

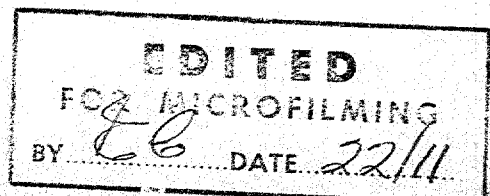
#62-F-246C

W.P. #928-61

NEPEAN BAY

OTTAWA

OUTFALL SEWER



RACEY, MACCALLUM AND ASSOCIATES
LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers
AND ASSOCIATED STAFF

MONTREAL



OTTAWA

TORONTO

TORONTO DIVISION
59 CURLEW DRIVE
DON MILLS, ONT.

DONALD C. MACCALLUM, B.ENG., M.E.I.C., P.ENG.

H. JOHN RACEY, B.SC., M.E.I.C., P.ENG.

GEORGE L. HOUGHTON, A.M.I.MECH.E., M.E.I.C., P.ENG.

OUR REFERENCE NO. S-632/T-3432

December 31, 1962.

Mr. D. W. Farren,
Asst. Road Design Engineer,
Department of Highways of Ontario,
Parliament Buildings,
TORONTO, Ontario.

RE: SOIL INVESTIGATION
PROPOSED OUTFALL SEWER, NEPEAN BAY
OTTAWA

Dear Mr. Farren,

Further to our telephone conversation of today with Mr. E. von Mirbach from Messrs. DeLeuw, Cather and Company, in Ottawa we are forwarding you five (5) copies of our report regarding the above project.

The same number of copies of our earlier report dealing with the marine section of the same project will be sent to your attention in the near future.

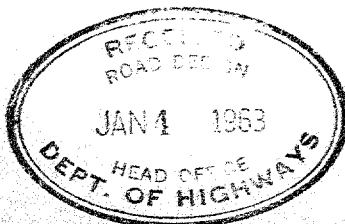
Yours very truly,

RACEY, MACCALLUM AND ASSOCIATES LIMITED,

I. P. Lieszkowszky

I. P. Lieszkowszky, P. Eng.,
Project Engineer.

IPL/eb



*One copy of report retained
by Foundation Section
Jan 7, 1963 A. J. Stennison*

25-9

RACEY, MACCALLUM AND ASSOCIATES LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers
AND ASSOCIATED STAFF

WP 928 -C1

MONTREAL



OTTAWA

TORONTO

DONALD C. MACCALLUM, B.ENG., M.E.I.C., P.ENG.

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TORONTO DIVISION
59 CURLEW DRIVE
DON MILLS, ONT.

Reference: S-632/T-3432

December 19, 1962.

De Leuw, Cather & Company of Canada Limited,
2277 Riverside Drive,
OTTAWA, Ontario.

Attention: Mr. H. Von Mirbach, P.Eng.

RE: SOIL INVESTIGATION,
PROPOSED NEPEAN BAY OUTFALL SEWER,
OTTAWA, ONTARIO.

Dear Sirs:

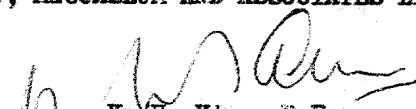
This letter accompanies our detailed report on the soil investigation carried out at the above site.

Detailed information on the soil and ground water conditions is given in the report for the design of the sewer and of construction procedures. In general the investigation has shown that the soil and ground water conditions at the site are complex and variable. The main problems which will be encountered in the construction of the outfall sewer are given by a combination of high ground water levels and relatively permeable soils. The purpose of the discussion has been primarily to emphasize the major construction problems envisaged and to suggest means whereby they might be overcome.

We trust that this report contains all of the required information. Should you have any questions or if we can be of assistance in any other way please do not hesitate to call on us.

Yours very truly,

RACEY, MACCALLUM AND ASSOCIATES LIMITED


K. H. King, P.Eng.,
Divisional Soils Engineer

KHK/KA

I N D E X

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Our reference: S-632/T-3432

December 19, 1962

INTRODUCTION

On October 27th, 1961, Racey, MacCallum and Associates Limited was authorized to carry out a subsurface investigation in the Nepean Bay area in Ottawa. The object of the investigation was to determine and to report on the soil conditions along the proposed route of the Nepean Bay outfall sewer designed by Messrs. De Leuw, Cather & Company, Consulting Engineers, Ottawa.

The investigation was divided into two programmes. The first, consisting of four boreholes, was carried out off-shore in the outlet area of the sewer, and a report giving the results of this investigation was presented November 16th, 1961, under the same reference number.

The object of the second programme was to explore soil conditions on land and present factual data relevant to design and construction.

This report deals with the second phase of the investigation only. It presents the soil conditions encountered in the boreholes and contains recommendations for design and construction procedures based on these results.

PROCEDURE

The field work for the eighteen boreholes on land was carried out on two occasions about one year apart. Work for boreholes 5 to 12 inclusive was commenced November 23rd, 1961 and finished December 7th, 1961. Adverse water conditions necessitated sinking additional boreholes (5A, 5B, 6A, 6B, 6C, 6D, 13 and 14) in the area where tunnelling was required. The field work for these was carried out between December 13th, 1961 and January 25th, 1962. Finally, because of a change in the alignment of the sewer, three boreholes (15, 16 and 17) were performed on November 12th, 13th and 14th, 1962.

The boreholes were located by the client, who also obtained the borehole elevations with reference to geodetic datum. The approximate locations of the boreholes are shown on Drawing No. 1 attached.

The boreholes were put down using a skid-mounted diamond drill rig, equipped for soil sampling. All field work was carried out in accordance with our standard procedures as detailed in Appendix "A".

For a better assessment of dewatering problems during construction, a series of borehole seepage and pumping tests, together with wellpoint pumping test, was performed in the area of tunnelling. In addition to these, two test pits were excavated at the locations indicated on Drawing No. 1. Test results are presented in Enclosures 1 to 8 inclusive, and discussed under "Water Conditions".

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PROCEDURE - Continued

Because of the granular nature of the soils, there was no need for extensive laboratory testing, and the shear strength of the cohesive soils encountered was determined by field vane tests. The results of two mechanical sieve analyses are presented on Appendix "B".

SITE AND GEOLOGY

The site of the investigation is located north of The Queensway and extends to the Nepean Bay. On the east it is bounded by Champagne Street and by Breezehill Avenue on the west. Most of the area within these boundaries is part of the C.P.R. yard with many sidings and two main lines crossing it.

Available geological information indicates that sedimentary rocks of Ordovician Age constitute the bedrock formation at the site. This is overlain by glacial and post-glacial deposits. Most of the overburden in the area was laid down either by the ice during glaciation, or by the Champlain Sea which occupied the depression formed by the retreating glaciers. Present stream courses eroded much of these deposits replacing them with alluvial deposits. Recent granular fill placed over the alluvial deposits makes this picture even more complex.

SOIL CONDITIONS

Detailed soil conditions at the location of the test holes are given on the individual borehole logs included in this report.

Figure 2 attached shows a simplified soil stratification along the proposed sewer route.

