

#68-F-218 M

SHOREHAM

DRIVE

BLACK CREEK

BRIDGE

BA 2948
Site 37-881



68-F-218M

FOUNDATION INVESTIGATION
SHOREHAM DRIVE
BLACK CREEK BRIDGE SITE
NORTH YORK, ONTARIO

Prepared for

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Toronto, Hamilton, Sudbury, Ottawa

PROJECT: J 4676
October 11, 1968

90 Milvan Drive,
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Foundation Investigation
Shoreham Drive
Black Creek Bridge Site
North York, Ontario

SUMMARY

This report indicates that the excavations for the bridge piers and abutments will be in stiff to hard silty clay competent to support 3 t.s.f. footing pressures safely. Settlements with the above bearing pressures will be less than 1 inch under the maximum stated loads.

The same bearing value of 3 t.s.f. may be used for the design of the abutment footings provided that they are kept at least 3 footing widths inside the existing slope as outlined in more detail in the report. A minor easterly diversion of the creek is recommended as well, to maintain the long term stability of the west valley slope.

The natural subgrade soil underlying the easterly approaches to the bridge consists of dense clayey silt till which will provide a satisfactory road base provided that it is permanently drained.

1. GENERAL

It is understood that the proposed bridge will be a multiple span pre-cast concrete structure with maximum foundation loads, on the bridge piers, of 10 - 13 t.l.f.

The bridge spans are anticipated to be in the order of 60 - 70 feet and the height of the bridge in the order of 15 - 20 feet. For aesthetic reasons it is understood that the bridge is to be kept as open as possible and therefore no fill of consequence will be installed in the approaches to the bridge.

The field work and report have been completed with the above considerations in mind. The findings and recommendations are included in the following paragraphs.

2. SITE DESCRIPTION

The site for the proposed bridge lies on the line of Shoreham Drive, spanning across Black Creek Vally, Borough of North York.

At the time of the investigation water was flowing in the 6 foot wide creek in a southerly direction through the relatively shallow valley of Black Creek. The valley is estimated to be some 450 feet wide, approximately 25 - 35 feet in height and contains a flood plain about 200 feet in width.

Probing in the creek bed indicate approximately 1 foot of alluvial material consisting of sand, gravel and boulders. The depth of this material probably is indicative of the depth of scour that has occurred in the past.

The base of the valley was grassed and open, however the valley sides were covered with small trees and shrubs.

From Dwg. 11 it can be seen that the west bank of the valley rises approximately 33 feet from the creek bottom at an angle of approximately 16° over the majority of the slope and the east bank rises 25 feet at an angle of approximately 22° .

3. FIELD WORK AND SUBSOIL

The field work at this site consisted of six boreholes sunk to depths varying from 21 to 41 1/2 feet below ground level to investigate the subsoil in the bridge area. It should be noted that it was not possible to sink borehole No. 6 as this was on the east slope of the valley side.

A further five shallow boreholes and dynamic cone penetration tests were put down to the east of the proposed bridge site to investigate the subsoil conditions on or adjacent to the approach road, Shoreham Drive. The location of the boreholes and an interpreted subsoil stratigraphy



are shown on Dwg. 1. Detailed borehole logs are included on Dwgs. 2 - 8 and the dynamic cone penetration results are summarized on Dwg. 9.

From the borehole logs it can be seen that the subsoil at the proposed bridge site consisted essentially of a lacustrine deposit of silty clay, extending to a depth of at least 41 feet below creek level (see Dwg. 3). This clay deposit is laminated with numerous pockets and partings of silt and generally has a very stiff to hard consistency. It is considered to be heavily overconsolidated. On the east side of the valley however, see Dwg. 7, a dense silt till was proved at approximately 11 feet depth underlying the very stiff silty clay and on the west side of the valley the silty clay was overlain by approximately 7 feet of fill material. Some 2 - 4 feet of alluvial sand or silt overlies the silty clay in the majority of the valley bottom.

4. GROUNDWATER

Groundwater observations are detailed in the borehole logs (Dwgs. 2 - 8), plotted on the stratigraphy Dwg. 1 and summarized below.

GROUNDWATER OBSERVATIONS

B.H. NO.	WATER STRUCK AT (FT.)	FINAL WATER LEVEL (FT.)	EL. OF FINAL WATER LEVEL (FT.)	REMARKS
1	-	29.7	577	Open to 29.7 feet after 6 days.
2	10	3.0	575.2	Open to 10.9 feet after 6 days.
3	15	4.2	574.4	Open to 4.6 feet after 1 day.
4	16	3.4	575.2	Open to 7.8 feet after 5 days.
5	-	4.2	576.0	Open to 4.2 feet after 5 days.
7	-	-	-	Dry and open to 24.6 feet after 5 days.
	-	-	575.2	Creek Level

The shallow boreholes (Boreholes 8 - 12) were not sunk deep enough to encounter groundwater.

