APPENDIX G – REPORT ON SOILS INVESTIAGTION

R+S # 200/75

July 24, 1973

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TITLE: REPORT ON SOILS INVESTIGATION
File: OSBORNE STREET BRIDGE

LOCATION: WINNIPEG, MANITOBA

CLIENT: REID CROWTHER & PARTNERS

JOB NO: W-823

DATE: July 24, 1973

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July 24, 1973

Our File W-823

REPORT ON
SOILS INVESTIGATION
OSBORNE STREET BRIDGE
WINNIPEG, MANITOBA

FOR

REID CROWTHER & PARTNERS

BY

RIPLEY, KLOHN & LEONOFF INTERNATIONAL LTD.
CONSULTING GEOTECHNICAL ENGINEERS
WINNIPEG, MANITOBA

1. INTRODUCTION

This report gives the results of a soils investigation carried out for the proposed new Osborne Street Bridge which will cross the Assiniboine River in Winnipeg, Manitoba.

Conclusions and recommendations are given herein concerning foundation support for the new structure and stability of the riverbank in addition to other pertinent items.

This investigation was authorized by Mr. G. Langman of Reid Crowther & Partners in his letter dated the 11th of May, 1973.

2. CONCLUSIONS & RECOMMENDATIONS

2.1 Riverbank Stability

The bridge abutments should be placed behind a slope of 6 horizontal to 1 vertical from the toes of the banks. This will eliminate the necessity of placing fill on the bank for approaches which would lower the factor of safety of the bank to less than 1.5 using a shear strength of 500 lbs per sq ft.

The banks should be graded back from summer water level to the abutment locations so that lower bank movements will not affect nearby piers. Slope protection will be required on these graded banks from winter water level to flood water level.

Every attempt should be made to minimize the amount of clearing done around the site to facilitate construction operations, as in our opinion, the trees in the area tend to contribute to the stability of the banks.

We should be given the opportunity to review the final grading plan around the banks prior to tender.

2.2. <u>Foundation Support</u>

The limestone bedrock formation is recommended as the bearing strata for foundation support of piers and abutments.

Because of the possible occurrence of shattered rock and clay/silt infilled crevices or cavities in the bedrock, we recommend a maximum design bearing pressure of 15,000 lb per sq ft for direct bearing of piers placed a minimum of 12 inches below the bedrock surface. Driven end-bearing piles should be spaced so that the recommended design pressure of 15,000 lb per sq ft is not exceeded at the level of the pile tips.

We recommend the use of steel driven piles to achieve some penetration of the bedrock formation. Strengthening of the tips of piles may be necessary to achieve penetration. Piles should be driven to practical refusal with a hammer capable of delivering at least 20,000 ft lb per blow.

Measures should be taken to resist corrosion of steel piles.

[illegible] bored cast-in-place concrete pile (caisson) [illegible] may also be used for foundation support. It is recommended that the design bearing pressure for caissons placed a minimum of 12 inches into sound rock not exceed 60,000 lb per sq ft. Socket friction may also be combined with end-bearing if the caisson is socketed into sound rock. A design friction value not exceeding 150 lb per sq in is recommended.

We recommend that river piers be founded directly on bedrock. The choice between direct bearing on bedrock and driven end-bearing piles or caissons for riverbank piers is a matter of economics. End-bearing piles or caissons are recommended for abutment support.

Foundations will have to be designed to resist any horizontal loads that may be acting.

The foundation installation must be inspected by qualified geotechnical personnel to assess the bedrock surface for direct bearing footings, to inspect the driving of driven piles, or to inspect the rock for caisson installations. For direct bearing piers or caissons, the bearing surface should be drilled by several test holes in order that a proper assessment of the rock can be made. The minimum depth of these test holes should be 6 feet below the bearing surface. Adjustments in design bearing pressures or excavation to greater depths may be required as a result of the inspection of bearing surfaces.

Foundations, excavated or socketed into rock, should not be considered as capable of acting like anchors.

2.3. Construction

Excavation for piers and abutments in the riverbanks and in the river should be done along neat lines and tightly shored or cofferdammed. Shoring diagrams for riverbank excavations are given in Appendix D.

Measures should be taken to ensure that excavations are completely dewatered.

Shoring or cofferdam designs or any other method proposed for excavation and dewatering on this site must be reviewed by a qualified geotechnical engineer prior to award of contract.

No temporary fill or stockpiles of materials should be allowed on the riverbank forward of a hypothetical slope of 4 horizontal to 1 vertical eminating back from the toes of the riverbanks.

Due to the possible non-conformities that can be encountered in the bedrock, we anticipate that there could be construction problems with the installation of caissons as follows:

a) Caving alluvial soils encountered while drilling down to bedrock.
b) Excavation in decomposed rock and in clay/silt.
c) Heavy water inflow that may necessitate the use of special procedures to pour concrete.

2.4. Backfill

Backfill between piers/abutments and the sides of excavations should be placed and compacted prior to removal of shoring support. Backfill should be clean granular material compacted to a minimum of 97% of Standard Proctor Maximum Dry Density. The abutment backfill should be drained. The lateral earth pressure on the abutment may be computed using a fluid pressure distribution with a unit weight of 40 lb per cu ft.

The top of the backfill around piers and on the river side of abutments should be sealed with 2 feet of compacted clay.

2.5. Sulphates

All concrete in contact with soils should be made with Sulphate Resisting Cement.

2.6. Future Work

We recommend that a caisson test be carried out in order to assess the problems that may be encountered during the construction of caissons through the alluvial riverbank soils.

Detailed soundings of the river bottom should be carried out over the right-of-way of the bridge and included in the tender documents as required information for the method of installing river piers.

A minimum of 1 test hole should be drilled at each abutment as their locations behind a hypothetical slope of 6 horizontal to 1 vertical places them at some distance from our closest test holes. Basically, these holes would be drilled to assess the quality of overburden soils directly below the abutments as these may differ from those encountered in the other test holes.

2.7. Demolition

As sections of the new bridge will be constructed prior to demolishing the old bridge, it is necessary to ensure that demolition methods do not endanger the new structure. In particular, any techniques that use blasting must be carefully reviewed by both the structural and geotechnical engineer to assess their feasibility and to review the manner in which they would be conducted.

3. DESCRIPTION OF SITE

The existing Osborne Street Bridge is a four span structure with the distance from abutment to abutment in the order of 420 feet. The north approach fill to the abutment has a maximum height of approximately 10 feet, whereas on the south bank, the approach fill is not as high and is in the order of about 5 or 6 feet.

