66-F-259M
COUNTY ROAD #7
PARKS CREEK
LOTS 5 & 6. CON. 6

BH 2299

C.C. PARKER AND ASSOCIATES LIMITED
688 QUEENSDALE EAST
HAMILTON, ONTARIO

TYENDINAGA.

FOUNDATION CONDITIONS
PROPOSED RECONSTRUCTION OF PARKS CREEK BRIDGE
HASTINGS COUNTY ROAD #7, ONTARIO.

SITE 11-143
66-F-259 M

Project: H108

March 1966

William Trow Associates Limited

67 Queen Street South
Hamilton, Ontario
525-1407

William Trow Associates Ltd.

Soil Mechanics
Consultants
W. A. Trow
MSc. MEIC. P. Eng.
K. Peaker
PhD. MEIC. P. Eng.
D. H. Shields
PhD. MEIC. P. Eng.

HAMILTON OFFICE

Project: H108

March 22, 1966

C.C. Parker & Associates Limited,
688 Queensdale East,
Hamilton, Ontario.

66-F-259 M

Attention: Mr. D. Cramm, P.Eng.

Foundation Conditions
Proposed Reconstruction of Parks Creek Bridge
Hastings County Road #7, Ontario.

Dear Sirs,

As advised during a recent meeting, we have completed the survey of subsoil conditions at the site of this proposed new bridge over Parks Creek. The bridge is located between Lots 5 and 6, Concession VI, Township of Tyendinaga, County Hastings. Field work comprised two borings advanced into sound limestone bedrock, plus two associated dynamic cone penetration tests. A plan and section of the site is appended as Dwg. 1.

Our brief report of the survey follows:

1) SUBSOIL

1.1. The shallow valley of Parks Creek has been incised into a variable but generally quite competent series of silty clay or gravelly sand tills, laid down during the final phases of the Wisconsinan Glaciation. A thin cover of recent alluvium exists in the narrow flood plain to the south of the present creek channel.

1.2. Grey crystalline Trenton Limestone underlies the till a few feet below creek bed level, at approximately project Elevation 175-177 feet.

1.3. Existing bridge approach fills have been predominantly constructed with a mix of cohesive or clayey silt and silty sand and gravel.

1.4. Detailed soil descriptions and sampling information for the two test borings are given in Dwgs. 2 and 3.

2) FOUNDATIONS

2.1. Spread footings designed to a safe allowable bearing pressure of 5 ksf can be established at any level in the glacial clay or sand till subsoil. However, a uniform founding level at project Elevation 180.0 is recommended, to provide a minimum 4 feet of scour protection below the bed of the creek.

2.2. Excavation for the abutment foundations should be protected along the streamward side with small earth-fill coffer dams. Water entering the excavations can be pumped out from a system of peripheral collector ditches dug around the footing area. Alternatively, the excavation can be completed in the wet using a small dragline.

If the latter method is employed, dewatering would only be required during forming and pouring of the footing member.

2.3. Considerable assistance in the forming and construction of the abutment foundations would be provided by pouring a lean-mix concrete mat on the base of each excavation as soon as it has been cleaned out and dewatered.

2.4. Any tendency for the excavation sides to slough in under the strong seepage forces can be prevented by covering the cut faces with a coarse but well graded sand and gravel.

2.5. Because of the need for a small protective earth coffer dam around the streamward side of each abutment excavation, the two foundations must be constructed consecutively rather than at the same time. Each coffer dam will require a partial diversion or restriction of the creek channel.

2.6. Alternative consideration can be given to supporting the proposed structure on short timber or H piles driven to refusal on or close to bedrock. However, the design is not particularly recommended because of:-

a) The shortness of the pile section involved.

b) The need for additional sheet piling as scour protection unless the pile cap is founded four feet below creek bed level.

c) The presence of adequately competent bearing subsoil at the same "creek-bed-minus-four-feet" level.