From the above final water observation it can be seen that the water table coincides with the level of the water in the creek i.e. the water table varies from approximately El 574 - El 577 feet over the bridge site.

5) FOUNDATIONSa) Bridge Pier Footings

The bridge piers in the base of the valley may be supported on spread footings established below any loose alluvial deposits in the stiff silty clay. A safe net bearing pressure of 3 tons per square foot may be used for the design of these footings. This value is probably quite conservative for most footing locations.

This bearing pressure is governed by the shear strength of the clay and is determined from the following expression relating the allowable bearing pressure and the cohesion parameter of the clay.

$$q = \frac{CNc}{F}$$

where: C is the undrained shear strength of the clay as determined from the undrained triaxial tests shown on the borehole logs. Values of C range from 2.9 to 6.2 ksf.

Nc is a bearing capacity factor conservatively estimated equal to 7 for these site conditions.

F is the factor of safety.

It is recommended that the footings be founded at least 4 to 5 feet below present creek bed level (in the area of the stream) for adequate scour protection. If the creek is to be channelized in a positive manner a slightly higher bearing level should be permissible.

Settlements resulting from the above recommended bearing pressure and the stated maximum foundation loads will be less than 1 inch. This presumably is well within the tolerable limits for the proposed bridge.

b) Bridge Abutments

The same bearing value of 3 tsf may be applied for the design of abutment footings provided that the footings are set deep enough below the existing slope as indicated on Dwg. 11. This statement assumes that the bridge abutment will be near the top of the hill and therefore no approach fill of consequence will be incorporated in the overall valley crossing. It also assumes that the creek will be diverted slightly to the east to maintain the existing stable slope conditions. If the abutments are moved closer to the valley and a considerable amount of approach fill is placed then the long term instability of slopes must be examined and permanent slope drainage incorporated in the design.

The existing slopes are believed to be close to their geological long term stable condition (Factor of Safety = 1) i.e. they have become adjusted to the various weather conditions prevailing in the last few centuries (freezing, thawing, complete saturation, etc.). This opinion is based upon measurements of the plasticity of the clay⁺ and upon actual measurements of the existing slopes* (see Dwg. 11).

⁺ Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering Vol. 1, 1953. Gibson.

* Stability Coefficients for Earth Slopes. Geotechnique. Vol. 10 No. 4, Bishop, A.W. and Morgenstern, N. 1960.

6. EXCAVATIONS AND PERMANENT DRAINAGE

Excavation to the depth required for the construction of the bridge piers should be relatively straightforward. At the bridge pier site near the existing creek (Station 2), the slow moving brook may be diverted clear of the working area.

Any seepages entering excavations may be collected in ditches and pumped away from the area. Alternatively, interlocking sheeting may be driven into the impermeable silty clay underlying the silty alluvial valley floor cover.

The sides of the excavations may be sloped at an angle of 45 degrees in the alluvium.

It is recommended that the ground surface vegetation in the area of the abutments should be maintained as far as possible during construction. On completion of the construction, vegetation should be added to wherever possible. This will add to the protection of the existing slope and loss of soil on the slopes will be prevented.

7. SCOUR PROTECTION

Positive measures against possible scour and erosion of the river bank should be provided. Rip-rap placed on a bed of pit-run gravel will provide adequate protection. It should extend up to the highest anticipated flood level.



Because isolated scouring could take place in the backfill around the footings it is recommended that the footings be poured flush with the sides of the excavation.

8. EARTH PRESSURES

No details or exact location of the bridge abutments are known, however the following are general comments applied to similar structure.

It is assumed that permanent drainage will be installed in the fill at the back of the abutment. The earth pressure, p , at any depth, h , exerted by free-draining granular fill on the back of the abutment is given by the expression:

$$p = K (\gamma h + q)$$

where: $K = 0.25$ if a slight inward yield of the abutment is possible. This value must be increased to 0.35 if a rigid type structure is used.

$\gamma = 130$ p.c.f. the estimated unit weight of the granular backfill

$q =$ any surcharge loading

The expression assumes that there will be no water pressure behind the wall.