The riverbanks on both sides are covered with fairly large trees. The north bank is steep at the river's edge and then it flattens out to form a ledge for some distance back from the river where it steepened again to the top of the bank. The average existing slope of the north bank is approximately 6 horizontal to 1 vertical.

The south bank rises fairly steeply from the river to an elevation not too much lower than the top of the bank and the average slope is about 2½ horizontal to 1 vertical.

There are visual signs of instability on the north bank just downstream from the existing bridge which take the form of old landslide scars and ledges. There are no signs of instability on the upstream side of the existing bridge where the new bridge will be located, but such signs may have been masked by the construction activity for the existing bridge. On the south bank, upstream from the existing bridge, there [illegible] the edge of the river.

Other pertinent data with regard to the site is as follows:

a)	Approximate water width at summer water level:	205 ft.
b)	Approximate width at the top of the banks:	500 ft.
c)	The approximate elevation of summer water level:	735 ft. geodetic
d)	The approximate elevation of winter water level:	727 ft. geodetic
e)	The approximate elevation of the top of the north bank:	759 ft. geodetic
f)	The approximate elevation of the top of the south bank:	757 ft. geodetic

4. DESCRIPTION OF NEW BRIDGE

It is proposed to construct the new bridge immediately upstream of the existing bridge. The new bridge will be precast-prestressed concrete with an overall width of 90 feet and the elevation of the new bridge will approximately be the same as the old. A three span or four span structure is under consideration with a curving alignment to meet the existing approach streets.

It is proposed to build and open one-half of the width of the new structure prior to demolishing the existing bridge in order to alleviate traffic conjestion.

5. FIELD AND LABORATORY INVESTIGATION

A total of 5 test holes were drilled using a skid mounted diamond drill rig. Two test holes were located on each bank and one was drilled from the existing bridge. The locations of the test holes are shown on the sketch of the site at Appendix A.

Overburden soils were sampled using shelby tubes and split spoon samplers. Cores were recovered from the bedrock. Standpipe piezometers were installed in some of the test holes. We also made one line of soundings of the river bottom along the upstream side of the existing bridge.

Samples were visually classified in our laboratory and were tested for unconfined compression, consolidation properties, natural moisture content, grain size, and liquid limit.

6. SUBSURFACE DATA

Details of the soils encountered in the test holes are presented in the test hole logs at Appendix B.

Essentially, three soil types were observed in the test holes drilled. These were:

a) Alluvium (clays and silty clays)-

- this was found in all test holes drilled and the formation extended down to bedrock in Test Holes 101, 103 and 105. The alluvium was generally highly plastic with the upper 10 feet or so precompressed by dessication. Strengths generally decreased with depth down to elevation 730 ft geodetic below which they exhibited an increasing trend except in Test Hole 104. It is believed that in Test Holes 102 and 104, glacial Lake Agassiz Clay separated the alluvium and bedrock. Sand and silt seams were found in the alluvium down to about elevation 725 ft geodetic.

b) Lake Agassiz Clay -

- this deposit was found in Test Holes 102 and 104 at about elevation 715 ft and 725 ft geodetic respectively. This clay is highly plastic and appears to be somewhat weaker than the alluvial clay and silt deposits at the same elevation.

c) Bedrock -

- limestone, at elevation 710 which was observed to exist in various physical states in the holes drilled. For example, the rock appeared to become softer towards the end of Test Hole 102, it was found to be shattered in the top 5 to 10 ft in Test Hole 105, and it was found to contain layers of stiff clay or dense silt in Test Hole 103.

It is normal to find a glacial till formation directly overlying limestone in the Winnipeg area. However, we found that this formation was practically non-existent in the test holes drilled.

We have taken a few readings of the installed piezometers since the time of their installation and the results of our observations are given in the Table below. It should be noted that piezometers are reflecting water levels in the bedrock whereas some are reflecting water levels in the overburden. The stand-pipe type of piezometer requires the movement of a large volume of water before closely registering the change in soil pore water pressures. Therefore these piezometers cannot be considered to be accurately recording the water pressures in a soil of low permeability on the date of observation, as they are lagging behind the time of the actual pore pressure change.

PIEZOMETER	TIP ELEVATION (ft)	DATE	WATER ELEVATION (ft)	REMARKS
P 201	736	May 23/73	739	
		May 24/73	738	
		May 28/73	737	
		June 18/73	736	No Water
		July 27/73		No Water
P 202	725	May 24/73	738	
		May 28/73	738	
		June 18/73	738	
		July 27/73	736	
P 203	701	May 31/73	731	
		June 18/73	725	In rock
		July 27/73	721	In rock
P 204	710	June 18/73	724	Just above bedrock
		July 27/73	720	Just above bedrock

7. DISCUSSION

7.1. Riverbank Stability

The following are the major points to be considered with regard to the development of this site:

a) Stability of the riverbanks,

b) The lack of competent soil strata overlying the bedrock formation, such as glacial till,

c) The nature of the bedrock and its bearing capacity.

The stability of the riverbank was analysed using the so-called $\emptyset = 0$ method. The problem with respect to the use of this method is the selection of the correct value of undrained shear strength considered to be acting on the potential failure plain. The following points were considered prior to selection of this shear strength:

a) The lowest value of undrained shear strength from strength tests was in the order of 600 lb per sq ft, although the shear strength ranged as high as 1500 lb per sq ft.

b) An analysis of the old failure slips immediately downstream of the north abutment of the existing bridge gave a shear stress of about 500 lb per sq ft. It is thought that these slips are at a factor of safety slightly greater than unity as there are no obvious signs of recent movements.

c) Crack patterns in the walls of the north and south abutments as well as the observation that the lower chords of the steel trusses of the old bridge have penetrated approximately 4 to 6 inches into the north abutment wall, indicate that the abutments have moved towards the river. These movements may be the result of high lateral earth pressures of the abutment backfill and/or deep seated bank instability beneath the abutments. We are concerned about the possibility that the high forces required to push the steel chords into the wall result from bank movements and that the bank is not as stable around the old bridge as it appears.

d) Local practice, for structures on riverbanks in the Winnipeg area, calls for setting back structures from the toe of the bank for such a distance that the factor of safety of the intervening bank is not less than 1.5 using a shear strength of 500 lb per sq ft. This practice has evolved from the fact that the undrained shear strengths determined by tests in the laboratory do not accurately reflect the long term strength prevailing in the Winnipeg riverbanks. Many failures have been observed where computed stresses were far lower than the undrained shear strength obtained by tests, ie, the factor of safety based on laboratory strengths was much greater than unity at failure which is not possible.