The assumed soil properties for each type of soil are tabulated in Table 1. Therefore only a brief summary of the different soil types encountered in the boreholes will be given in the following.

Peat

A soft organic peat was encountered in Borehole 5, extending from the surface to a depth of 18 inches. The wet unit weight of this material can be assumed to be 90 p.c.f. and to weigh 40 p.c.f. while submerged. The shear strength of the peat is so small that for all practical purposes it should be neglected.

Fill

A predominantly granular fill, consisting of sand, gravel and boulders with occasional ashes and cinders, was encountered in Boreholes 5, 5A, 5B, 6, 13, 17, 11 and 12. The standard penetration resistance ranges from 4 to 26 blows per foot indicating that the fill is in a loose to dense state. The wet unit weight of the fill can be taken as 110 p.c.f. and the submerged unit weight as 60 p.c.f.

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SOIL CONDITIONS - ContinuedBrown gravel and boulders

Boreholes 5, 5A, 5B, 6A, 13 and 15 revealed a stratum of dense gravel and boulders of varying thickness. Because of the similar nature of this stratum to the overlying granular fill, it was often difficult to differentiate between the two, and thus the boundaries between them as shown on the borehole logs and on the profile are approximate only. The standard penetration resistance was generally over 20 blows per foot, indicating that the relative density of the stratum is dense. However it is possible that the high resistance is due to the stone content in the stratum. In fact, in many cases the holes could be advanced only by diamond drilling, because of the presence of large boulders.

The wet unit weight of this stratum can be assumed to be 130 p.c.f. and 76 p.c.f. while submerged.

Glacial till

Overlying the bedrock in most of the boreholes (5A, 5B, 13, 15, 16 and 17) there is a stratum of loose to dense glacial till. Most of the samples contained a considerable gravel content with a trace of clay in a sandy and silty matrix. This wide range of particle sizes is typical of glacial deposits. The till had a high resistance against penetration and the recorded blows per foot range from 5 to 140, indicating that the relative density of the stratum is loose to very dense. The assumed wet unit weight of the loose and dense till is 125 p.c.f. and 150 p.c.f. respectively and the corresponding submerged unit weights are 65 p.c.f. and 90 p.c.f.

Soft grey clay

Boreholes 6A, 15, 16 and 17 encountered a soft, sensitive marine clay typical of the Ottawa area. The stratum overlies the glacial till, or the bedrock directly where the till has been eroded.

The shear strength of the clay has been determined by field vane tests, indicating an in-situ shear strength of 1500 p.s.f. to 3000 p.s.f. However it is possible that the higher values are due to sand seams present in the stratum, therefore we suggest using only 1500 p.s.f. for design. The actual values are shown on borehole logs 15, 16 and 17. The sensitivity, i.e. the ratio of the undisturbed to the remoulded shear strengths, is 4 to 5, indicating that the clay is very sensitive to disturbance.

The standard penetration resistance was less than 4 blows per foot. Based on this, and on the measured strengths, the consistency of the stratum can be described as soft.

The unit weights may be assumed as follows: Wet unit weight 115 p.c.f., submerged unit weight 53 p.c.f.

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SOIL CONDITIONS - ContinuedFirm grey silty clay

This stratum, which was encountered in boreholes 16, 17 and 12, can be considered as the desiccated crust of the soft marine clay. It has a brownish grey colour and occasional sand seams are encountered throughout. The undrained shear strength of the stratum can be assumed to be about 2000 pounds per square foot, the wet unit weight 120 p.c.f. and the submerged unit weight 58 p.c.f. The standard penetration resistance varies from 7 to 14 blows per foot, indicating that the consistency of the stratum is firm.

Bedrock

The sedimentary bedrock was cored in boreholes 5A, 6A, 12, 13, 16 and 17. As shown on the individual borehole logs the bedrock varied from shale to limestones. The shale, intersected by mud seams, showed a banded structure to the depth it was explored. From the limestone it was possible to recover 90 - 100 percent of the core indicating a generally sound rock; however the presence of joints and cracks was also detected which might be water bearing.

WATER CONDITIONS

The position of the ground water table was observed in each borehole and the measured ground water levels are plotted on the individual borehole logs. In addition, enclosure No. 9 summarizes these results. The water table in the C.P.R. yard appears to reflect the water level in the nearby Nepean Bay whereas south of Wellington Street the phreatic surface shows a rising tendency, following the profile of the bedrock. However, water conditions could vary from place to place dependent on the local soil conditions.

Because of the great importance of water conditions to tunneling operations, the ground water observations were augmented by several seepage and pumping tests and two test pits which were carried out in the area where tunnelling will be required.

Boreholes and observation holes No. 5A, 5B, 6A, 6B, 6C, 6D, 13 and 14 were put down mostly for this purpose. The results of these tests are summarized as follows:

B.H. 5A. Seepage observations were carried out when the borehole had been advanced to depths of 5.0, 7.5, 10.0, 12.5, 15.0 and 16.5 feet. Down to a depth of 12.5 feet the rate of seepage was slow, and moderate below this depth. For details see Enclosure No. 1.

B.H. 5B. Water was pumped into the borehole and the rate of drop was recorded when the borehole was at depths of 3, 6, 9, 12, 15, 18 and 19.2 feet. The rate was found to be low at 3, 9 and 18 feet depths; moderate at 6 and 15 feet; rapid at 12 and extremely rapid at the bedrock surface at 19.2 feet.

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WATER CONDITIONS - Continued

B.H. 6A. A pumping test was attempted at this location but the size of the pump was such (5 gal. per minute) that it was impossible to lower the water table in the hole by more than 6 inches. Detailed results are given on Enclosure 3.

B.H. 6B. In contrast to B.H.6A, seepage into this hole was so slight that the proposed pumping with a submersible pump was not carried out. See Enclosure 4.

B.H. 6C. Seepage tests were carried out at depths of 8.0, 10.0, 11.0, 16.0 and 18.3 feet. At 8 feet the soil seems to have low permeability, but the permeability was high at the other elevations. For results see Enclosure 5.

B.H. 6D. In order to get a better idea of the drainage characteristics of the strata, it was decided to install a wellpoint, similar to one a contractor might use, and observe the drawdown at the wellpoint as well as at two observation holes, one 10 feet and the other 20 feet away. The method of installing the wellpoint and the observation holes is shown on Enclosure 11. The details of the test are shown on Enclosure No. 6. The relationship between drawdown and distance from the wellpoint is plotted on a semi-logarithmic basis on Enclosure No. 10. The test indicates that the stratum is very permeable and the phreatic surface after drawdown remains relatively flat.

B.H. 13. A seepage test indicated that the permeability below a depth of 15 feet is high and medium at higher levels. For detailed results Enclosure No. 7 should be consulted.

B.H. 14. The seepage test performed in this test hole yielded similar results to the test in hole No. 13. Enclosure No. 8 contains the results of the tests at 11, 15, 20 and 24 feet.

The above test results were used to calculate the coefficient of permeability of the soil at different locations and depths. The calculated values in cm/sec are shown together with the test results on Enclosures No. 1 to 8.

