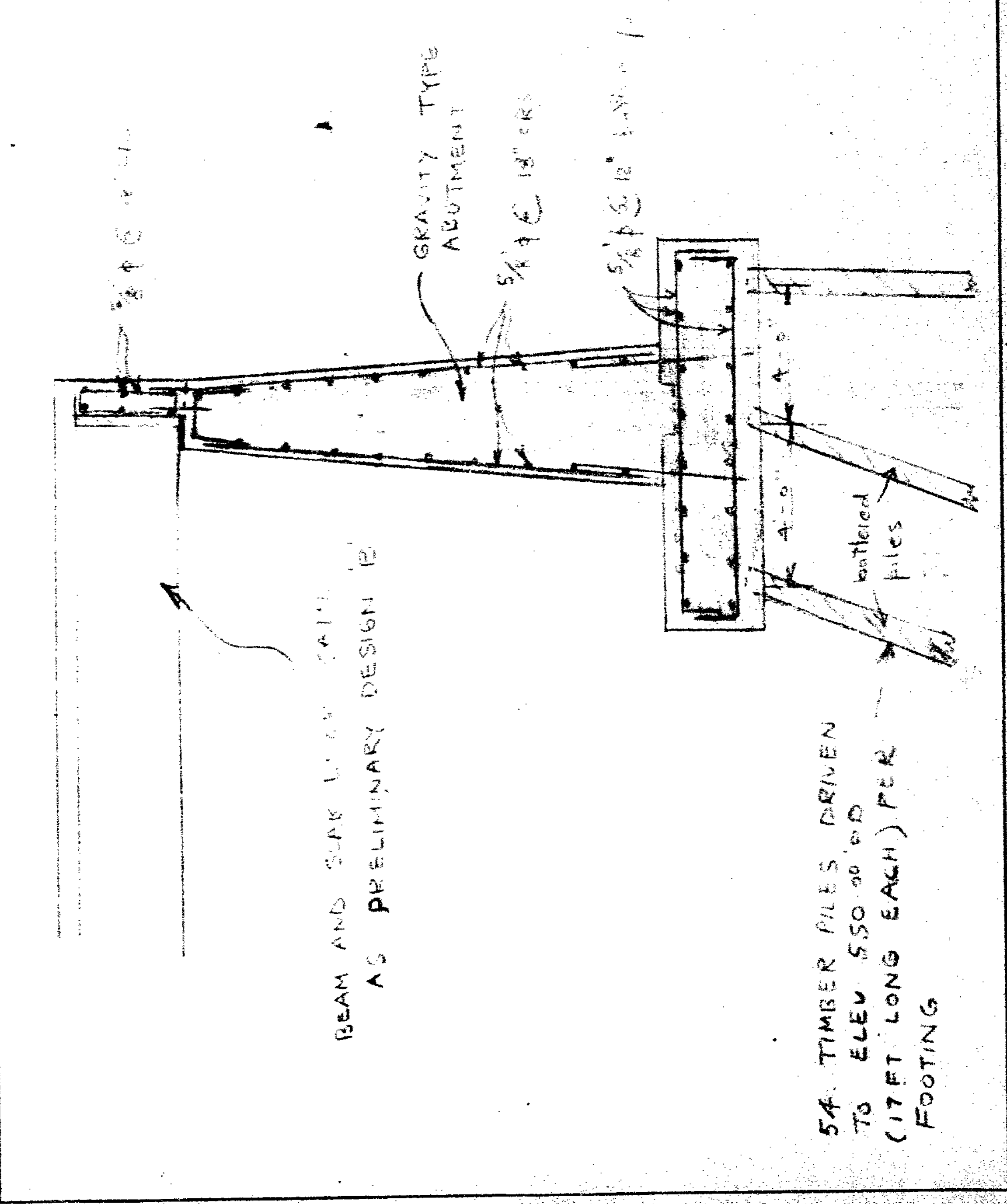
ONTARIO LAND SURVEYOR

SARNIA — ONTARIO

LAMBTON COUNTY ROAD
BRIDGE OVER TALFORD CREEK

TITLE:
PRELIMINARY DESIGN
'C'

TWP #13
No. E-62-45
Sheet 3 Of 3
By RDL Appd. JDN
Date AUG 62
Scale N.T.S.



Mr. K. L. Kleinsteiber,
Municipal Bridge Liaison Engr.,
Bridge Division.

Attn: Mr. G.C.E. Burkhardt.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

October 4, 1962.

Re: County of Lambton Bridge over Talford Creek, Twp. of Moore & City of Sarnia, Lot #72 & #2, Con. Front & River Range, Bridge Office Ref. #BA 1476, Structure Site #15-75.