Based on the above considerations, we feel that the riverbanks in question should have a factor of safety of 1.5 using a shear strength of 500 lb per sq ft in order to reduce the risk of bank instability affecting the bridge structure.

The results of our stability analyses on the existing banks using the above criteria are given in the following table (see Cross-Sections of the banks at Appendix C).

EXISTING BANK STABILITY (Ø = O METHOD)

	Bank Height (ft)	Slope Angle (^{o})	Shear Stress (psf)	Factor of Safety *
North Bank	40	9.8^{o}	334	1.5
Toe of North Bank	26	24.5^{o}	415	1.2
South Bank	34	24.0^{o}	541	<1.0

* Shear Strength = 500 lbs per sq ft.

Neither the existing north and south banks have a factor of safety in excess of 1.5 using a shear strength of 500 lb per sq ft. The placement of approach [illegible] will therefore reduce these factors of safety well below 1.5.

The abutments should then be set back a sufficient distance so that approach fills are not required.

Furthermore, the analyses show that there is an apparently high risk of failure at the toes of the banks because of their steep slopes. These potential failures could cause movement of piers located nearby. Therefore banks should be graded back to the abutment locations in order to reduce this risk and to increase overall stability.

7.2. Foundations

Due to the lack of competent strata immediately overlying the bedrock formation, driven end-bearing piles can be expected to refuse directly in bedrock. As there is the possibility of some lateral creep forces developing on piles driven through the riverbank as well as lateral forces exerted by live loads, we feel that there should be some penetration of piles into the bedrock to provide lateral resistance at the pile tip. This penetration cannot be achieved with a driven concrete pile and this type of pile is therefore not recommended. This leads us to the use of a steel driven pile or alternatively, a large diameter bored cast-in-place pile (caisson) excavated into rock.

The quality of the decomposed rock and the silts and clays found in rock cavities and layers are the ruling factor with regard to the bearing capacity of the bedrock. Although the silt and clay was observed only in Test Hole 103, we find in our experience that the quality of bedrock may be quite erratic over a particular site and therefore we should not rule out the possibility of occurrences of this silt/clay material elsewhere. Therefore the design bearing pressures of a pier bearing directly on the bedrock surface will be low compared to that used for a considerable depth of sound rock. The design bearing pressure should not be exceeded by the average bearing pressure at the level of the tips of a group of driven end-bearing piles.

Caissons, which are designed for high bearing pressures, will have to be excavated through the decomposed rock layers to reach sound limestone bedrock. [illegible] to Test Hole 103, a caisson may have to be excavated through [illegible] rock, silts and clays in conditions of high water inflows under a substantial head.

The thickness of the overburden on the bedrock in the river was observed to be as thin as 7 or 8 feet as recorded by our soundings. It is possible that this overburden could in fact be thinner. The thickness and strength of this overburden material has to be taken into consideration for the design of coffer-dams required for the construction of river piers directly on bedrock in the dry. The design of cofferdams and the method of excavating and dewatering for piers in the river should be carefully reviewed by a Geotechnical Consultant prior to acceptance of Tender.

Yours very truly,
RIPLEY, KLOHN & LEONOFF
INTERNATIONAL LTD.

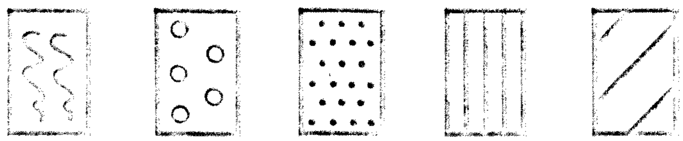
PROVINCE OF BRITISH COLUMBIA
R. J.
RENNIE
REGISTERED ENGINEER

ROBERT J. RENNIE, P. ENG.
EXECUTIVE ENGINEER

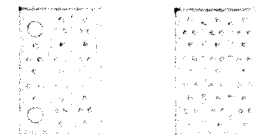
RJR/el

SYMBOLS AND TERMS USED IN THE REPORT

SYMBOLS



The symbols may be combined to denote various soil combinations, the predominant soil being heavier.



CLASSIFICATION BY PARTICLE SIZE

Boulders—larger than 8 inches
Cobbles—3 inches to 8 inches
Gravel—#4 sieve to 3 inches
Sand—#200 sieve to #4 sieve
Silt—0.002 mm. to #200 sieve
Clay—finer than 0.002 mm.

DENSITY OF SANDS AND GRAVELS

Descriptive Term	Relative Density	Standard Penetration Test
Very loose	0 - 20%	0 - 4 blows per ft.
Loose	20 - 40%	4 - 10 blows per ft.
Medium dense	40 - 70%	10 - 30 blows per ft.
Dense	70 - 90%	30 - 50 blows per ft.
Very dense	90 - 100%	Over 50 blows per ft.

NOTES

1. Relative density determined by laboratory tests.
2. Standard Penetration Test uses 140 lb. weight, 30 inch drop, 2" O.D. sampler.
3. The "R.K.L." Penetration Test uses 50 lb. weight, 30 inch drop, 1¼" O.D. drive cone attached to a single line of 1" diameter rods. The penetration diagram is a measure of skin friction plus point resistance. An approximate relationship between the Standard Penetration Test and the "R.K.L." Penetration Test exists for sands. This is shown in the following table.

Depth—Ft.	0 - 20	20 - 40	40 - 60
Std. Pen. Test / "R.K.L." Test	0.7	0.5	0.3

CONSISTENCY OF CLAYS AND SILTS

Descriptive Term	Unconfined Compressive Strength—Tons Sq. Ft.	Remarks
Very soft	less than 0.25	Can penetrate with fist
Soft	0.25 to 0.50	Can indent with fist
Firm	0.50 to 1.0	Can penetrate with thumb
Stiff	1.0 to 2.0	Can indent with thumb
Very stiff	2.0 to 4.0	Can indent with thumb-nail
Hard	4.0 and greater	Cannot indent with thumb-nail

DESCRIPTIVE SOIL TERMS

Well graded . . . . having wide range of grain sizes and substantial amounts of all intermediate sizes.

Poorly graded . . . predominantly of one grain size.

Slickensided . . . . refers to a clay that has planes that are slick and glossy in appearance; slickensides are caused by shear movements.

Sensitive . . . . . . exhibiting loss of strength on remolding.

Fissured . . . . . . containing cracks, usually attributable to shrinkage. Fissured clays are sometimes described as having a nugget structure.

Stratified . . . . . . containing layers of different soil types.

Organic . . . . containing organic matter; may be decomposed or fibrous.