Test Pits

Because of the variations in the above test results it was decided to excavate two test pits at two convenient locations as close to the area of the proposed tunnels as possible.

Both test pits were excavated by a back-hoe and were carried down to bedrock.

Test Pit No. 1. This pit was located in the C.P.R. yard north of Wellington Street Bridge, between the existing platform and the shed. The excavation was about 18 feet deep, 25 feet long and about

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December 19, 1962

WATER CONDITIONS - ContinuedTest Pit No. 1 - Continued

20 feet wide at the top and 10 feet at the bottom. The soil stratification corresponded generally to that encountered in Borehole No. 6A, however the clay was thicker in the test pit and it was encountered 10 feet below surface. Also it appeared to diminish in thickness in the northerly direction. Water first entered the excavation about 6 feet below surface and water seeped into the pit from all sides through the granular material overlying the clay. The rate of flow can be described as moderate and somewhat heavier on one corner of the excavation. With time, the rate of flow appeared to decrease. Although no attempt was made to measure the rate of flow it appeared that regular sump pumps could keep the excavation dewatered.

Test Pit No. 2. A 17 feet deep, 20 feet wide and approximately 30-40 feet long hole was excavated just north of the "River Lead" railway tracks and about 40 feet east of the proposed sewer line. The excavation revealed a similar stratification as indicated by borehole No. 5B. Contrary to expectations the excavation in the granular material was carried out in the "dry". Even though the excavated material was wet it contained a sufficient amount of fines to make the strata relatively impervious and to keep the trench free of water for the short time it was open. As the excavation reached the rock elevation water started to seep into the trench through the fissured rock. However the rate of flow was quite low and could easily be pumped from sumps.

As previously noted, the water conditions will vary considerably from place to place dependent on the local soil conditions. Furthermore there are indications that the bedrock may be quite permeable in places.

DISCUSSION

It is understood that the diameter of the proposed sewer will vary from 48 inches at Gladstone Avenue to 72 inches at the outlet, and that it will be laid in open trench with the exception of places where the sewer crosses the C.P.R. tracks where tunnelling is required.

Open Cut Section(a) Trench in bedrock

Between the Queensway M.H. No. 311 and a point about 1200 feet north the invert of the sewer lies within the shale and limestone bedrock. Though it is possible that the upper one or two feet of the rock formation could be excavated with heavy machinery some drilling and blasting seems to be unavoidable. If the sides of the trench above rock level could not be excavated on a 1 in 1 slope because of limited space, shoring will be required. Ground water running into the excavation from aquiferous sand seams could be pumped or drained by gravity.

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December 19, 1962

DISCUSSION - Continued(b) Trench in cohesive soil

Approximately 1000 linear feet of soft marine clay will be intersected in the vicinity of boreholes 15, 16 and 17. Field vane tests indicated an average undrained shear strength of 1500 p.s.f. but also a high sensitivity to disturbance. Also in this section the excavations will be the deepest, about 21 feet in places.

Excavation in this material will be easy and during the short time of construction the sides of the trench should stand temporarily on a nearly vertical face with minimum support. Due to the high sensitivity of this clay it is recommended that a working mat of granular material be placed at the bottom of the trench before extensive work in the trench is commenced. If this clay is disturbed to any extent it will lose its strength and tend to become fluid. Because of the low permeability of the clay, no special dewatering problems along this section of the sewer are anticipated although the infiltration of surface water must be handled.

(c) Trench in cohesionless soil

North of Somerset Street, along the Railway tracks and across the C.P.R. yard the sewer will pass through dense glacial deposits, dense sand and gravel, and granular fill, all of which contain a high percentage of stones and boulders. Beside the excavation of this hard material the main source of difficulty is the high water table, generally corresponding to the water level in the Bay, and the relatively high permeability of the strata. Even though large quantities of water might have to be handled it is believed that draining by gravity would be not only feasible but also the most economical. By working in an upstream direction, the collected water could run in the finished part of the sewer down to the next manhole or to a temporary sump from where it could be pumped out. It would be advisable to keep only short sections of the trench open at any time, thereby reducing the amount of water to be handled.

To protect the railway tracks and other structures we recommend that the trenches should be adequately sheeted and braced in a manner that lateral movement of the retained soil is minimized. The earth pressure on the sheeting can be calculated using the assumed properties of the different soil types, as tabulated in Table 1. However, because of the variations in soil conditions present, it is recommended that for practical purposes the pressures may be calculated assuming an average equivalent fluid weight for the soil of 35 pounds per cubic foot above the water level and 20 pounds per cubic foot under the water. The full hydrostatic pressure due to the head of water retained must be added to the lateral soil pressure.

DISCUSSION - Continued(d) Protection of existing structures

Special consideration must be given to the protection of some important structures along the route of the sewer. Because of their great importance and somewhat different conditions they will be dealt with separately.

Bridge Piers at Somerset Street

Here the sewer passes midway between the piers. The footings for the piers are only 14 feet apart, thereby leaving only somewhat over 2 feet between the structure and the sides of the trench. The bottom of the trench at this point will be at elevation 176 and original plans of the bridge show the footings at elevation 186. However, field measurements on the exposed footings indicated that the actual elevations are about 2 feet higher, i.e. they are at elevation 188. With the trench bottom 12 feet below the footing elevation the safety factor against the footing sliding into an open excavation is less than unity, therefore special means of securing the stability of the footings must be considered.

The simplest way to protect the structures is driving closed sheeting and providing adequate bracing to prevent excessive lateral movement of the retained soil. The sheeting consisting of heavy, interlocking type steel sections, should be driven prior to excavation below elevation 188. The sheeting should be driven at least three feet into the dense glacial till underlying the soft clay. As excavation within the sheeting proceeds, adequate bracing should be provided to prevent excessive horizontal movement of the sheeting. After construction the sheeting shall be left in place and the backfill in the trench be well compacted. We also recommend that well-graded granular material be used for backfilling up to elevation 188, and the material be placed in lifts not exceeding 6 inches. Each layer shall be well compacted with vibratory compactors. As an additional precaution the length of the trench open at any time should be limited to about 30 feet.

Difficulties in driving the sheeting through the gravelly sand stratum could be expected, and especially to maintain the proper interlocking of the sheeting. The jamming of the interlocks can be best prevented by starting all the sections and driving each piece in series a few feet, then coming back again as many times as may be necessary. Excessive vibration when driving the sheeting should be avoided as this could cause settlement of the piers.

The sheeting and bracing should be designed to withstand in addition to the earth and water pressures the lateral pressure caused by the structure. If these pressures are so high that the

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December 19, 1962

DISCUSSION - Continued(d) Protection of existing structuresBridge Piers at Somerset Street - Continued

shoring can not resist them safely without excessive lateral movement, underpinning should be considered.

Bridge Piers at Wellington Street

Here the bottom of the approximately 13 feet wide trench will be about 10 feet below the footing elevation of the piers. The space between the side of the trench and the footings will be about 9 feet.