Since the footings will be taken to the clay the earth pressure exerted on the walls of the abutments will be resisted initially by the



sliding resistance developed along the bottom of the footing and, to a certain degree, by the passive resistance developed in front of the abutment face. This base resistance, R , is given approximately by the expression:

$$R = \frac{1}{F} (CB. + 2 CH + \frac{\gamma H^2}{2}) \text{ (plf)}$$

where: C = the undrained shear strength of the soil, conservatively assumed 2000 psf.

B = the footing width in feet

H is the depth of the clay above bearing level in front of the abutment.

γ is the unit weight of the clay

$F = 2$ is the recommended factor of safety.

From the long term viewpoint when the soil assumes a drained granular character, the base resistance, R , can be conservatively estimated from the expression:

$$R = \frac{1}{F} (C'B + 2C'H + \frac{\gamma H^2}{2} + 0.5 q B) \text{ (plf)}$$

where: 0.3 = the estimated friction coefficient between the base of the footing and the underlying drained soil

q = the footing bearing pressure on the soil

C' = effective cohesion of the soil, conservatively estimated to equal 100 psf on a long term basis.

The remaining symbols are similar to those given for the previous expression.



If the horizontal active force exceeds the base resistance, the passive earth pressure developed in front of the abutments may be increased by increasing the depth of the footings and thus the effective height, H. Alternatively, a reinforced concrete 'key' extending below the footing may be taken to the depth required to develop the necessary passive resistance.

9. SHOREHAM DRIVE EAST OF BRIDGE

The five shallow boreholes sunk to the east of the bridge site along approximately 400 feet of the proposed Shoreham Drive indicated that the subsoil was a sandy silt with some clay content. A grading of a typical sample of the till is shown on Dwg. 10. This till is sensitive to moisture change and therefore the top few inches of it will become softened during periods of thaw or prolonged wet weather. During these wet periods the strength of the soil and its capacity to support vehicular traffic will be reduced. To minimize the amount of softening, heaving, and subsequent decrease in supporting capacity a sufficient thickness of granular base must be provided to spread out traffic loads and in addition it must be kept well drained. The following procedures are recommended.

The subgrades of areas carrying traffic should be crowned with a centre-to-edge slope of at least 4 inches in 10 feet. The subgrade

should be subjected to 5 or 6 passes per square foot of a heavy roller after all underground services have been installed and backfilled and after the crowning operation has been carried out. This operation should assist in increasing the density of the backfill over the existing sewer.

Four inch farm tile, or an equivalent, should be installed in trenches along the edges of the roadway and they should be located at least 2 feet below the subgrade surface. It will thus be effective in intercepting surface runoff from the adjacent areas as well as any water being shed by the subgrade. The tile should be surrounded with at least 4 inches of pea gravel and covered top and sides with at least 12 inches of concrete sand. It must drain to a positive frost-free sump or outlet from which the water is removed.

If the above procedures are followed, it is considered that about 18 inches of granular basecourse should suffice to spread out the traffic loadings anticipated. This assumes that the basecourse will be covered by about 3 inches of asphaltic concrete. If deep lift asphalt is used the thickness of granular material could be reduced assuming an empirical ratio of 2 inches of granular equals 1 inch of asphaltic concrete. The foregoing recommendation is based upon the

attached grading curve and reference to Department of Highways of Ontario empirical rules for design of flexible pavements on frost susceptible soils.

WILLIAM TROW ASSOCIATES LIMITED

Ian W. Gore.
I.W. Gore, M.Sc.

W.A. Trow

W.A. Trow, P.Eng.

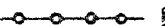
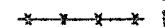
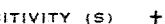

IWG/gh
Encls.

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- The Corporation of the Borough
of North York (2)
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Willowdale, Ontario.

BOREHOLE LOG

JOB No. J 4676BOREHOLE No. 1DRAWING No. 2PROJECT Proposed Bridge,LOCATION Shoreham Drive,North York.

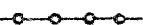
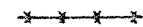
HOLE LOCATION AND DATUM SEE DRAWING No. 1

2" O.D. SPLIT TUBE 2" I.D. SHELBY TUBE 2" DIA. CONE PUSHED  PVANE TEST AND SENSITIVITY (S)  + SNATURAL MOISTURE  XPLASTIC AND LIQUID LIMIT UNDRAINED TRIAXIAL AT
OVERBURDEN PRESSURE % STRAIN AT FAILURE 



L 3 0	SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FT.	PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT.				NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT			NATURAL UNIT WEIGHT P.C.F.	
					SHEAR STRENGTH		K.S.F.						
					20	40	60	80	20	40	60		
		SILT TILL FILL-sandy silt in upper 5 ft. depth; becoming predominantly silt below 5 ft. depth; mottled, moist, cohesive, brown; small pockets of topsoil.	606.4	0	2	4	6	8	10	20	40	60	
				5									125.
			600.	10									135.
		SILTY CLAY-very stiff to hard, laminated; numerous partings and seams of compact silt; grey, moist (odd stone inclusions in upper 3 ft.)		12.9									129.
				15									134.
				20									128.
				25									126.
			577.	30									128.
		END OF BOREHOLE	575.										
NOTES: 1) Hole advanced by continuous flight auger equipment, Sept. 26, 1968. 2) Hole dry and open to 30 ft. depth on completion. 3) Water level at 29.4 ft. depth, hole open to 29.7 ft. depth after 1 day and 6 days.													