We have reviewed the foundation report for the above-mentioned structure, which was prepared by Dominion Soil Investigation, Ltd.; also, the preliminary bridge drawings by James D. Nisbet, Consulting Engineer. Our comments are as follows:-

(1) Structure Foundations:

Subsoil at the site consists of a deep deposit of firm silty clay with an average shear strength of about 750 p.s.f. Conventional spread footings, therefore, are not a practical proposition as the safe bearing capacity of such material is in the order of 0.75 tons/sq.ft. A piled foundation is, therefore, necessary and for this purpose, two alterantives are proposed. End-bearing piles driven to bedrock, would probably be the most practical. Either steel tube, or steel 'H' piles may be used. The design loads will depend on the pile section and may be as high as 75 tons per pile in the case of 12 BP at 74 'H' piles. The piles should be designed to support all induced lateral thrusts. A scheme utilizing timber friction piles with a design load of 15 tons per pile, may also be adopted. In this case, a pile loading test will be necessary to determine the exact lengths to be used.

(2) Foundation Report:

The Soils Consultant has assumed bedrock at refusal of one dynamic cone test, only. Our experience has shown that refusal of the dynamic cone does not guarantee that end-bearing

cont'd. /2 ...

Mr. K. L. Kleinsteiber,
Municipal Bridge Liaison Engr.
Attn: Mr. G.C.E. Burkhardt

October 4, 1962.

(2) <u>Foundation Report</u>: (cont'd.) ...

piles will reach refusal at the same elevation. We feel that the Consultant should have made every effort to prove the bedrock at least at two locations. In this particular case, exact bedrock elevations could have been determined at very little extra cost, in both the boreholes which he drilled.

If you have any further queries in connection with this matter, please contact this Office.

K. G. Selby

K. G. Selby,
SR. FOUNDATION ENGR.,
For:

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

KGS/MdeF

cc: Foundations Office
Gen. Files.

ONTARIO

DEPARTMENT OF HIGHWAYS

Memo to Mr. A. Stermac,
Principal Foundations Eng.,
Materials & Research Section.

Date Bridge Division,
September 28, 1962

From G.C.E. Burkhardt

Subject County of Lambton,
Bridge over Talford Creek
Twp. of Moore & City of Sarnia,
Lot #72 & 2, Con Front & River
Range,
Our File #BA1476
Structure Site #15-75.

We are enclosing herewith one copy of the Foundation Report, by Dominion Soil Investigation Limited, one copy of the Hydrology Report and one copy of the Preliminary Plans for your comments.

The designer is proposing a simply supported type of structure or a rigid frame type. We would like to know what proposal you prefer in regard to the soil at the site.

We intend to approve the preliminary design as soon as possible and would appreciate it very much, if we could have your comments at your earliest convenience.

G.C.E. Burkhardt,
for K.L. Kleinsteiber,
Municipal Bridge Liaison Engineer.

GCEB/dm

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. K.L. Kleinsteiber,
Municipal Bridge Liaison Engineer,
Administration Bldg.,
DOWNSVIEW, Ontario.

From: J.D. Harris.

Date: September 27, 1962.

Our File Ref.

In reply to

Subject: Lambton County, structure 15-75,
Moore/Sarnia Twp. Lot 72 Front Con,
Talford Creek north of Corunna,
Municipal District 1, BW 708,
L.C. 366-62-69.

The proposed span of 60' will be adequate for the 25 square mile watershed.

The only serious problem at the site is that of ice jamming. This occurs at the existing bridge and at the acute bend 500' downstream, causing flooding of the upstream properties.

In order to improve this situation as much as possible, it is strongly recommended that the new bridge be located as far west from the existing bridge as property and other considerations will permit. This would reduce or eliminate the skew and also ease the sharp bend immediately downstream from the bridge. If possible, the right-angled bend 500' downstream should also be eased.

As it will not be possible to eliminate ice jamming completely, relief flow should be permitted in case of a severe jam by providing a sag in the west approach at an elevation of approximately 585.0.

The lowest point of the new soffit should be at or above elevation 585.0, to give 1' clearance above ice level, or preferably more is a straight soffit is used.

A three span structure is not recommended due to the heavy ice conditions.

H:jb

J. D. Harris,
for B. Wilkie,
Bridge Hydrology Engineer.

BA 1476

MR. JAMES D. NISBET
CONSULTING ENGINEER
P.O. BOX 51
SARNIA, ONT.

File No. E-62-45

62-F-276M

Report on

SOIL INVESTIGATION

for

LAMBTON COUNTY BRIDGE TWP #13

LOT 72, FRONT CONCESSION TOWNSHIP OF MOORE
AT TALFORD CREEK

by

DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO

Reference No. 2-4-L2

April, 1962

CONTENTS

ENCLOSURES

INTRODUCTION

In accordance with a letter of authorization from Mr. Nisbet dated April 4, 1962 a soil investigation has been carried out at a site in the Township of Moore where it is proposed to replace an exisiting County road bridge with a new structure.