There is no borehole in the immediate vicinity of this section but one could expect similar conditions as encountered in borehole No. 6A or test pit No. 1, both of which are about 200 feet away. The dense gravelly deposit close to the surface might be underlain by a soft clay stratum resting on bedrock or glacial till. Conditions are therefore similar, though less severe, to those under the Somerset Street bridge, and therefore similar precautions should be taken. At this location there is no need to consider underpinning, but a good shoring system with sheeting driven below the bottom of the trench through any soft strata and into underlying dense deposits will be required. For the excavation and backfilling similar methods could be used as outlined for the trench at Somerset Street bridge.

Department of Public Works Building

North of Gladstone Avenue in the vicinity of borehole No. 11 the sewer will pass alongside the Department of Public Works Building. The invert of the sewer will be about 9 feet below the existing wall footings and the centre line of the sewer at places will be not more than 10 feet away from the building.

The invert of the sewer is below the rock elevation, and therefore the excavation might involve some blasting of the rock. Care should be taken that only small charges are used as shock due to violent explosions might cause settlements of the granular foundation material of the existing building. Otherwise the security of the wall footings will depend on the stability of the retained trench walls and therefore a good shoring system should be employed in this area.

Tunnel Sections

Tunnelling will be required in the C.P.R. yard under the "River Lead" tracks, the Auxiliary tracks and under the C.P.R. South Main line. Most probably in all three areas the tunnel will be driven through dense glacial till or gravel and boulders. Boulders up to a

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December 19, 1962

DISCUSSION - ContinuedTunnel Sections - Continued

foot or two in diameter may be expected and the sandy character of both strata will probably prove abrasive on the excavating equipment.

As the water table lies well above the invert elevations of the sewer and the tunnels will be driven through relatively permeable strata, dewatering of the tunnels will be necessary. To size up this problem two test pits were excavated: one, test pit No. 2, just north of the tunnelling area under the "River Lead" tracks, and test pit No. 1 about 240 feet north of the tunnel under the C.P.R. South Main line. From these test excavations which are described in detail under the "Water Conditions", no serious water problems are anticipated at these two locations. The water seeping into the tunnel through the sides and the face of the tunnel could be gathered into sumps from where it could be pumped out. However, there is indication that in the area of Borehole No. 6 where tunnelling under the "Auxiliary Track" will be required water conditions might be more severe, and special provisions for dewatering, such as well-points or deep wells, may be required.

CONCLUSIONS

1. Boreholes drilled along the proposed route of the sewer revealed heterogeneous soil conditions, varying from gravel and boulders to soft clay or dense glacial till.
2. Besides excavating in the dense gravelly deposits, the permeability of these strata and the high water table indicate that dewatering will be a construction problem. Even though large quantities of water might have to be pumped from sumps, draining by gravity seems feasible for most part of the sewer. However special means of dewatering, such as a well-point system or deep wells, might have to be considered for one of the tunnel sections.
3. To protect the existing structures along the route of the sewer the sides of the trench should be supported by braced sheeting. The sheeting and bracing should be designed to withstand in addition to the earth and water pressures the lateral pressure caused by the structure. If these pressures are so high that the shoring can not resist them safely without excessive lateral movement, underpinning should be considered.



I. P. Lieszkowsky
I. P. Lieszkowsky, P.Eng.,
Project Engineer

Reference: S-632/T-3432

Project: Nepean Bay
Outfall Sewer

TABLE 1.

ASSUMED SOIL PROPERTIES

<u>Soil Type</u>	<u>Wet Unit Weight p.c.f.</u>	<u>Submerged Unit Weight p.c.f.</u>	<u>Angle of Internal Friction ϕ°</u>	<u>Cohesion c p.s.f.</u>	<u>Ph/Pv K_a</u>
Organic Peat	90	40	0	0	1.0
Fill: Granular	110	60	30 ^o	0	0.33
Dense Sand, Gravel, Boulders	130	76	33 ^o	0	0.29
Loose Glacial Till	125	65	31 ^o	0	0.32
Dense Glacial Till	150	90	42 ^o	0	0.20
Dense Silty Sand	140	84	37 ^o	0	0.25
Stiff Grey Silty Clay	120	58	0	1500	1.00
Soft Grey Silty Clay	115	53	0	1000	1.00

Equivalent fluid unit weight 35 p.c.f.

under W.T. 20 p.c.f. + water pressure.

<p align="center">RACEY, MacCALLUM AND ASSOCIATES LIMITED</p> <p align="center">LOG OF BOREHOLE NO. 5</p>			
ORDER NO. S-632 / T-3432	PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA		
LOCATION SEE LOCATION PLAN	GROUND ELEVATION 180.5		DATUM GEODETIC
BORING METHOD		FIELD SUPERVISION BY H.A.G. DATE NOV. 23 / 61	
WASHING WITHIN 8 X CASING		LOG COMPILED BY I.G.B. DATE	
		LOG CHECKED BY I.P.L. DATE	

SUBSURFACE PROFILE			SAMPLES			PENETRATION RESISTANCE BLOWS/FOOT		WATER CONTENT %		UNIT WEIGHT (P.C.F.)	REMARKS — OTHER TESTS
ELEV. DEPTH	STRATUM	SYMBOL	WATER CONDITIONS	NUMBER	TYPE	BLOWS/FOOT	ELEVATION SCALE	STANDARD CONE SHEAR STRENGTH (P.S.F.) COMPRESSION TEST VANE TEST	NATURAL LIQUID LIMIT PLASTIC LIMIT		
160.5 0	GROUND SURFACE						0				
	FILL (CINDERS, SAND, GRAVEL, BOULDERS)	FI		1	S.S.	7					
173.0 7'-6"				2	S.S.	4					
	VERY DENSE, BROWN SAND, GRAVEL AND BOULDERS	CW		3	S.S.	39					
				4	S.S.	96					
166.5 14'-0"	END OF BOREHOLE			5	S.S.	74					

LOSS OF
WASH WATER
AT 13'-0"

NO GROUND
WATER

RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 5A

ORDER NO. 5-632/T-5432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 176.7

DATUM GEODETTIC

BORING METHOD

WASHING WITHIN 8" CASING

FIELD SUPERVISION BY H.A.G. DATED 5/6/61

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY I.P.L. DATE

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE BLOWS/FOOT		WATER CONTENT %		UNIT WEIGHT (P.C.F.)	REMARKS OTHER TESTS
ELEV. DEPTH	STRATUM	SYMBOL	WATER CONDITIONS	NUMBER	TYPE	BLOWS/FOOT	STANDARD CONE	SHEAR STRENGTH (P.S.F.) COMPRESSION TEST VANE TEST	NATURAL LIQUID LIMIT PLASTIC LIMIT			
176.7 0'-0"	GROUND SURFACE											
175.2 1'-6"	BLACK ORGANIC PEAT	PL	-2'-6"	1	S.S.	5						
173.6 3'-1"	SOFT, BROWN, SANDY SILT	ML		2	S.S.	5						
	DENSE, BROWN SAND, GRAVEL AND BOULDERS	GW		3	S.S.	23						
168.7 8'-0"	VERY DENSE, GREY, SILTY, FINE SAND WITH GRAVEL (GLACIAL TILL)	SW		4	S.S.	97						
				5	S.S.	100						
160.2 18'-6"	SEAMED LIME- STONE & SHALE	LI		6	S.S.	60						
156.2 20'-6"	BOREHOLE TERMINATED											