BOREHOLE LOG

JOB No. J 4676BOREHOLE No. 2DRAWING No. 3PROJECT Proposed Bridge,LOCATION Shoreham Drive,North York.2" O.D. SPLIT TUBE 2" I.D. SHELBY TUBE 

2" DIA. CONE

PUSHED VANE TEST AND SENSITIVITY (S)  + SNATURAL MOISTURE  XPLASTIC AND LIQUID LIMIT UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE % STRAIN AT FAILURE 

HOLE LOCATION AND DATUM SEE DRAWING No. 1

F & S	SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FT.	PENETRATION RESISTANCE				NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT			NATURAL UNIT WEIGHT P.C.F.			
					350 FT. LB		BLOWS/FT.								
					20	40	60	80							
					SHEAR STRENGTH				K.S.F.						
					2	4	6	8	10	10	20	30			
1/4"	[diagonal lines]	TOPSOIL-20 inches	578.2	0											
		CLAYEY SILT-alluvium with sand and gravel sizes, brown with hair roots	577.												
			575.												
			573.												
		SILTY CLAY-hard, laminated, layers, partings and pockets of grey, compact silt; grey, moist.													
		Wet below 10 ft. depth.													
		END OF BOREHOLE	536.7	40											
NOTES: 1) Hole advanced by continuous flight auger equipment, Sept. 26, 1968. 2) Hole wet and open to 11 ft. depth on completion. 3) Water level at 3 ft. depth, hole open to 10.9 ft. depth after 1 day and 5 days.					45										
					50										



William Trow Associates Ltd.