Peat . . . . a fibrous mass of organic matter in various stages of decomposition. Generally dark brown to black in color and of spongy consistency.

TEST HOLE LOG

DATE May 16/17, 1973

HOLE NO. 101

SAMPLE DATA

WEIGHT HAMMER 140 lbs

HEIGHT DROP 30"

BBS2 Diamond Drill

ELEV. GROUND Rig w/wash water EL 745.3

CO-ORD. LOCATION See Site Plan

Unconfined Compression Tons per Sq. Ft.

DEPTH ELEV.	O.D. I.D.	BLOWS FT.	NO.
5	3"Sy	11	1
10	3"Sy	4	2
15	3"Sy	9	3
20	3"Sy	8	4
25	3"Sy	13	5
30	3"Sy	20	6
35	S/S	13/3	7
40			

DESCRIPTION OF MATERIAL

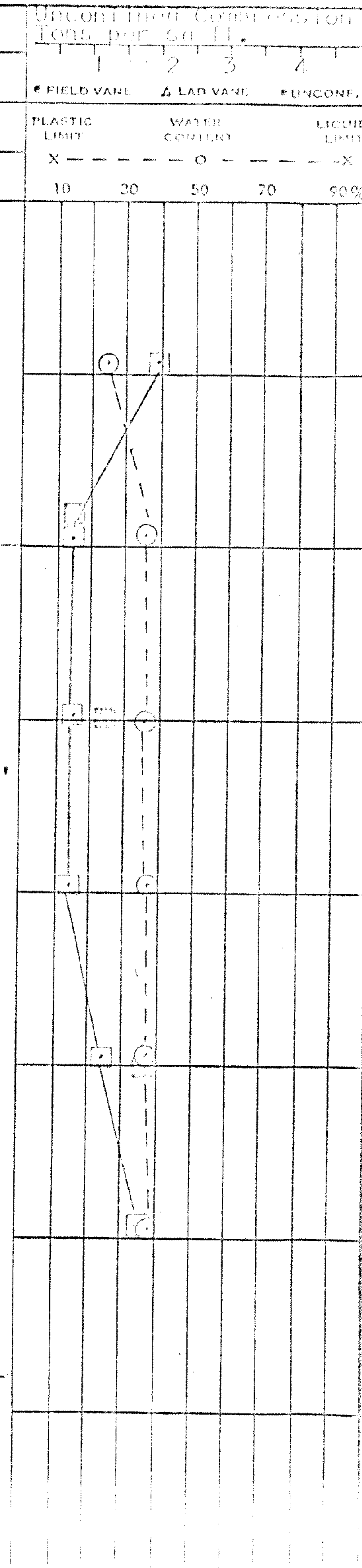
SILT - clayey, sandy
- low plasticity
- roothairs, fine sand seams
- brown
- stiff

10

CLAY - silty
- low to medium plasticity
- organic inclusions
- fine sand seams
- mottled brown/grey, grey @ 15' to end of strata
- soft to stiff

34

BEDROCK - limestone
- recovery = 99 to 100%
- hard, white



PROJECT

TEST HOLE LOG

DATE May 16/17, 1973 HOLE NO. 101

SAMPLE DATA				SYMBOL	ELEV. COLLAR	COHESION – TONS/SQ. FT. 0.2 0.6 1.0 1.4 1.8
WEIGHT HAMMER					ELEV. GROUND	● FIELD VANE △ LAB VANE ▪ UNCONF.
HEIGHT DROP					CO-ORD. LOCATION	PLASTIC LIMIT X — — — WATER CONTENT O — — — LIQUID LIMIT X
DEPTH / ELEV.	O.D. / I.D.	BLOWS / FT.	NO.		DESCRIPTION OF MATERIAL	10 30 50 70 90%
40						
45					46	
					End of hole	
50					NOTES: 1. Used "N" casing. 2. Water observations: a) casing @ 36', Hole @ 34', water loss = 7'6" during night of May 16/17, 1973. b) No water loss in rock during drilling. c) Uncased hole @ 13', water loss.	☐ Pocket Penetrometer ☐ Qu Lab Test

PROJECT

LOCATION

TEST HOLE LOG

DATE May 18-22, 1973

HOLE NO. 102

SAMPLE DATA		
WEIGHT HAMMER	140 lbs	
HEIGHT DROP	30"	

BBS2 Diamond Drill Rig w/wash water

ELEV. GROUND EL. 747.0

CO-ORD. LOCATION See Site Plan

Unconfined Compression Tons per sq ft.

1 2 3 4

● FIELD VANE △ LAB VANE ■ UNCONF.

PLASTIC LIMIT X — — — — — WATER CONTENT O — — — — — LIQUID LIMIT X

10 30 50 70 90%

DEPTH / ELEV.	O.D. / I.D.	BLOWS / FT.	NO.
			1
5	3"Sy	22	
			2
	3"Sy	36	
			3
	3"Sy	7	
10			
			4
	3"Sy	15	
15			5
	3"Sy	17	
			6
	3"Sy	17	
			7
20	3"Sy	16	
25			8
	3"Sy	29	
30			9
	3"Sy	35	
35			
46			

DESCRIPTION OF MATERIAL

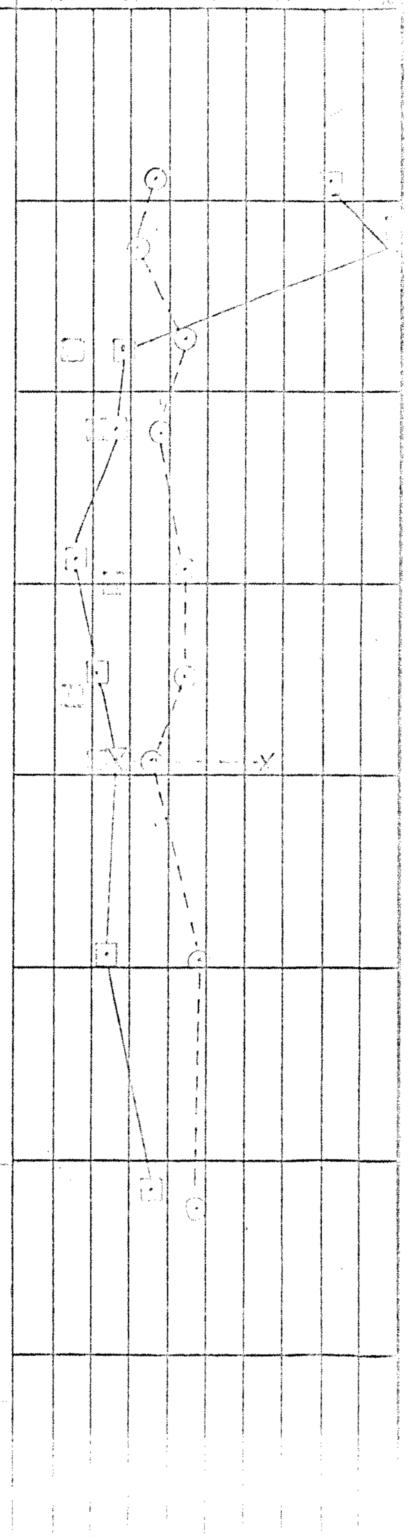
CLAY – silty
- low to medium plasticity
- organic inclusions
- thin silt and sand seams
- brown to 14', mottled brown/ grey to 18' then grey