FOR SEEPAGE
TEST RESULTS
SEE ENCL. 1

FOR SEEPAGE
TEST RESULTS
SEE ENCL. 1

RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 5 B

ORDER NO. S-632 / T-3432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 180.5

DATUM GEODETIC

BORING METHOD

WASHING WITHIN Bx CASING

FIELD SUPERVISION BY H.A.G. DATE JAN/24/67

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY I.P.L. DATE

SUBSURFACE PROFILE			SAMPLES		ELEVATION SCALE	PENETRATION RESISTANCE BLOWS/FOOT		WATER CONTENT %		UNIT WEIGHT (P.C.F.)	REMARKS — OTHER TESTS
ELEV. DEPTH	STRATUM	SYMBOL	WATER CONDITIONS	NUMBER		TYPE	CONE STANDARD	SHEAR STRENGTH (P.S.F.)	NATURAL		
180.5 0'-0"	GROUND SURFACE										
175.5 5'-0"	FILL (ASHES, GRAVEL AND BOULDERS)	FI	-5'7"	1	S.S.	2					
				2	S.S.	13					
	COMPACT TO DENSE GRAVEL WITH SAND, SOME SILT, TRACE OF CLAY	GW		5	S.S.	74					
166.5 14'-0"	DENSE SANDY TILL, SOME BOULDERS	SW		4	S.S.	76					
161.3 19'-2"	END OF BOREHOLE POSSIBLE BEDROCK										

FOR SEEPAGE
TEST RESULTS
SEE ENCL. 2

RACEY, MacCALLUM AND ASSOCIATES LIMITED
LOG OF BOREHOLE NO. 6

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

GROUND ELEVATION 180.0

DATUM GEODETIC

WASHING WITHIN BX CASING

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY I.P.L. DATE

SUBSURFACE PROFILE			SAMPLES			PENETRATION RESISTANCE BLOWS/FOOT		WATER CONTENT %		UNIT WEIGHT (P.C.F.)	REMARKS — OTHER TESTS
ELEV. DEPTH	STRATUM	SAMPLES	WATER CONDITIONS	NUMBER	TYPE	BLOWS/FOOT	ELEVATION SCALE	STANDARD	SHEAR STRENGTH (P.S.F.) COMPRESSION TEST VANE TEST		
180.0 0'-0"	GROUND SURFACE						0				
	FILL (CINDERS, WOOD, SAND AND BOULDERS)		5'-4"	1	S.S.	16	5				
				2	S.S.	16	10				
					Ax CORING						
160.0 20'-0"	END OF BOREHOLE						25				

RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 6 A

ORDER NO. S-632/ T-3432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 181.6

DATUM GEODETTIC

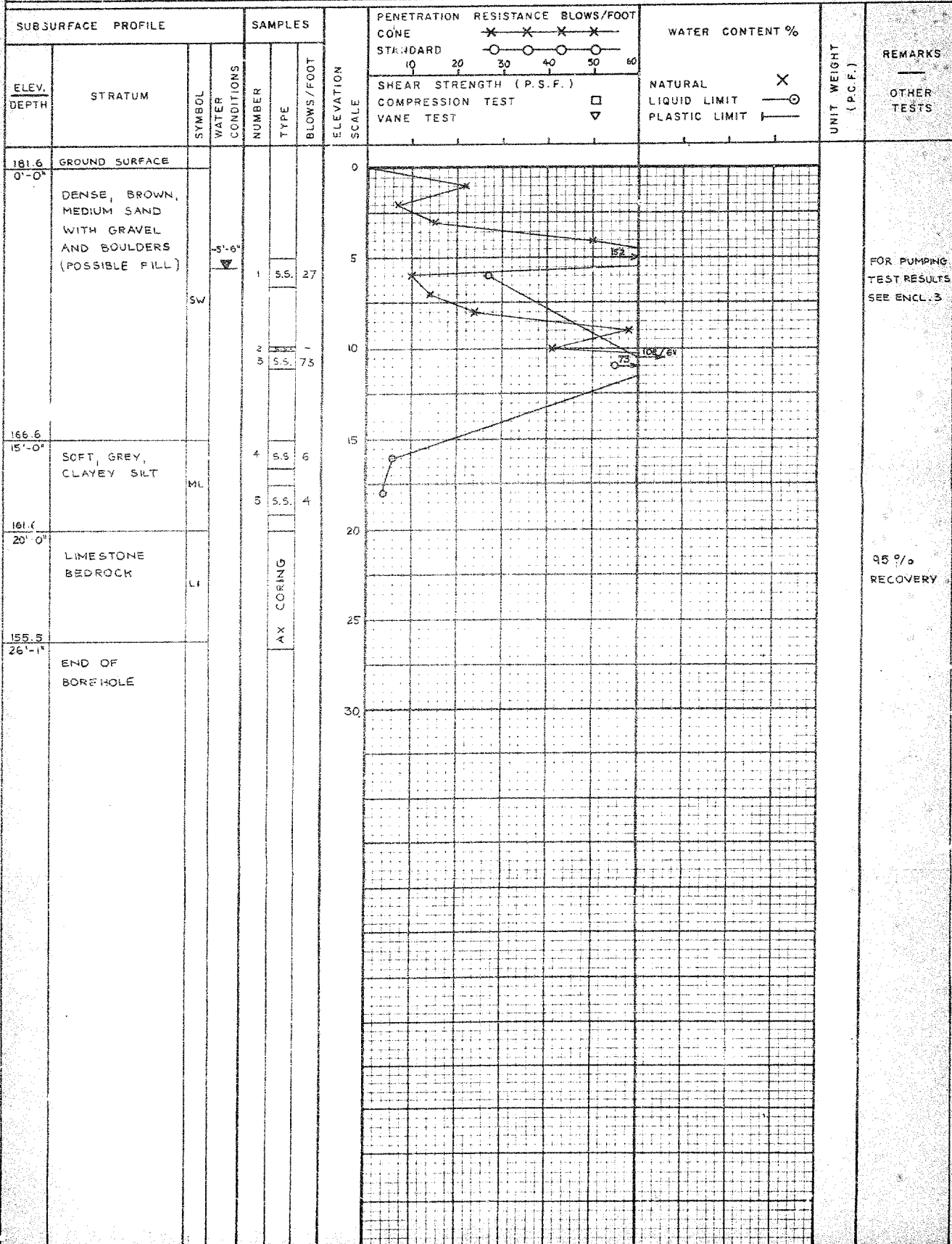
BORING METHOD

WASHING WITHIN Bx CASING

FIELD SUPERVISION BY H.A.G. DATE DEC/7/61

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY I.P.L. DATE



RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 7

ORDER NO. S-632 / T-3432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 184.5