BOREHOLE LOG





JOB No. J 4676

BOREHOLE No. 3

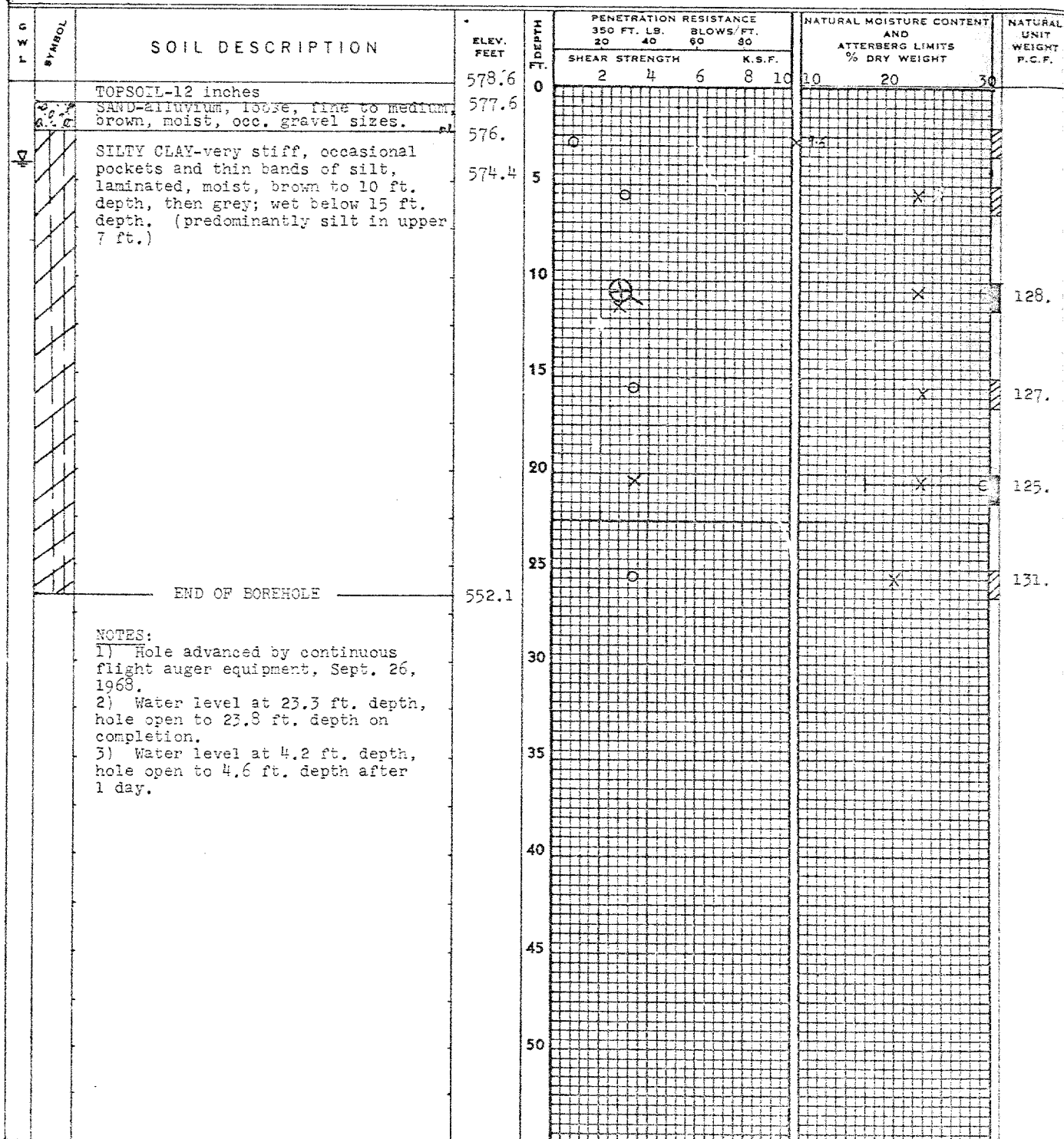
DRAWING No. 4

PROJECT Proposed Bridge,
 LOCATION Shoreham Drive,
North York.

2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 2" DIA. CONE 
 PUSHED 
 VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE 
 PLASTIC AND LIQUID LIMIT 
 UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
 % STRAIN AT FAILURE 

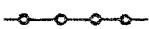
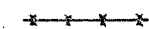


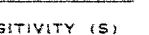
HOLE LOCATION AND DATUM SEE DRAWING NO. 1







BOREHOLE LOG

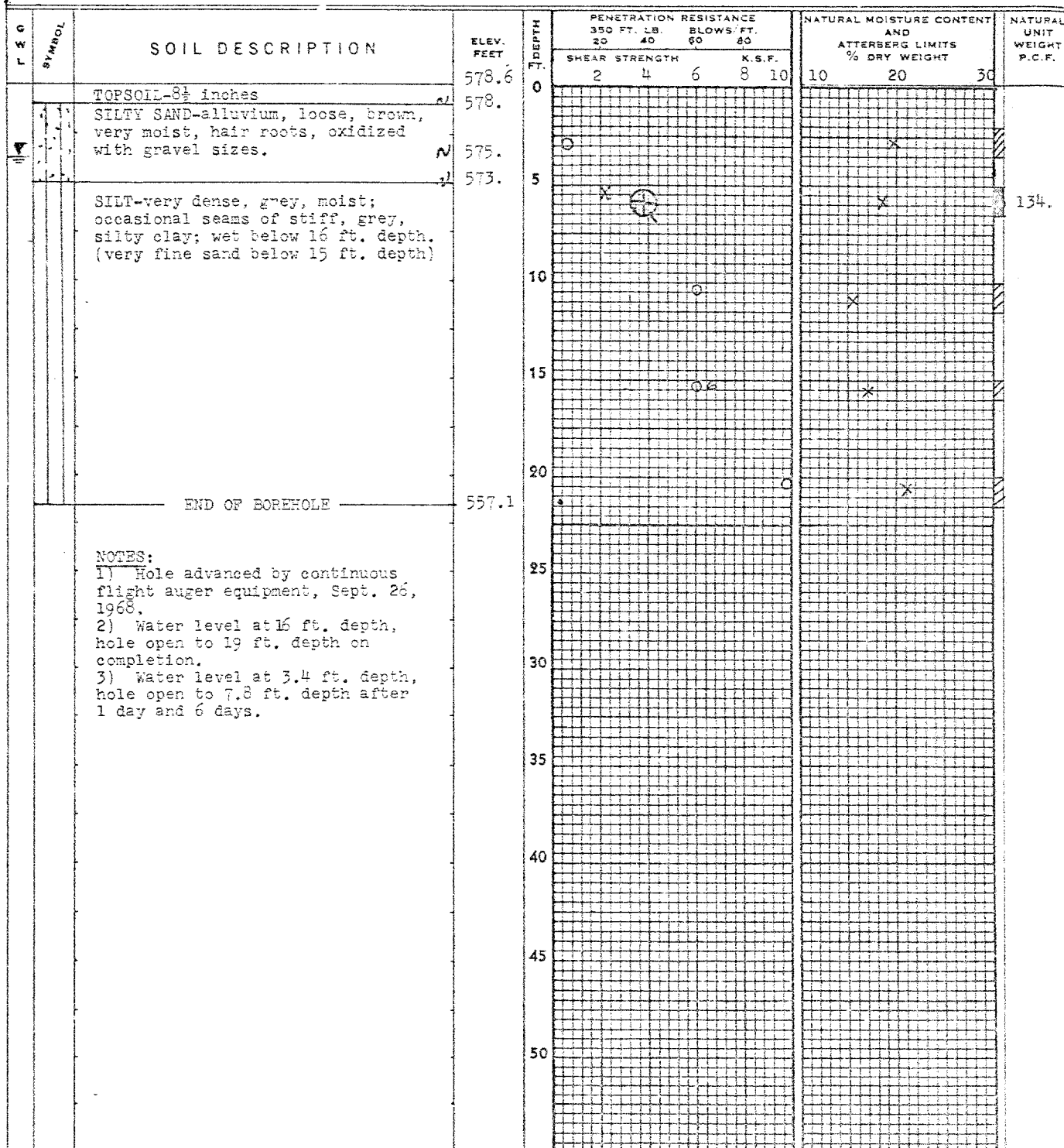
JOB No. J 4676BOREHOLE No. 2DRAWING No. 5