2.7. Settlements beneath the recommended spread footing design will be of a very small order and can be disregarded.

3) <u>EARTH PRESSURES AND SLIDING RESISTANCE</u>

3.1. Horizontal pressures exerted on the abutments by the retained fill and traffic surcharge and impact loadings can be determined at any depth from the expression:

$$p = K\ (\gamma h_1 + \gamma' h_2 + q)$$

where: K = 0.35 the recommended earth pressure coefficient

γ = 130 pcf, the estimated unit weight of retained fill

γ' = 65 pcf, the estimated submerged density of alluvial soil and fill material below the water level

h_1 = height of retained soil above the water table

h_2 = depth below the water table of the point under consideration

q = any surcharge, in psf

With free-draining backfill there will be no unbalanced water pressure on the back of the wall.

3.2. A friction coefficient of 0.65 should be used in computing the sliding resistance developed along the footing bases. If the resisting force is less than 2 times the estimated sliding force, the footing must be extended back beneath the fill to increase the sliding resistance.

4) APPROACH FILLS

4.1. No stability problem exists for either of the low approach embankments. The base soils are essentially granular.

4.2. Existing topsoil or noticeably organic-rich pockets along the sides of the existing fills should be removed prior to any widening and heightening. Additional fill materials should be placed in successive lifts not exceeding 10 to 12 inches thickness, with each layer compacted to not less than 95 percent of standard proctor density using a vibratory-type roller.

4.3. Both upstream and downstream side slopes adjacent to the abutments should be faced with 12 inches of well graded pit run gravel followed by 18 to 24 inches of stone riprap, for scour protection during periods of peak flood. Slope sections more than 50 feet back from the abutment faces should be turfed.

4.4. As indicated in section 3.1. above, the fill immediately behind the abutment walls should be of coarse granular character, to facilitate rapid equalization of hydrostatic pressures on either side of the walls during rapid flood level oscillations.

No further problems are anticipated.

This investigation was processed and completed through our Hamilton office during the period March 11th - 21st, 1966. We trust you will find the report and recommendations in order. If further queries should arise, please do not hesitate to contact us.

Yours very truly,

John D. Morton

John D. Morton

JDM/yg

Encls. (3)

DIST: C.C. Parker Associates (6)

W Trow

William A. Trow, P.Eng.

WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS — SOIL MECHANICS CONSULTATION

DRAWING No. 2

PROJECT No. H108

LEGEND

PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	
2" O.D. SPLIT TUBE	—o—o—o—	ATTERBERG LIMITS	
2" I.D. SHELBY TUBE	—+—+—+—	LIQUID LIMIT	—o
2" DIA. CONE	———	PLASTIC LIMIT	⊢—
SHEAR STRENGTH		SAMPLE TYPE	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	⊕	2" O.D. SPLIT TUBE	▨
UNCONFINED COMPRESSION	⊗	2" I.D. SHELBY TUBE	■
VANE TEST AND SENSITIVITY (S)	+S	3" O.D. SHELBY TUBE	▤

BOREHOLE No. 1

PROJECT Bridge Replacement

LOCATION Parks Creek & Co.Rd. #7, Hastings County

HOLE LOCATION South side of Creek (see Dwg.1)

HOLE ELEVATION 191.1

DATUM Top of Bridge Seat, South end (see Dwg.1)