DATUM GEODETIC

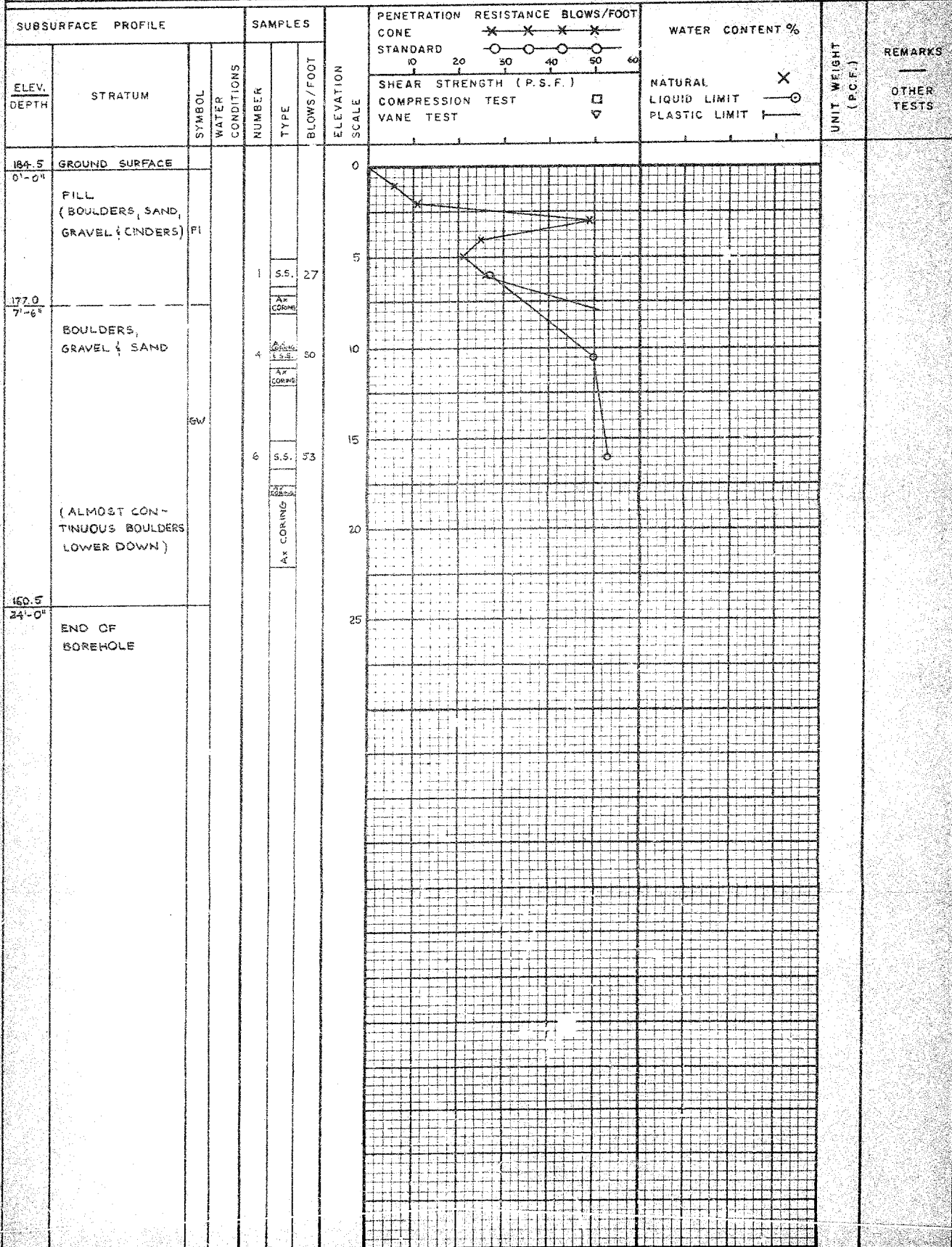
BORING METHOD

WASHING WITHIN Bx CASING

FIELD SUPERVISION BY H.A.G. DATE NOV 29/61

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY K.H.K. DATE SEPT 17/62



RACEY, MACCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 8

ORDER NO. S-632/T-3432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 185.9