PROJECT Proposed Bridge,
 LOCATION Shoreham Drive,
North York.

2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 2" DIA. CONE 
 PUSHED 
 VANE TEST AND SENSITIVITY (S) 

NATURAL MOISTURE  X
 PLASTIC AND LIQUID LIMIT  O
 UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE  15
 % STRAIN AT FAILURE  10

HOLE LOCATION AND DATUM SEE DRAWING NO. 1



William Trow Associates Ltd.


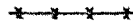


BOREHOLE LOG





JOB No. J 4676

BOREHOLE No. 5

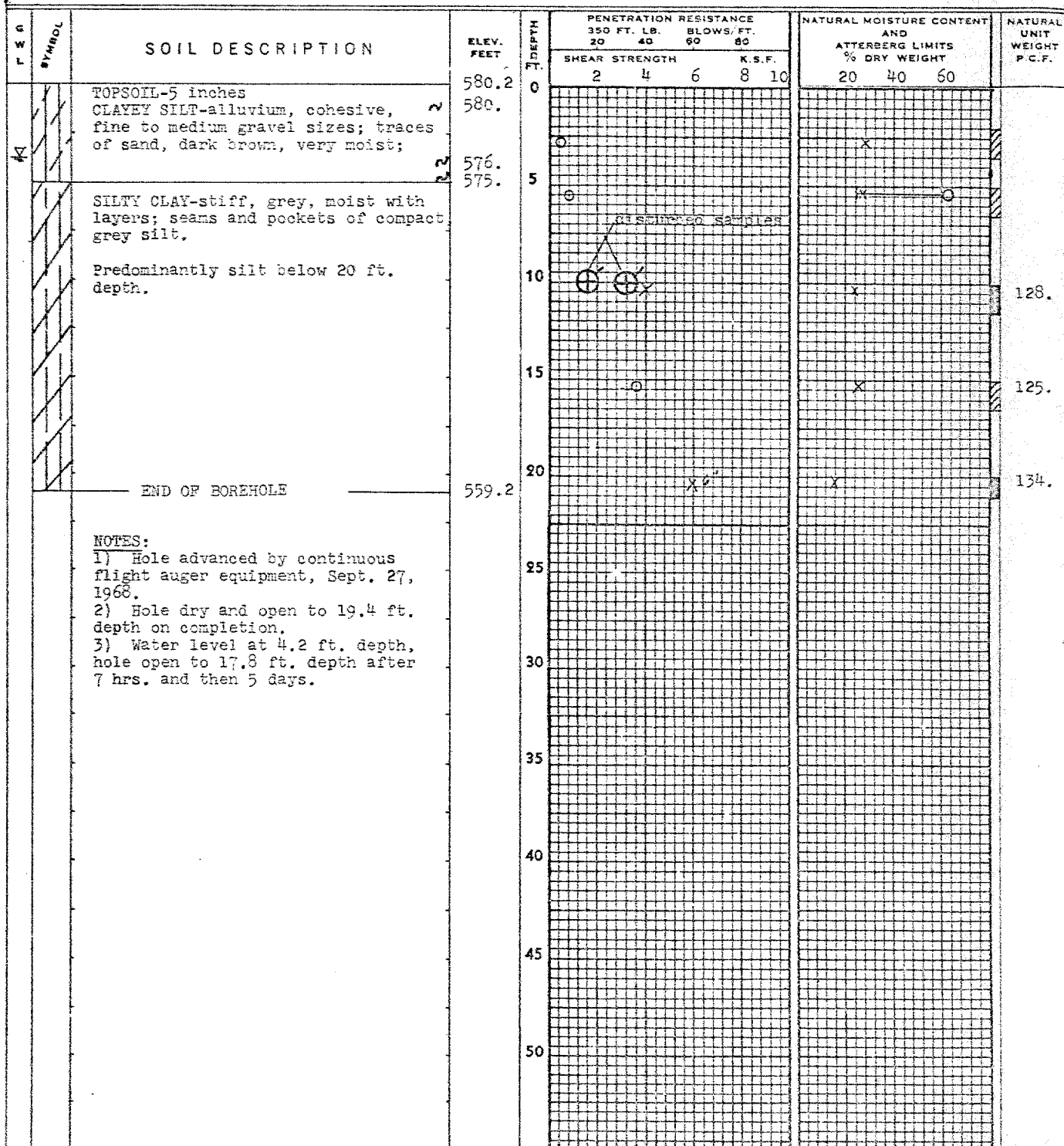
DRAWING No. 6

PROJECT Proposed Bridge,
 LOCATION Shoreham Drive,
 North York.

2" O.D. SPLIT TUBE 
 2" I.D. SHELBY TUBE 
 2" DIA. CONE 
 PUSHED 
 VANE TEST AND SENSITIVITY (S) + S

NATURAL MOISTURE 
 PLASTIC AND LIQUID LIMIT 
 UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 
 % STRAIN AT FAILURE 



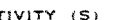



HOLE LOCATION AND DATUM SEE DRAWING No. 1



BOREHOLE LOG

JOB No. J 4676BOREHOLE No. 7DRAWING No. 7PROJECT Proposed Bridge,LOCATION Shoreham Drive,North York.

HOLE LOCATION AND DATUM SEE DRAWING No. 1

2" O.D. SPLIT TUBE  22" I.D. SHELBY TUBE  32" DIA. CONE  4PUSHED  PVANE TEST AND SENSITIVITY (S)  + SNATURAL MOISTURE  XPLASTIC AND LIQUID LIMIT  0UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE  15 5 10
% STRAIN AT FAILURE

LEG	SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FT.	PENETRATION RESISTANCE 350 FT. LB. BLOWS FT.					NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT					NATURAL UNIT WEIGHT P.C.F.
					SHEAR STRENGTH					K.S.F.					
					20	40	60	80	100	10	15	20	25	30	
		TOPSOIL-8 inches	605.1	0											
1/4		SILTY CLAY-very stiff, cohesive, odd fine gravel sizes, brown, moist.	604.3	5											
				10											
				15											
				20											
		CLAYEY SILT TILL-very dense, grey, moist; traces of fine gravel sizes.	594.	25											
		END OF BOREHOLE	578.6	30											
		NOTES: 1) Hole advanced by continuous flight auger equipment, Sept. 27, 1968. 2) Hole dry and open to 24.6 ft. depth on completion and after 5 hrs. and then after 5 days.		35											
				40											
				45											
				50											



RECORDS OF SHALLOW BOREHOLES NOS. 8-11

BOREHOLE NO.	ELEVATION OF BOREHOLE (FEET)	DEPTH OF BOREHOLE (FEET)	STRATA
8	609.7	5	8" topsoil overlying SANDY SILT TILL, brown, moist, cohesive with odd gravel sizes.
9	611.7	5	4" topsoil overlying SILT TILL clayey brown, moist with odd gravel sizes.
10	613.1	5	10" topsoil overlying SANDY SILT TILL brown, moist, with odd gravel sizes.
11	615.1	5	4" topsoil overlying SILT TILL, cohesive brown, moist with odd gravel sizes.
12	617.2	5	7" topsoil overlying SILT TILL, cohesive brown; moist with odd gravel sizes.

NOTES:

- i) The above boreholes were advanced by continuous flight auger equipment on September 27th, 1968.
- ii) All the boreholes were dry and open to 5 feet depth on completion.