It is understood that the new bridge will have a span of approximately 60 feet and a width of 30 feet. We have been asked to consider the use of a single-span freely supported deck on spread footings and as an alternative a 3-span composite wood and concrete structure supported on timber piles.

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.

It has been useful in preparing this report to refer to a similar project at the intersection of Talford Creek and Highway No. 40 approximately half a mile South of this site, (D.S.I.L. report 105/F86, December 5th, 1958) where almost identical soil conditions were encountered.

I. DESCRIPTION OF SITE & GEOLOGY

The site lies approximately 2 miles North of the Town of Corunna and within one-half mile of the St. Clair River. The existing bridge carries the LaSalle Road across Talford Creek, one of the many small streams draining the St. Clair Clay Plain which covers most of Lambton County. The area surrounding the site is slightly undulating and thinly wooded.

II. FIELD WORK

Field work was carried out during the period 5th to 7th of April, 1962 and comprised two boreholes at the locations shown on enclosure 1. The holes were drilled with a 5 inch diameter power auger. A dynamic cone penetration test was made adjacent to borehole 1 to a depth of 22 feet. A second cone test was made from the bottom of borehole 1 at 61'-6" until refusal was encountered at a depth of 86 feet. From the sudden increase in penetration resistance and the bouncing of the drive hammer together with other records* of bedrock elevations, it is assumed that this level of 86 feet (corresponding to geodetic elevation 496 feet) is bedrock.

Standard Penetration tests were made at frequent intervals using a 2 inch O.D. Split-Spoon. A constant drive energy was employed in the Standard Penetration and dynamic cone tests using a 140 pound hammer dropping 30 inches. The former tests provided disturbed samples of the strata and the latter a continuous record of soil density. Undisturbed samples of the soil were recovered in 2 inch diameter thin-walled Shelby tubes and insitu vane shear tests were performed using a four-bladed vane, 2 inches in diameter and 4 inches in length. Both the undisturbed and remoulded vane shear strengths were measured and the ratio of these figures is defined here as the sensitivity of the soil.

The results of the field tests are recorded on data sheets comprising enclosures 2 and 3. Elevations have been referred to a geodetic bench marked on the Northwest wing wall (El.587.75 feet).

III. SUBSURFACE CONDITIONS

A subsurface profile is shown on enclosure 1.

Above Els. 576.5 in borehole 1 and 579.3 in borehole 2 the soil is a backfill placed during construction of existing bridge. The upper 5 feet of material in borehole 2 is rubble, and the remainder of the fill is grey and brown silty clay mixed with traces of gravel and organics.

---

* See for example reference 6.

Below the fill in borehole 2 a deposit of very stiff grey-brown silty clay extends for 5 feet.

Below Els. 576.6 in borehole 1 and 574.3 in borehole 2 a thick deposit of grey clay extends to what is assumed to be bedrock at El. 496.2. In borehole 1 a 2-foot layer of hard silt till, overlain by 6 inches of fine silty sand, was found within the clay layer near El. 550. In borehole 2 a corresponding layer of sand, 6 inches thick, was found at the same elevation but no till was found in this hole.

The results of laboratory tests show the clay to be of intermediate plasticity having a natural moisture content of about 35% and a plasticity index of 22%. The stiffness of the material measured by its reaction to both field and laboratory tests is significantly greater above El. 550 at borehole 2 than at borehole 1, as shown by the following comparison.

Test	Ratio of Results (Borehole 1:Borehole 2)
Blow count (Split Spoon)	1:1.6
Insitu vane shear	1:2.5
Unconfined compressive strength	1:1.2
Pocket penetrometer	1:2.5

The sensitivity of the clay as measured in the field vane shear tests varied between 1.3 and 2.0.

IV. LABORATORY TESTS

A series of laboratory tests has been carried out to determine the physical properties of the thick clay stratum. These include one consolidation test from which the results are recorded on enclosure 4, and a summary of the remaining test data appears on enclosure 5.

The estimated preconsolidation load on sample 4 from borehole 1 is 1.25 t.s.f. compared with an existing overburden pressure of 0.8 t.s.f. The overconsolidation ratio is therefore 1.6.

From the combined results of laboratory unconfined compression tests and field vane shear strengths a cohesion value of 750 p.s.f. is assumed for the clay.

V FOUNDATIONS

The analysis of the soil conditions has led to the consideration of 4 possible types of foundation arrangement:

(a) conventional spread footings
(b) footings carried on piles
(c) 3-span composite timber and concrete structure on piles of 25 to 30 foot length
(d) use of long (60 foot) timber piles.