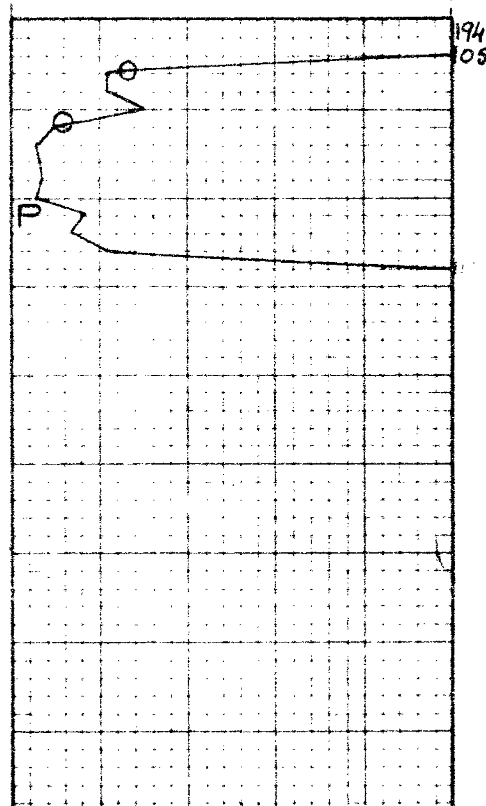
SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	CORE SIZE	RECOVERY
		191.1	0			
	FILL - greyish brown silty sand and gravel to 4.5ft., very dense. ▽	187.1		1		
	ALLUVIUM (or **FILL**) brown, silty clay, scattered trace organics & occasional sand & gravel. Firm, wet.	186.7		2		
	CLAY - silty; stiff, grey	181.6	10	3		
	SAND & GRAVEL - very hard	178.1		4		
	LIMESTONE - well bedded, hard, grey. Water loss in bedding plane at 18'9"	175.1			AX	83%
			20		AX	81%
					AX	100%
	Terminated	165.0			AX	30% *
			30			
			40			

Core Size Recovery

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT — 20, 40, 60, 80

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS — % DRY WEIGHT

NATURAL UNIT WEIGHT P.C.F.



Notes:

1. Hole advanced by wash boring and rotary core drilling. Cased to 16ft.
2. Cone located 5ft. south.
3. Water encountered at 4ft.

* Core lost from core-barrel when pulling out.

Borehole 1

WILLIAM TROW ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No 3

PROJECT No H108

BOREHOLE No 2

PROJECT Bridge Replacement

LOCATION Parks Creek & Co.Rd. #7, Hastings County

HOLE LOCATION North side of Creek (see Dwg.1)

HOLE ELEVATION 193.8

DATUM

LEGEND

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

2" DIA CONE

SHEAR STRENGTH

UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕

UNCONFINED COMPRESSION ⊗

VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT

PLASTIC LIMIT

SAMPLE TYPE

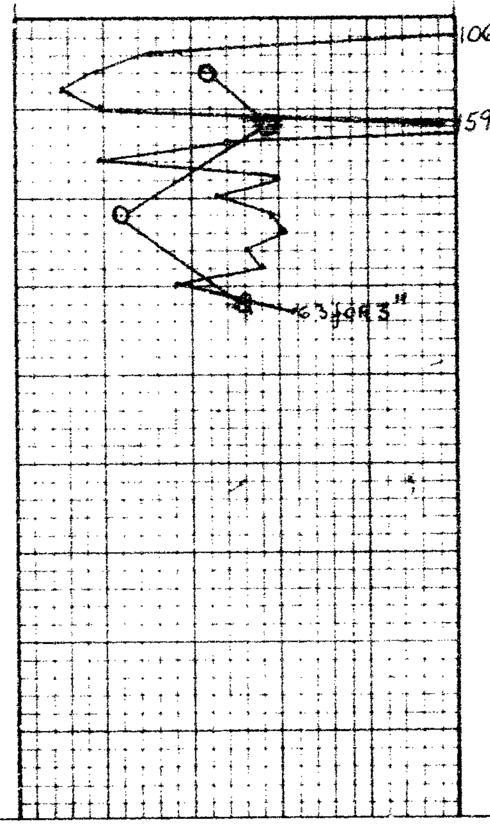
2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
		193.8	0		
	FILL - brown sand & gravel, some cohesive silt, compact to very dense, wet at 6ft.			1	
	∇	187.8		2	-
	ORGANIC TOPSOIL	186.3			
	SAND & GRAVEL - v.dense, silty, grey (glacial till)	185.8	10	3	
		179.3			
	CLAYEY SILT - grey, v.stiff	177.0		4	
	LIMESTONE - grey, hard, crystalline, well bedded. Water loss at 18'9"		20	AX	95%
	Terminated	172.0		Core Size	Recovery
			30		
			40		