DATUM GEODETTIC

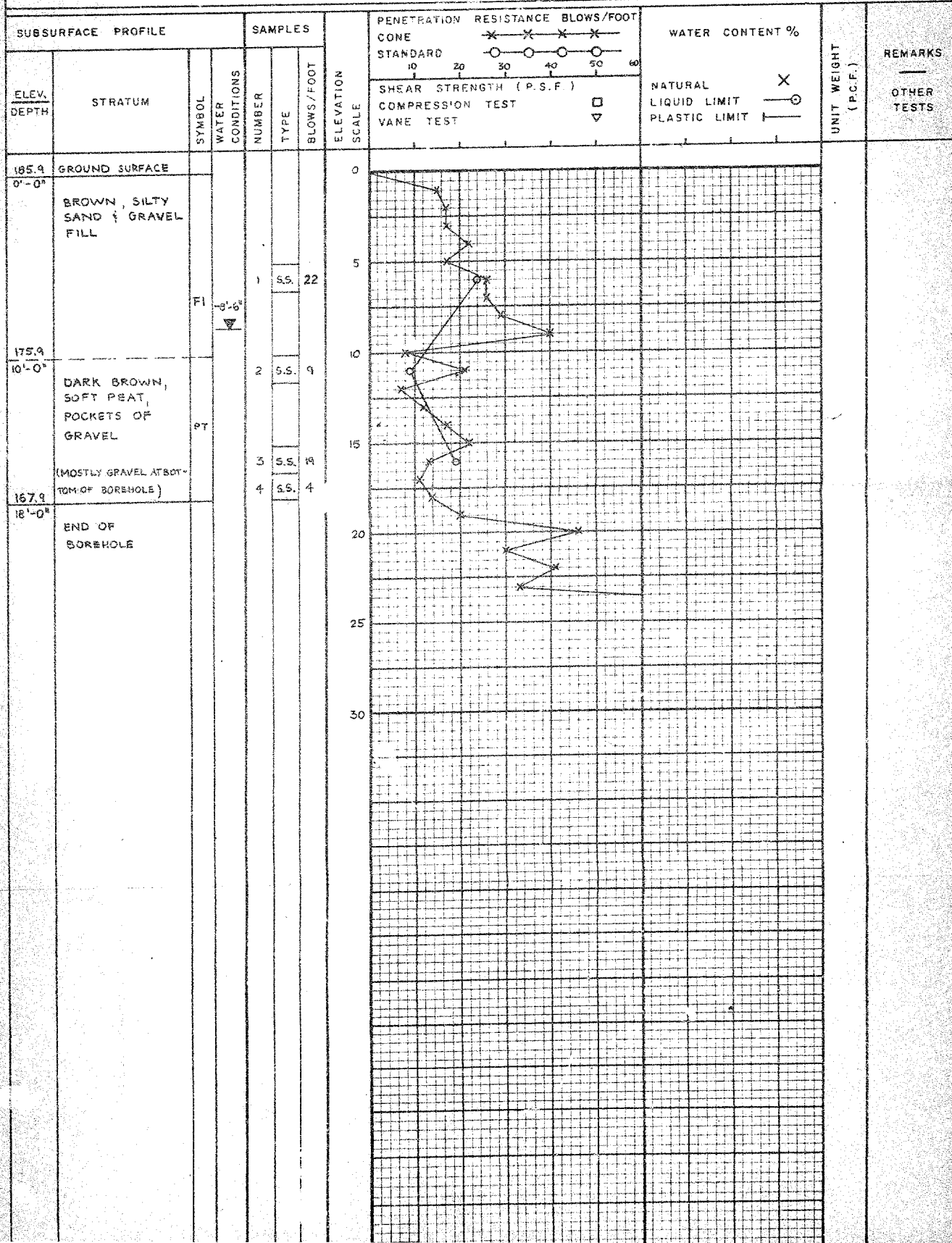
BORING METHOD

WASHING WITHIN Bx CASING

FIELD SUPERVISION BY H.A.G. DATE DEC/1/62

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY K.H.K. DATE SEPT/17/62



RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 9

ORDER NO. S-632 / T-3432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 191.5

DATUM GEODETIC

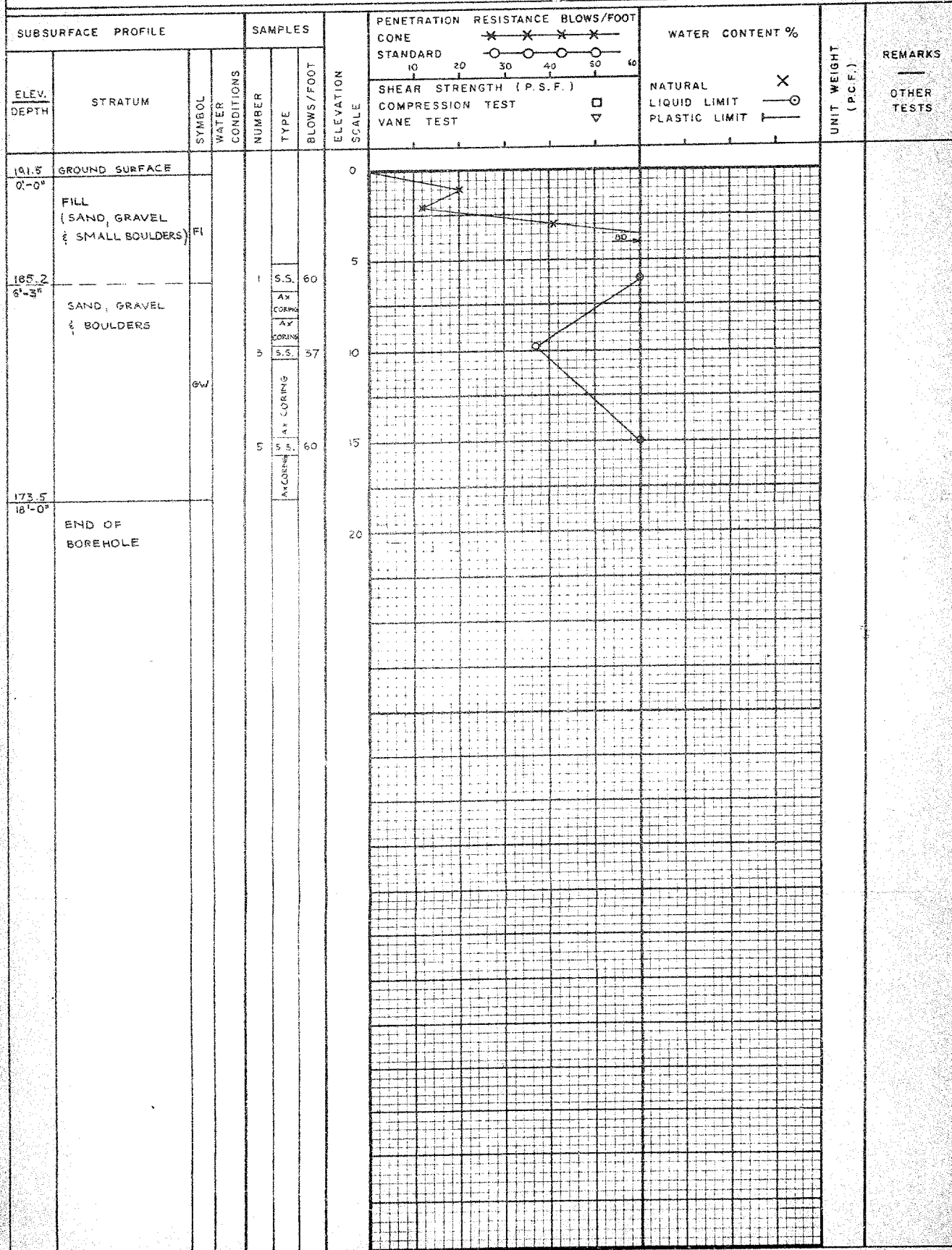
BORING METHOD

WASHING WITHIN 8x CASING

FIELD SUPERVISION BY H.A.G. DATE DEC 11/61

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY K.H.K. DATE SEPT 17/62



RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 10

ORDER NO. S-652 / T-3432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 197.75