(a) Spread footings

The level of the bed of the creek is at El. 574 feet, so that allowing 7 feet for erosion the highest elevation of footings would be 567 feet. Assuming a deck load of 240 p.s.f. (from prestressed beams) and a live load of 9.1 kips per lineal foot, the total loading on a footing 15 feet wide is calculated to be 3860 p.s.f.

Using a value of 750 p.s.f. for cohesion of the clay, the ultimate bearing capacity of the footing according to Meyerhof is calculated to be 4425 p.s.f. The soil has therefore insufficient shear strength for this type of foundation if a reasonable factor of safety is to be obtained.

(b) Footings carried on piles

The total load on each abutment is estimated to be 330 tons.

The use of timber piles driven to El. 550 is now considered. This elevation is chosen because of the thin layer of relatively dense material found there which may contribute to the bearing capacity of the piles

The adhesion between the soil and the pile shaft is taken as 87% of the cohesion value, or 650 p.s.f.* The effective length of the piles is assumed to be 12 feet, or 5 feet less than the depth from the erosion level (El. 567) to the tip. For piles of 12 inches diameter the calculated ultimate bearing capacity is 15 tons. Applying a safety factor of 2.5 the safe working load is thus 6 tons per pile.

---

* See reference 7.

A total of 55 piles per abutment would be required, or 5 rows of 11 piles spaced over an area 30 feet x 10 feet. Considering the pile group as a deep footing the shear strength around the perimeter of the group over an effective depth of 12 feet is 360 tons which is slightly more than the applied load. The bearing capacity of the group is therefore not critical. The increase in pressure due to dead load on a horizontal plane immediately below the pile tips is 900 p.s.f. The resulting consolidation settlement is estimated to be 4 inches* but this will occur over a very long period of time. The calculated time periods for 25, 50 and 90% consolidation are 2.5, 10 and 70 years respectively. The stiff layer of soil at El. 550 may reduce the total settlement by spreading the applied load, but this effect is not calculable.

In view of the large number of piles and large calculated settlements arising from the foregoing arrangement, the use of concrete piles driven to bedrock should be considered. The Raymond cast-in-place pile or Franki displacement caisson would be suitable in these conditions. Working loads of 40 tons per pile could be used and the number of piles per abutment would be reduced to 8 or 9. The total settlement of the structure would then be approximately equal to the elastic deflection of the piles. Because of the reduced pile footage required, it is believed that such piles would compete economically with a timber pile arrangement.

(c) <u>3-span composite structure</u>

For the proposed 3-span arrangement the total load on each pier is approximately 125 tons of which 30% is dead load. This requires 21 six-ton piles. Assuming 3 staggered rows at 5 foot centres the group may be regarded as a deep footing with dimensions of 30 feet x 6 feet. As in (b) above the shear strength around the perimeter of the group is greater than the total applied load. The consolidation settlement arising from dead loading is calculated to be 1.5 inches, and the time periods are as for the single-span arrangement described above. Again, the total settlement may be significantly reduced by the stiff layer at El. 550.

---

* The pore-pressure coefficient A has been taken as 0.5 giving a "Skempton" correction factor of 0.7 which has been applied to the oedometer settlement.

(d) Long timber piles

Higher bearing capacities could be obtained by driving timber piles to greater depths. For example piles of 12-inch diameter driven to El. 521 (which is mid-way between the stiff layer at El. 550 and bedrock at El. 492) would have a safe working load of 18 tons per pile with a factor of safety of 2.5. The total length however would be of the order of 60 feet which is probably beyond the economic maximum for timber piling. In addition there is some danger that the piles would be damaged when driven through the stiff till layer at El. 550.

The most favoured of the foundation arrangements described here is the use of concrete piles bearing on bedrock. This has the advantages of simplicity and complete freedom from detrimental settlement effects. Because of the reduced pile footage allowed by higher working loads, it is believed that this method will also compete economically with timber pile designs.

VI CONSTRUCTION

No unusual construction problems are anticipated. If the design chosen requires excavation, the surface waters should be diverted away from the site using temporary cofferdams of earth or sheet piles. The soil has a very low permeability so that no difficulty will be encountered in keeping the excavation dry.

VII SUMMARY

1. Below the elevation of the creek bed the soil is a firm to stiff grey clay extending to bedrock at El. 492.0 feet. A 2-foot thick layer of very stiff till overlain by 6 inches of fine sand, was encountered near El. 550 in borehole 1. At borehole 2 only the sand layer was present.

2. Laboratory tests show that the clay is of intermediate plasticity and slightly overconsolidated. From results of unconfined compression and vane shear tests the cohesive strength has been taken as 750 p.s.f. Other physical properties are recorded on enclosure 5.

3. The soil has insufficient shear strength to accomodate conventional spread footings.

4. Several piling arrangements have been considered, and of these the use of concrete piles bearing on bedrock is considered the best solution.