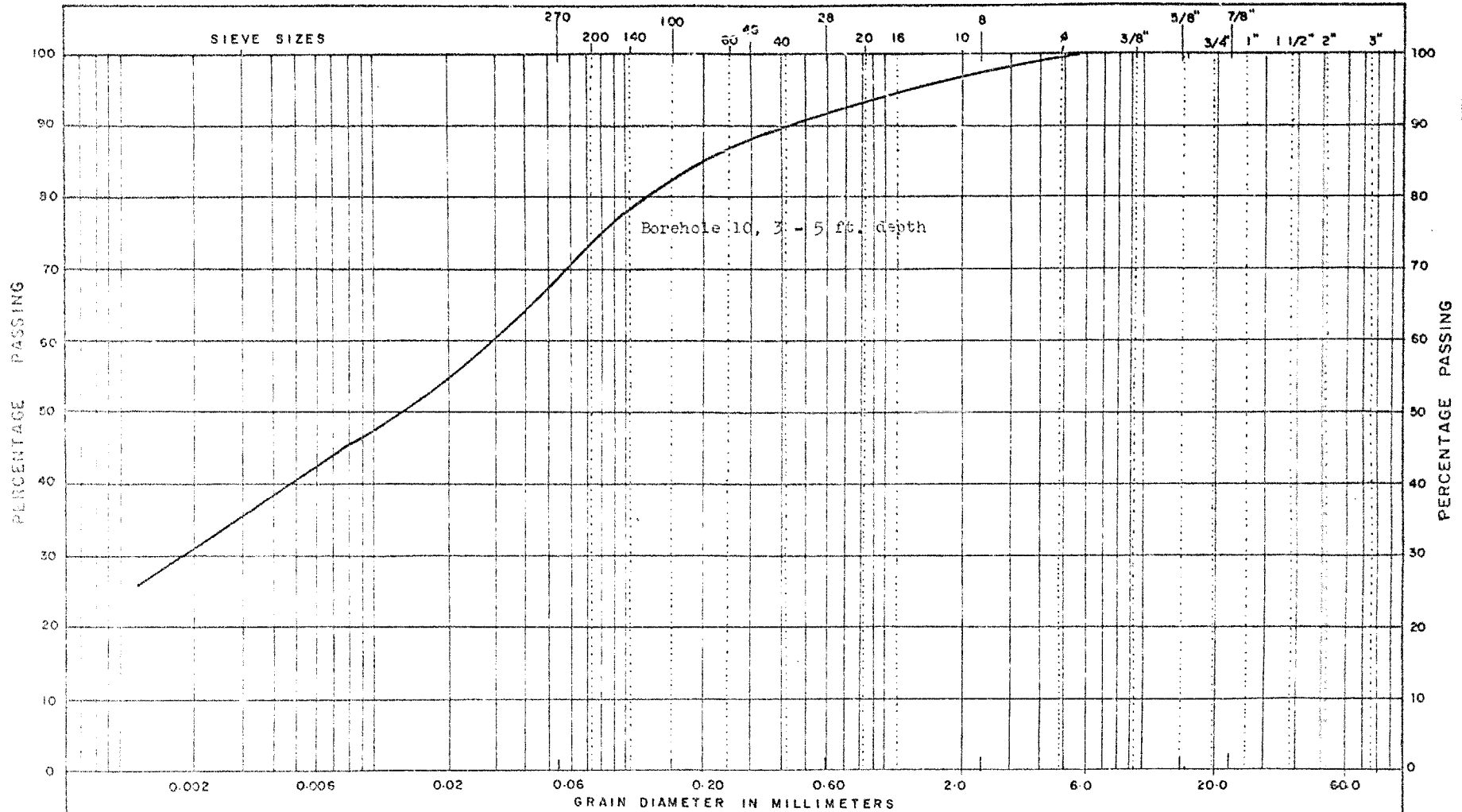
RECORDS OF DYNAMIC CONE TESTS

CONE NO.	ELEVATION OF STATION (FEET)	RECORDED BLOWS PER FOOT PENETRATION INTO SEWER BACKFILL
1	615.4	4 - 3 - 2 - 3 - 3 - 2
2	616.2	6 - 4 - 2 - 6 - 5 - 3
3	616.8	5 - 3 - 3 - 3 - 6 - 4
4	616.6	5 - 2 - 3 - 5 - 5 - 4
5	615.6	5 - 5 - 3 - 2 - 2 - 7

NOTES:

- i) Location of cone tests shown on Dwg. No. 1.
- ii) All cones driven to a final depth of 6 feet below ground level.

MECHANICAL ANALYSIS



MODIFIED M.I.T. CLASSIFICATION

GRADING OF SILT TILL FROM BOREHOLE NO. 10

SHOREHAM DRIVE

NORTH YORK

WILLIAM TROW



ASSOCIATES LTD.

PROJECT: J 4676

DWG. NO. 10