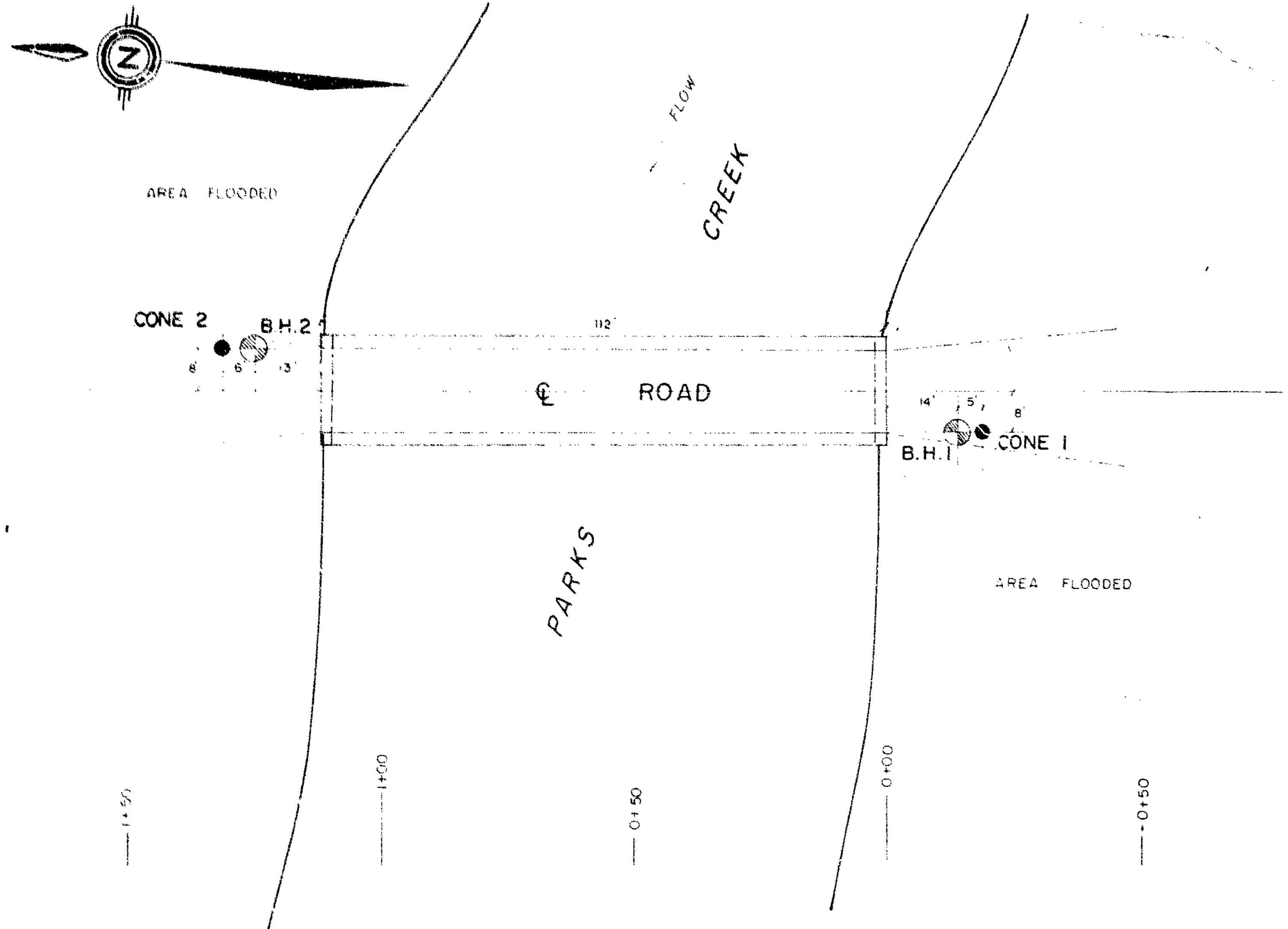
PENETRATION RESISTANCE 350 FT. LB BLOWS/FT 20 40 60 80 — SHEAR STRENGTH P.S.F.