VIII REFERENCES

1. The Physiography of Southern Ontario by L.J. Chapman and D.F. Putman of the Ontario Research Foundation - University of Toronto Press 1951.

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

3. Terzaghi and Peck: Soil Mechanics in Engineering Practice. John Wiley and Sons, New York 1948.

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

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

6. Soderman, Kenney & Loh, Geotechnical Properties of Glacial Clays in Lake St. Clair Region of Ontario, Proc. 14th Canadian Soil Mechanics Conference, 1960.

7. Tomlinson, Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, Vol. II, London, 1957.

8. Woodward and Lundgren, Pile Loading Tests on Stiff Clays, Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering, Paris 1961.

9. N.A. Csutovich, Calculation of Settlement by the Method of Equivalent Layer Thickness, Extract from "Foundations" by Szechy, Budapest 1957, Trans. L.R. Szalatkay.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
J. PARK
PROVINCE OF ONTARIO

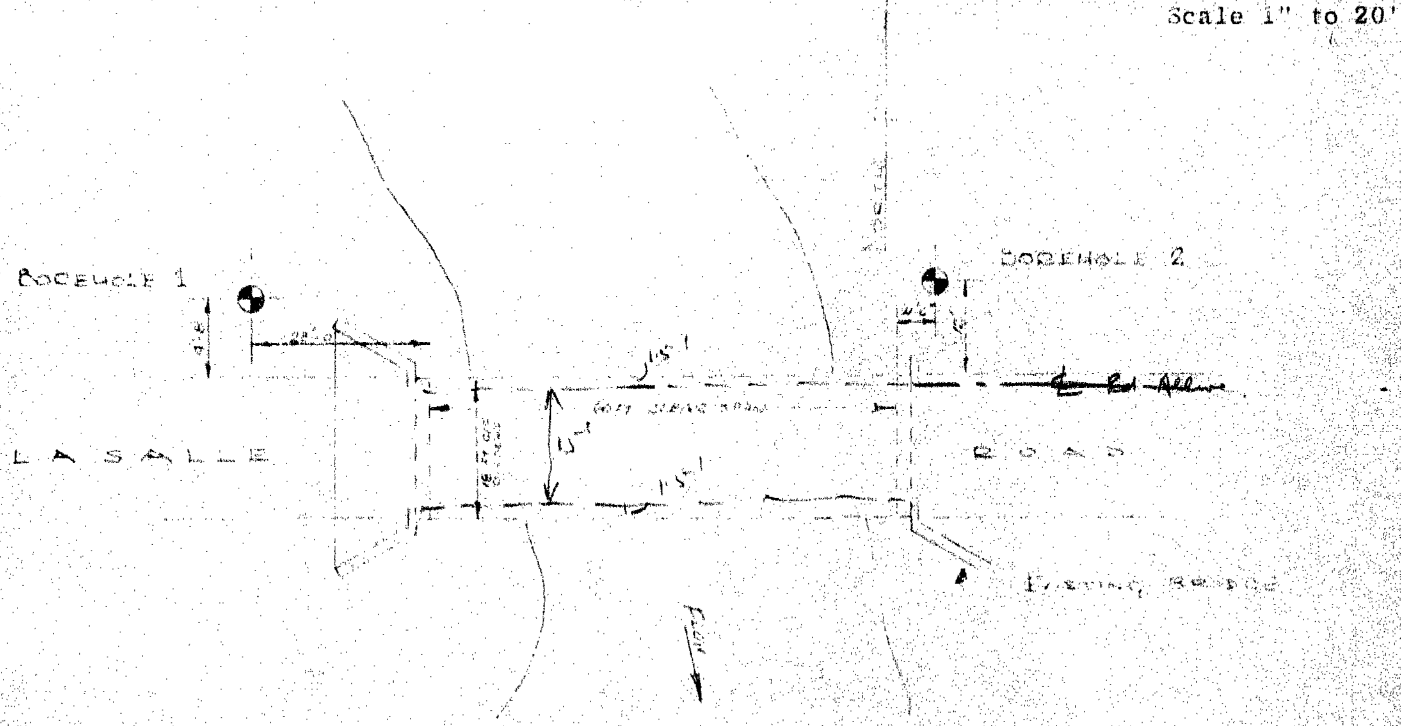
DOMINION SOIL INVESTIGATION LIMITED

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

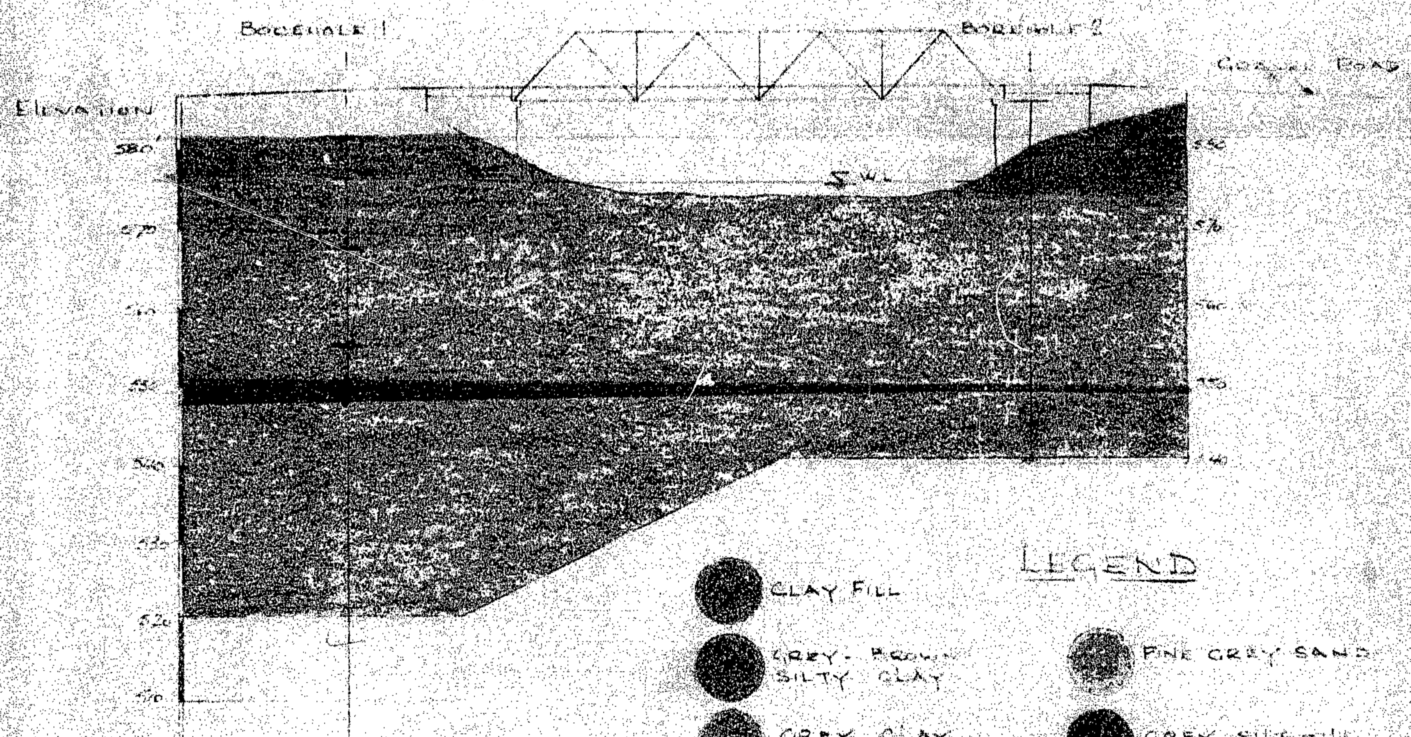
Reference No. 2-4-L1

Enclosure No. 1

Scale 1" to 20'



LOCATION OF BOREHOLES



SUBSURFACE PROFILE

DOMINION SOIL INVESTIGATION LIMITED

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 1 . . .