30

CLAY – high plasticity
- small silt inclusions
- mottled brown
- stiff

36

BEDROCK – limestone
- 99% recovery to 46½' then 90% recovery
- white hard



Ripley, Klohn & Leonoff International Ltd.

PROJECT

LOCATION

TEST HOLE LOG

DATE ______ HOLE NO. 102

SAMPLE DATA				SYMBOL	ELEV. COLLAR / ELEV. GROUND / CO-ORD. LOCATION	COHESION – TONS/SQ. FT. 0.2 0.6 1.0 1.4 1.8 ● FIELD VANE △ LAB VANE ✱ UNCONF. PLASTIC LIMIT X – – – WATER CONTENT O – – – LIQUID LIMIT X 10 30 50 70 90%
WEIGHT HAMMER						
HEIGHT DROP						
DEPTH / ELEV.	O.D. / I.D.	BLOWS / FT.	NO.		DESCRIPTION OF MATERIAL	
40						
45						
50					50 End of hole	☐ Pocket Penetrometer
55						☒ Qu by Lab Test

End of hole

NOTES:

1. Used "H" casing.
2. Water observations:
   a) Water loss @ 3'.
   b) Casing @ 36' Hole @ 50', water loss = 10' overnight.
3. Piezometers installed at 11' and 22'.

TEST HOLE LOG

DATE May 16, 1973 HOLE NO. 103

SAMPLE DATA	
WEIGHT HAMMER	140 lbs
HEIGHT DROP	30"

BBS2 Diamond Drill Rig w/wash water

ELEV. GROUND EL. 744.9

CO-ORD. LOCATION See Site Plan

DEPTH ELEV.	O.D. I.D.	BLOWS FT.	NO.
			1
5	3"Sy	0	
	3"Sy	13	2
			3
10	3"Sy	15	
			4
	3"Sy	10	
15			5
	3"Sy	11	
20			6
	3"Sy	9	
25			7
	3"Sy	11	
30			8
	3"Sy	34	
35			
40			

DESCRIPTION OF MATERIAL

CLAY – silty
- low to medium plasticity
- black organic intrusions, roothairs
- thin horizontal silt and sand seams to 25'
- dark brown, mottled brown/grey from 12' to 20', grey @ 20 to end of strata
- firm to stiff
- possible till from 31½ to 33'

33

BEDROCK – limestone
- core recovery:
  a) 70% 36'6" to 40'
  b) 25% 40' to 45'
  c) Hole caving @ 40'
  d) 33% 46' to 50'6"
- hard rock with small vug holes from 33'6" to 40'

Unconfined Compression Tons per sq. ft.

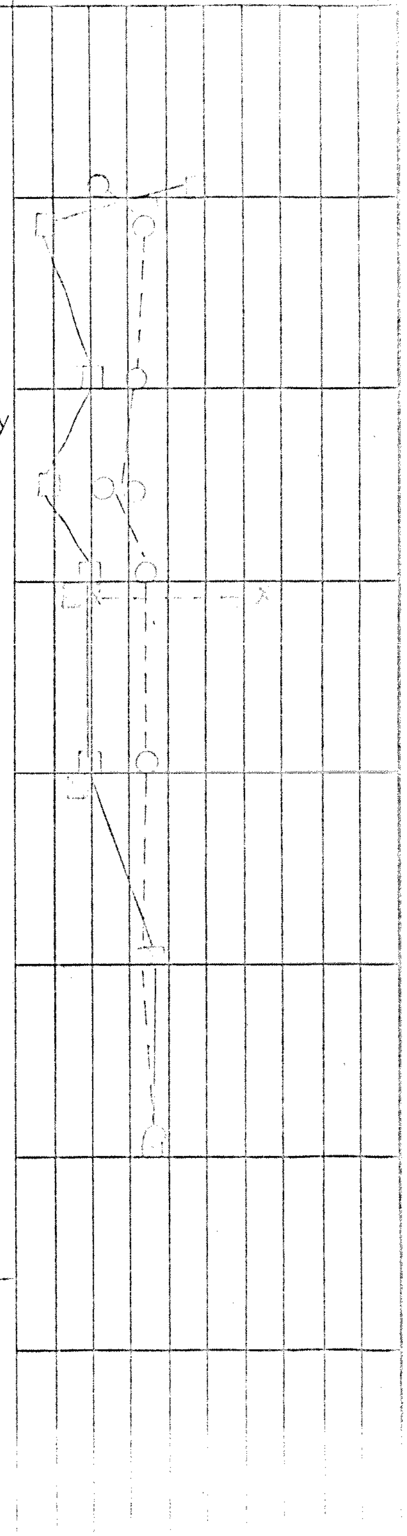
1 2 3 4

FIELD VANE LAB VANE UNCONF.

PLASTIC LIMIT WATER CONTENT LIQUID LIMIT

X — — — — O — — — — X

10 20 30 50 70 90%



Ripley, Klohn & Leonoff International Ltd.

PROJECT

TEST HOLE LOG

DATE May 28/29/30, 1973

HOLE NO. 103

SAMPLE DATA				SYMBOL	ELEV. COLLAR	COHESION - TONS/SQ. FT.
WEIGHT HAMMER					ELEV. GROUND	0.2 0.6 1.0 1.4 1.8 — ● FIELD VANE △ LAB VANE ● UNCONF.
HEIGHT DROP					CO-ORD. LOCATION	PLASTIC LIMIT X — WATER CONTENT O — LIQUID LIMIT X
DEPTH / ELEV.	O.D. / I.D.	BLOWS / FT.	NO.		DESCRIPTION OF MATERIAL	10 30 50 70 90%
40						
45						
50					50'6"	
					End of hole	□ Pocket Penetrometer
55						□ Qu by Lab Test

NOTES:

1. Drilled with "H" and "A" casing.
2. Water observations:
   a) Uncased hole @ 19', water loss = 9' during night of May 28/29.
   b) "H" to 35'6", "A" to 46'4" Hole @ 46'4", water loss = 6' during night 29/30, May 1973
   c) "H" to 35'6", "A" to 45'4" Hole @ 50'6", 15 ft of water loss in 15 minutes in "A" casing after water shut down.
   d) No water loss while drilling from 34' to 46'.
3. Piezometer installed @ 43'6".