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

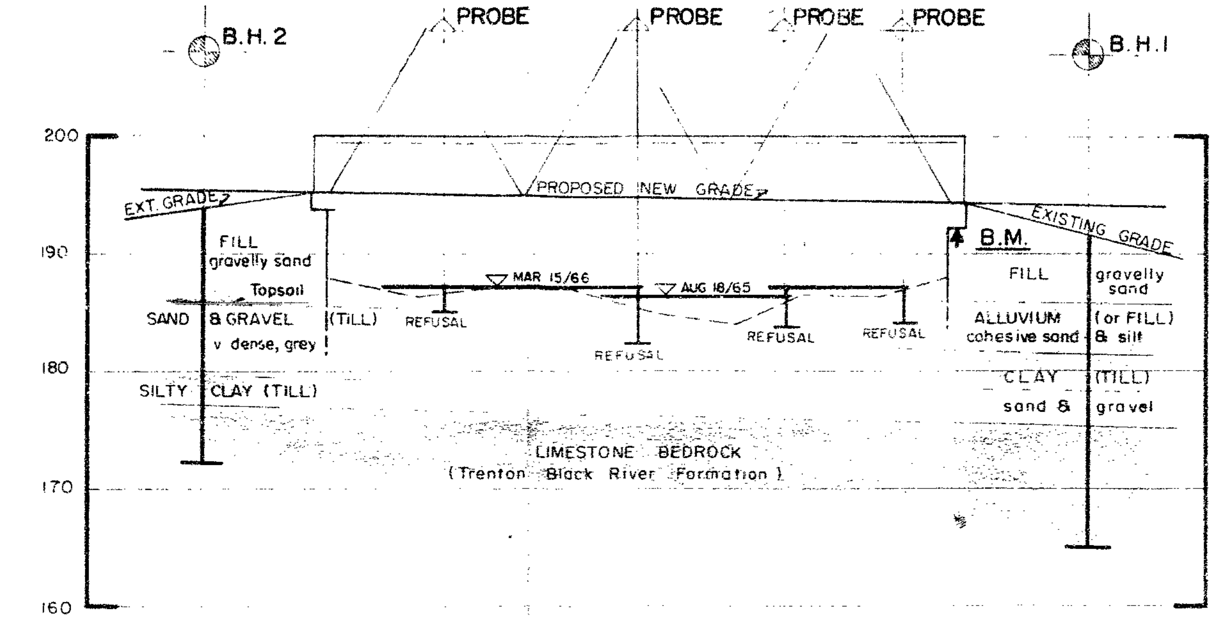
Notes:

1. Hole advanced by wash boring and rotary core drilling. Cased to 16ft. 9 inches.
2. W.L. 6ft., approximately same as creek level.
3. Cone located 6ft. north.



PLAN

SCALE 1 in. = 20 ft.



INTERPRETED SUBSOIL STRATIGRAPHY

B.M. Elev. 192·08

Top of bridge seat, south abutment

— NOTE —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

- NOTE -

Samples will be kept for 3 months from the date of this report unless otherwise directed.

William Trow Associates Ltd.

FOUNDATION INVESTIGATION

BRIDGE REPLACEMENT

COUNTY ROAD NO. 7 OVER PARKS CREEK

LOTS 5 & 6 CON. VI TWP. TYENDINAGA

COUNTY HASTINGS

PROJ. H 108	DATE MAR. 1966	DWG. Nº 1