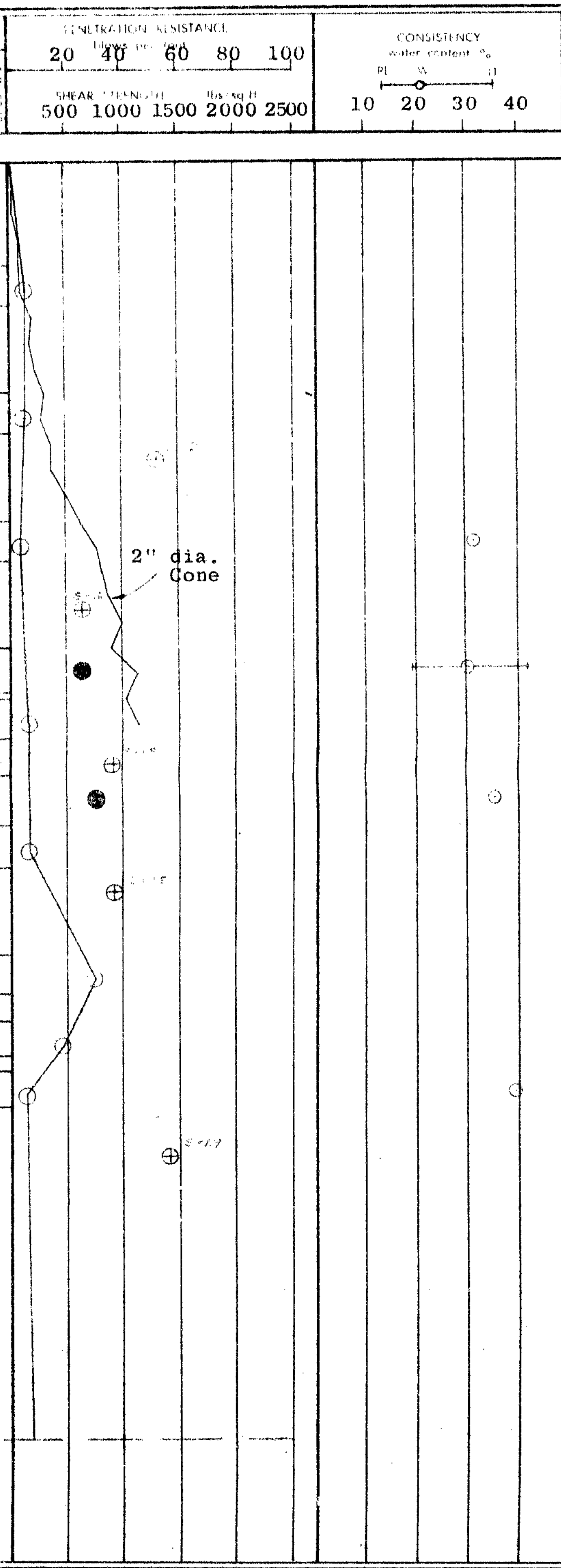
OUR REFERENCE NO. 2-4-L2

CLIENT: Mr. James D. Nisbet, Consulting Engr.
PROJECT: Lambton County Bridge, Twp. No. 13
LOCATION: LaSalle Road at Talfourd Creek
DATUM ELEVATION: Top of Northwest Wingwall 587.75

METHOD OF BORING: Power Auger
DIAMETER OF BOREHOLE: 5"
DATE: 5-6 April 1962

ENCLOSURE NO. 2

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	TYPE	Adv. (Blows/ft of Sample)
582.0	0	Ground Surface			
		Brown reworked silty clay, traces of gravel and organics			
	5		1	SS	5
576.5					
	10		2	SS	5
570.0				vane	
	15		3	SS	4
				vane	
	20	Grey clay	4	TW	
560.0			5	SS	6
				vane	
	25		6	TW	
			7	SS	6
				vane	
	30				
550.5		Fine grey silty sand			
550.0			8	SS	30
		Grey clayey sandy silt till			
548.0			9	SS	18
	35		10	SS	5
				vane	
	40	Grey clay			
540.0					
	45				
	50				
		Continued on Enclosure 2A			



VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: MC CH'D: JP

GEOTECHNICAL DATA SHEET FOR BOREHOLE 1 (Continued)

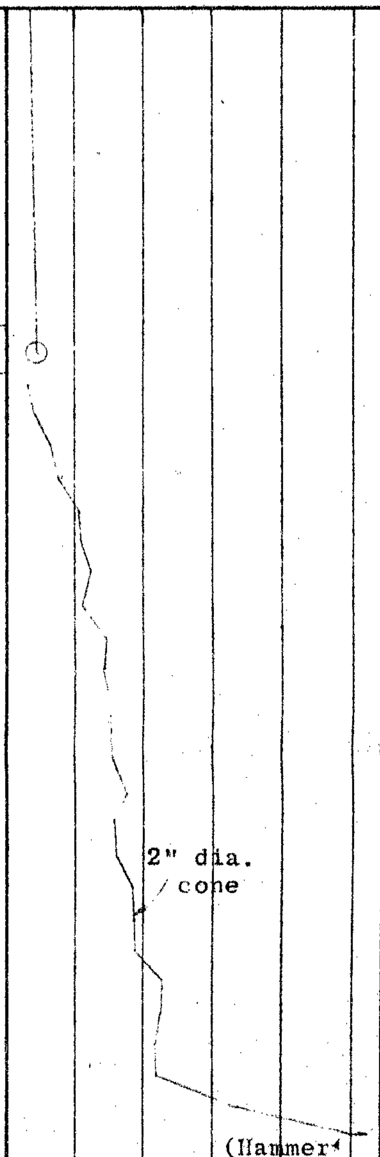
OUR REFERENCE NO. 2-4-L2

CLIENT:
PROJECT:
LOCATION:
DATUM ELEVATION:

METHOD OF BORING:
DIAMETER OF BOREHOLE:
DATE:

ENCLOSURE NO. 2A

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES -N- or Advancement of Sampler	PENETRATION RESISTANCE blows per foot 20 40 60 80 100 / SHEAR STRENGTH lbs/sq ft	CONSISTENCY water content % PL W LL 10 20 30 40	REMARKS
532.0	50								
	55	Grey clay							
	60								
520.5				11	SS	9			
	65								
	70								
510.0									
	75						2" dia. cone		
	80								
500.0									
	85								
496.2		End of borehole (assumed bed-rock)					(Hammer bouncing)		
	90								



LEGEND

SS denotes 2" O.D. split spoon (disturbed) sample

TW denotes 2" dia. thin-walled Shelby tube (undisturbed) sample

S=2 denotes insitu vane shear test and sensitivity

● denotes shear strength in unconfined compression test

⊙ denotes natural moisture content

VERTICAL SCALE: 1 IN. TO 5 FT.