PROJECT

LOCATION

TEST HOLE LOG

DATE May 31 - June 1-4/73

HOLE NO. 103A

SAMPLE DATA			BBS2 Diamond Drill
WEIGHT HAMMER	140 lbs	ELEV. GROUND	Rig with wash water EL. 744.9
HEIGHT DROP	30"	CO-ORD. LOCATION	$4\frac{1}{2}$' south east of 103.

Unconfined Compression Tons per sq. ft. 1 2 3 4 — ○ FIELD VANE Δ LAB VANE ● PENCONE.

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 50 70 90%

DEPTH / ELEV.	O.D. / I.D.	BLOWS / FT.	NO.	DESCRIPTION OF MATERIAL
				0 - 35' No sampling
30				
35				34'6" SILT - clayey - stiff - dark green & tan - some limestone gravel - decomposed rock
	S/S	112	1	
40				38'6" LIMESTONE ROCK - tricone to 39'6" - core 39'6" to 42' - 80% recovery, hard, white - core 42' to 45'6" - 50% recovery, tan to white rock
45	S/S	>/50	2	45'6" SILT - clayey - stiff - yellow, tan - some limestone gravel
50				48 - 49 LIMESTONE ROCK - core 48' to 49' - 100% recovery, white hard

End of hole

NOTES:

1. Casing and hole @ 35', water at 15' depth morning of June 1, 1973

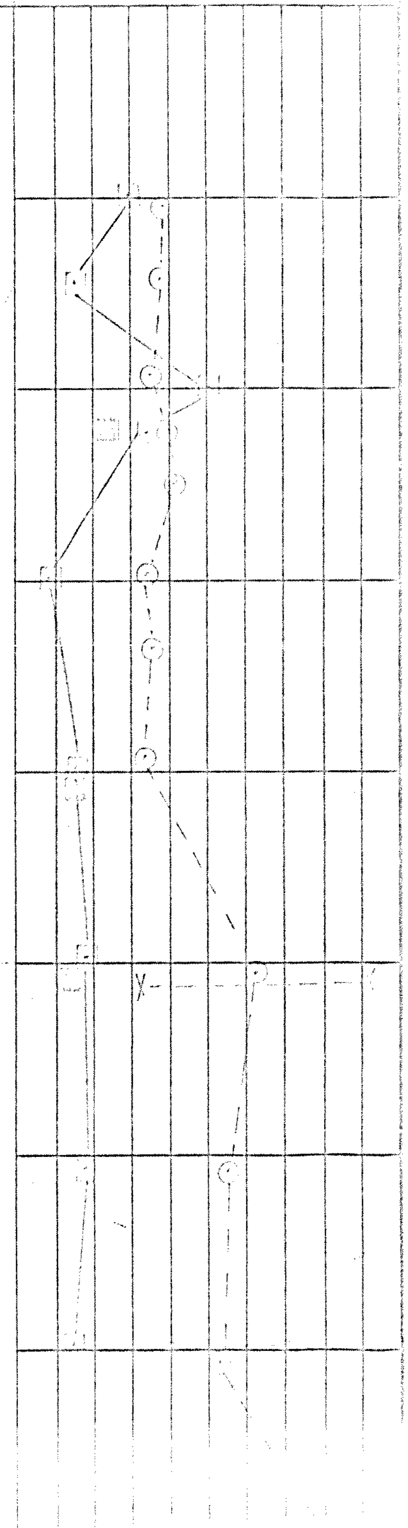
Ripley Klohn & Leonoff International Ltd.

PROJECT

TEST HOLE LOG

DATE June 5/6, 1973 HOLE NO. 104

SAMPLE DATA				SYMBOL	BBS2 Diamond Drill	Unconfined Compression Tons Per Sq. Ft.
WEIGHT HAMMER 140 lbs					ELEV. GROUND Rig w/wash water EL. 751.9	1 2 3 4 ⊙ FIELD VANE △ LAB VANE ▫ UNCONF.
HEIGHT DROP 30"					CO-ORD. LOCATION See Site Plan	PLASTIC LIMIT X — WATER CONTENT O — LIQUID LIMIT X
DEPTH / ELEV.	O.D. / I.D.	BLOWS / FT.	NO.		DESCRIPTION OF MATERIAL	10 20 50 70 90%
			1			
5	3"Sy	18	2		CLAY – silty – low to medium plastic – black organic intrusions – horizontal sand seam @ 19' – mottled brown/grey to 13', then grey to end of strata – firm to very stiff	
	3"Sy	13	3			
10	3"Sy	21	4			
	3"Sy	18	5			
15	3"Sy	10	6			
	3"Sy	11	7			
	3"Sy	11	8			
20	3"Sy	14				
25			9		25	
	3"Sy	8			CLAY – silty – highly plastic – small silt pockets – dark brown – firm – slickensides – silt/clay mixed @ 41' with some gravel.	
30			10			
	3"Sy	11				
35			11			
	3"Sy	13				
40						



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PROJECT [illegible]

LOCATION [illegible]

TEST HOLE LOG

DATE June 5/6, 1973

HOLE NO. 104

SAMPLE DATA				SYMBOL	ELEV. COLLAR
WEIGHT HAMMER					ELEV. GROUND
HEIGHT DROP					CO-ORD. LOCATION
DEPTH ELEV.	O.D. I.D.	BLOWS FT.	NO.		DESCRIPTION OF MATERIAL
40	3"Sy	44	12		
					42
45					BEDROCK - limestone - core recovery = 80%, 44'6" to 58'6" - very hard, white
50					
55					
					58'6"
60					End of hole

COHESION - TONS/SQ. FT.: 0.2, 0.6, 1.0, 1.4, 1.8

FIELD VANE, LAB VANE, UNCONF.

PLASTIC LIMIT X — WATER CONTENT O — LIQUID LIMIT X

10, 30, 50, 70, 90%

□ Pocket Penetrometer

Qu by Lab Test

NOTES:

1. Drilled with "H" and "A" casing.
2. Water observations:
   a) Uncased hole to 25', water loss = 6½' during night of June 5/6.
   b) Slight water loss @ 42'.
   c) No water loss below 42' during drilling.
3. Piezometer installed @ 41'6".