MADE: MC CH'D: JP

OUR REFERENCE NO. 2-4-L2

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 2 . . .

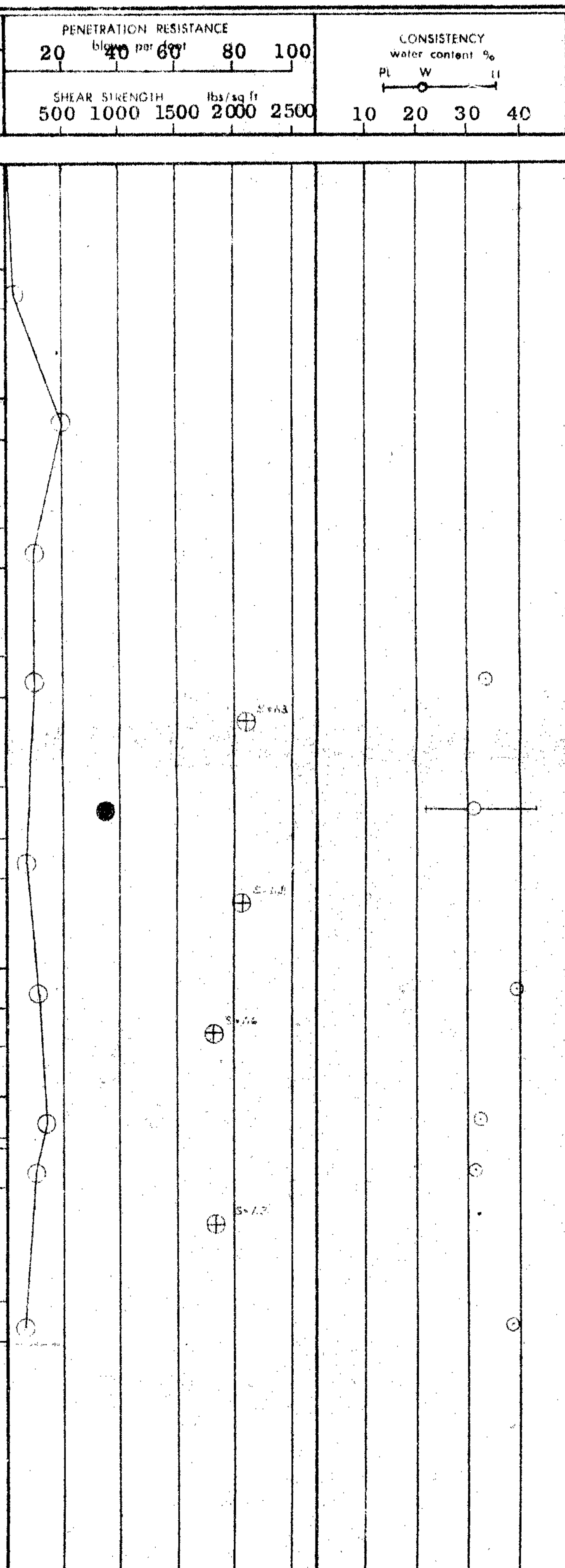
CLIENT: Mr. James D. Nisbet, Consulting Engr. METHOD OF BORING: Power Auger ENCLOSURE NO. 3

PROJECT: Lambton County Bridge, Twp. No. 13 DIAMETER OF BOREHOLE: 5"

LOCATION: LaSalle Road at Talfourd Creek DATE: 6-7 April 1962

DATUM ELEVATION: Top of Northwest Wingwall 587.75

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	TYPE	—N— or Advancement of Sampler	REMARKS
586.3	0	Ground Surface				
		Rubble fill				
581.3	5	Grey, reworked silty clay, traces of organics (fill)	1	SS	3	
579.3		Grey-brown silty clay				
	10		2	SS	20	
574.3		Grey clay				
	15		3	SS	9	
	20		4	SS	9	
				vane		
	25		5	TW		
560.0			6	SS	7	
				vane		
	30					
			7	SS	11	
				vane		
	35		8	TW		
550.0		seam of fine sand	9	SS	14	
			10	SS	10	
	40			vane		Legend See Borehole 1
540.8	45		11	SS	6	
		End of Borehole				
	50					



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

Order No. 7 - 4 - 12 Enclosure No. 4

Dominion Soil Investigation Ltd.

CONSOLIDATION TEST