TEST HOLE LOG

DATE: June 13-15, 1973

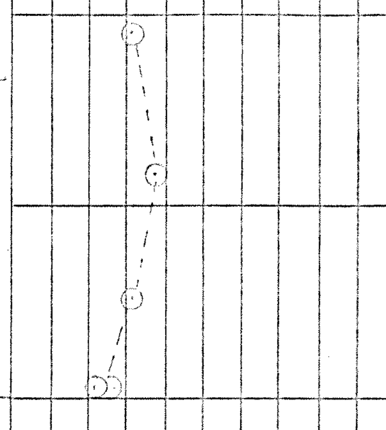
HOLE NO. 105

SAMPLE DATA			
WEIGHT HAMMER	140 lbs	ELEV. GROUND	BBS2 Diamond Drill Rig w/wash water EL. 762.4
HEIGHT DROP	30"	CO-ORD. LOCATION	Drilled from old bridge

Unconfined Compression Tons per Sq. Ft.: 1 2 3 4 (FIELD VANE, Δ LAB VANE, UNCONF.)

PLASTIC LIMIT X — WATER CONTENT O — LIQUID LIMIT X: 10 30 50 70 90%

DEPTH ELEV.	O.D. I.D.	BLOWS FT.	NO.	DESCRIPTION OF MATERIAL
				0 – 28' to river level
				0 – 41'6" to river bed
40			1	
	S/S	7		
			2	41'6" CLAY – sandy, silty – low to medium plasticity – organic inclusions
45	3"Sy	7	3	– horizontal sand seam @ 47'6", sand pocket @ 50'.
	3"Sy	19	4	– pce. of concrete @ 44' and 50'
	3"Sy	22		– grey, mottled brown/grey
50	3"Sy	21/6"	5	– firm to stiff
				50 BEDROCK – limestone
				– core recovery: 75% 52'4" to 53'4"; 80% 53'4" to 57'10"; 83% 57'10" to 60'10"; 96% 60'10" to 65'10"; 100% 65'10" to 66'6"
55				– white hard rock, 1" to 2" of reddish decomposed rock @ 53'6"
60				– water observations, "H" casing @ 50', "A" casing advanced inside. a) Slight loss @ 50' b) Lost water @ 51' then regained.
65				c) Lost water and regained several times from 53'6" to 57'
				66'6" d) Lost permanently @ 64'6"
70				e) June 18th – depth to water in "A" casing ≐ 35'



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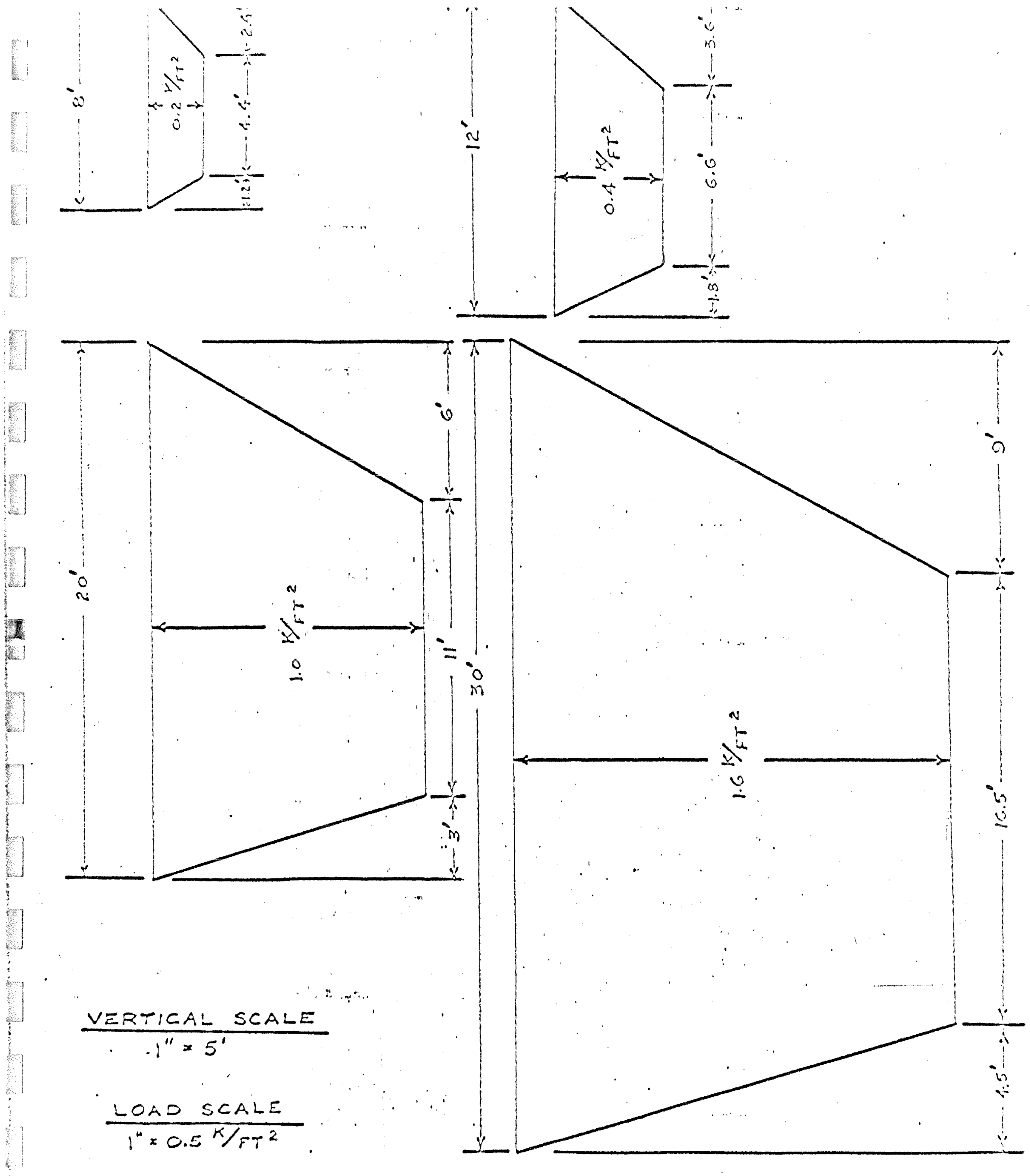
PROJECT

LOCATION



VERTICAL SCALE
.1" = 5'

LOAD SCALE
1" = 0.5 K/FT²

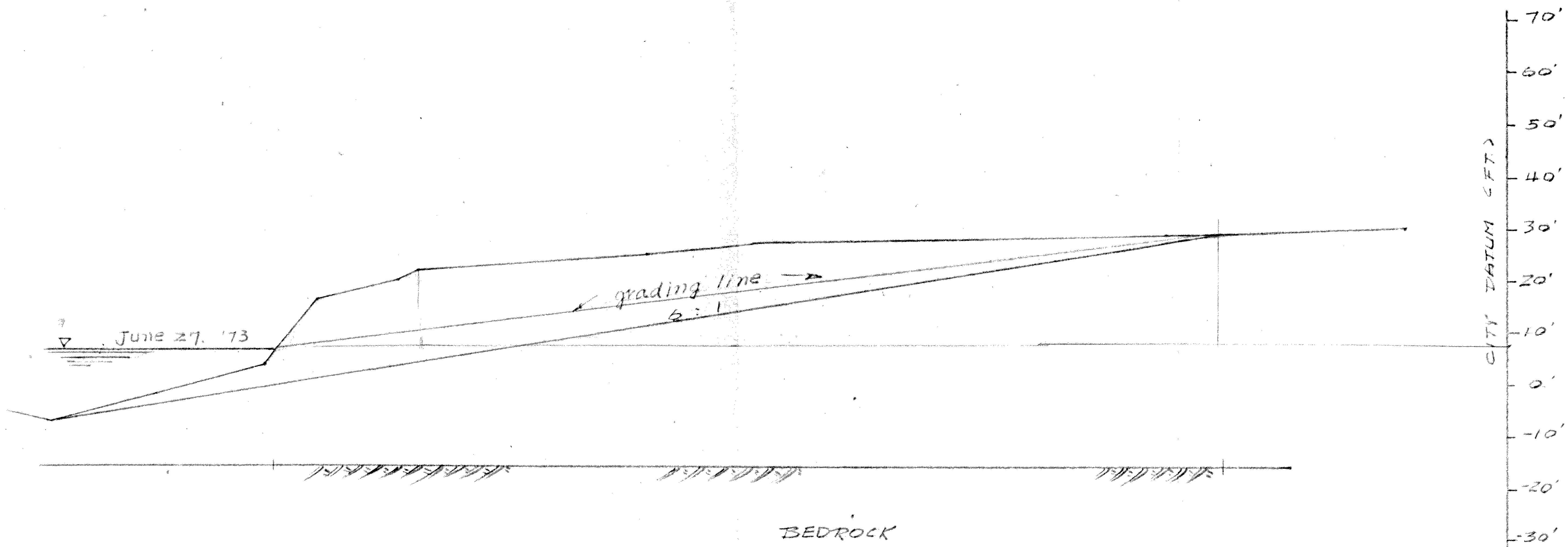
SCALE

Ripley, Klohn & Leonoff International Ltd.
CONSULTING ENGINEERS
VANCOUVER — EDMONTON — CALGARY — WINNIPEG CANADA

SHORING DIAGRAM
OSBORNE STREET BRIDGE
WINNIPEG, MANITOBA

CLIENT Reid Crowther & Partners

APPROVED R. J. Rennie | DATE 75/07/24 | R-W- 8.5



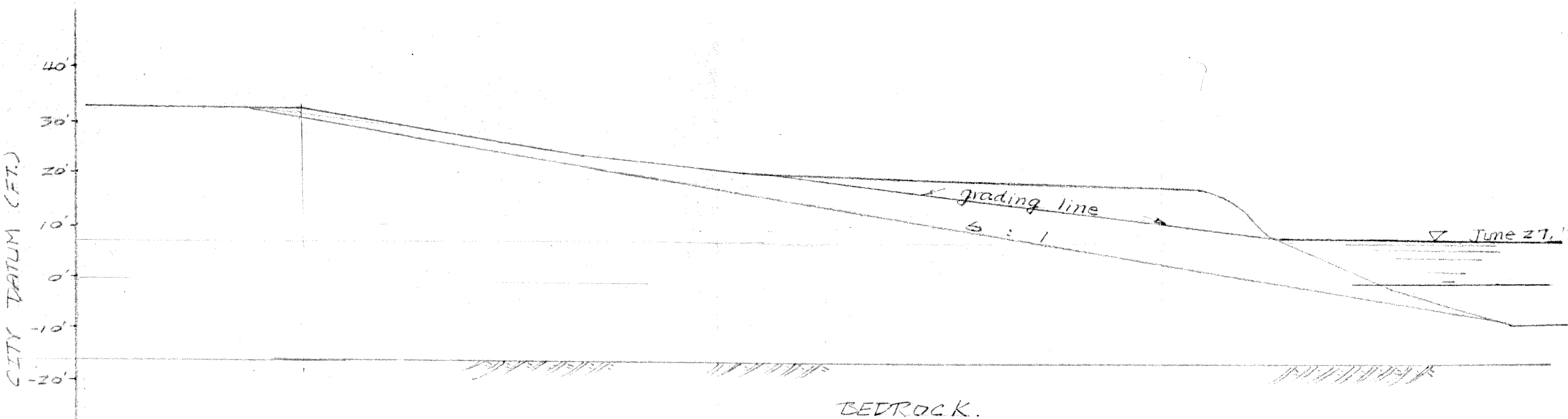
NOTES

1. For geodetic elevation, add 727.59 ft. to City Datum elevation.

2. Cross-sections drawn from elevations taken by Reid Crowther & Partners Ltd.

SCALE 1" = 20' H. & V.

Ripley, Klohn & Leonoff International Ltd. CONSULTING ENGINEERS VANCOUVER — CALGARY — WINNIPEG CANADA	CROSS SECTION OF SOUTH BANK.
CLIENT: Reid Crowther & Partners Ltd.	APPROVED: R.J.R. DATE: 26/7/73 B-W-323



NOTES

1. For geodetic elevation add 727.59 ft. to City Datum elevation.

2. Cross - section drawn from elevations taken by Reid Crowther & Partners Ltd.

SCALE 1" = 20' H & V

Ripley, Klohn & Leonoff International Ltd. CONSULTING ENGINEERS VANCOUVER — EDMONTON — CALGARY — REGINA — WINNIPEG CANADA	CROSS SECTION OF NORTH BANK		
CLIENT: Reid Crowther & Partners Ltd	APPROVED [illegible]	DATE 26/7/73	B-W-37[illegible]