DATUM GEODETTIC

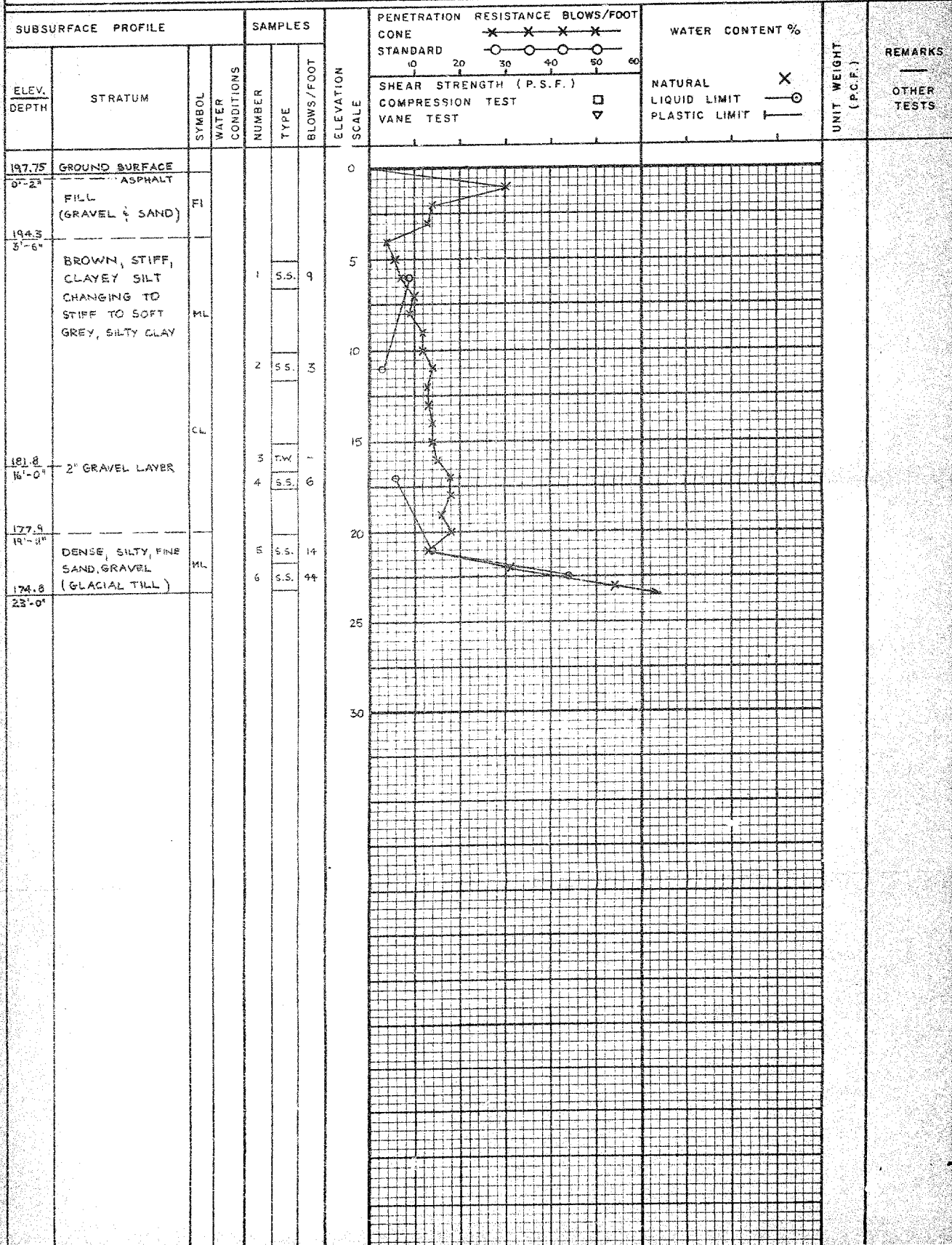
BORING METHOD

WASHING WITHIN 8x CASING

FIELD SUPERVISION BY H.A.G. DATE DEC/4/62

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY K.H.K. DATE



RACEY, MacCALLUM AND ASSOCIATES LIMITED
LOG OF BOREHOLE NO. 11

ORDER NO. 5-532/T-3432	PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA	
LOCATION SEE LOCATION PLAN	GROUND ELEVATION 196.5	DATUM GEODETIC
BORING METHOD WASHING WITHIN Bx CASING		FIELD SUPERVISION BY H.A.G. DATE DEC/8/6 LOG COMPILED BY J.G.B. DATE LOG CHECKED BY K.H.K. DATE SEPT/17/6

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE BLOWS/FOOT		WATER CONTENT %		UNIT WEIGHT (P.C.F.)	REMARKS OTHER TESTS
ELEV. DEPTH	STRATUM	SYMBOL	WATER CONDITIONS	NUMBER	TYPE	BLOWS/FOOT	ELEVATION SCALE	STANDARD CONE SHEAR STRENGTH (P.S.F.) COMPRESSION TEST VANE TEST	NATURAL LIQUID LIMIT PLASTIC LIMIT			
196.5 0'-0"	GROUND SURFACE						0	X X X X	X			
	GREY-BROWN CLAYEY SILT & GRAVEL FILL							10 20 30 40 50 60				
190.5		F1	-6'-0"	1	S.S.	3	5	O O O O				
	GREY, CLAYEY SILT, FINE SAND, GRAVEL (GLACIAL TILL)	ML		2	S.S.	7	10	□ ▽				
186.0 10'-6"	END OF BOREHOLE ENCOUNTERING BEDROCK						15					

RACEY, MacCALLUM AND ASSOCIATES LIMITED
LOG OF BOREHOLE NO. 12

PROJECT NEPEAN BAY, . OUTFALL SEWER, OTTAWA

GROUND ELEVATION 205.25

DATUM GEODETIC

WASHING WITHIN BX CASING

FIELD SUPERVISION BY H.A.G. DATE DEC/5/61

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY K.H.K. DATE SEPT/17/62

SUBSURFACE PROFILE			SAMPLES			PENETRATION RESISTANCE BLOWS/FOOT		WATER CONTENT %		UNIT WEIGHT (P.C.F.)	REMARKS — OTHER TESTS
ELEV. DEPTH	STRATUM	SYMBOL	WATER CONDITIONS	NUMBER	TYPE	BLOWS/FOOT	ELEVATION SCALE	STANDARD CONE SHEAR STRENGTH (P.S.F.) COMPRESSION TEST VANE TEST	NATURAL LIQUID LIMIT PLASTIC LIMIT		
205.25 0'-0"	GROUND SURFACE						0				
199.3 6'-0"	BROWN SAND, SILT AND GRAVEL (PROBABLY FILL)	FI		1	S.S.	26	5				
195.8 9'-6"	GREY, STIFF, CLAYEY SILT	ML					10				
	LIMESTONE BEDROCK 80% - 100% RECOVERY						15				
		LI					20				
							25				
180.9 24'-4"	END OF BORE HOLE						30				

RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 13

ORDER NO. S-632/T-3432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 182.8

DATUM

BORING METHOD

WASHING WITHIN B X CASING

FIELD SUPERVISION BY H.A.G. DATE DEC/13/61

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY I.P.L. DATE

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE BLOWS/FOOT		WATER CONTENT %		UNIT WEIGHT (P.C.F.)	REMARKS — OTHER TESTS
ELEV. DEPTH	STRATUM	SYMBOL	WATER CONDITIONS	NUMBER	TYPE	BLOWS/FOOT	ELEVATION SCALE	CONE STANDARD	SHEAR STRENGTH (P.S.F.) COMPRESSION TEST VANE TEST	NATURAL LIQUID LIMIT PLASTIC LIMIT		
182.8 0'-0"	GROUND SURFACE						0	X				
	FILL (CINDER, SAND)	FI										
178.8 4'-0"							5					
	COMPACT, BROWN, COARSE SAND AND GRAVEL	SP	7-14%	1	S.S.	12						
				2	S.S.	19						
170.3 12'-6"				3	S.S.	R						
	SOFT, GREY, CLAYEY SILT, SOME SAND AND GRAVEL (GLACIAL TILL)	ML		4	S.S.	3						
159.3 23'-6"				5	S.S.	40						
	DENSE, BROWN, SANDY TILL	SW		6	S.S.	R						
153.8 29'-0"							30					
	END OF BOREHOLE (POSSIBLE BEDROCK)											

REFUSAL
ON LARGE
GRAVEL OR
BOULDER

FOR SEEPAGE
TEST RESULTS
SEE ENCL. 7

REFUSAL
ON LARGE
GRAVEL OR
BOULDER

FOR SEEPAGE
TEST RESULTS
SEE ENCL. 7

RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 14

ORDER NO. S-632/T-3432

PROJECT NEPEAN BAY, DUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION

DATUM GEODETIC

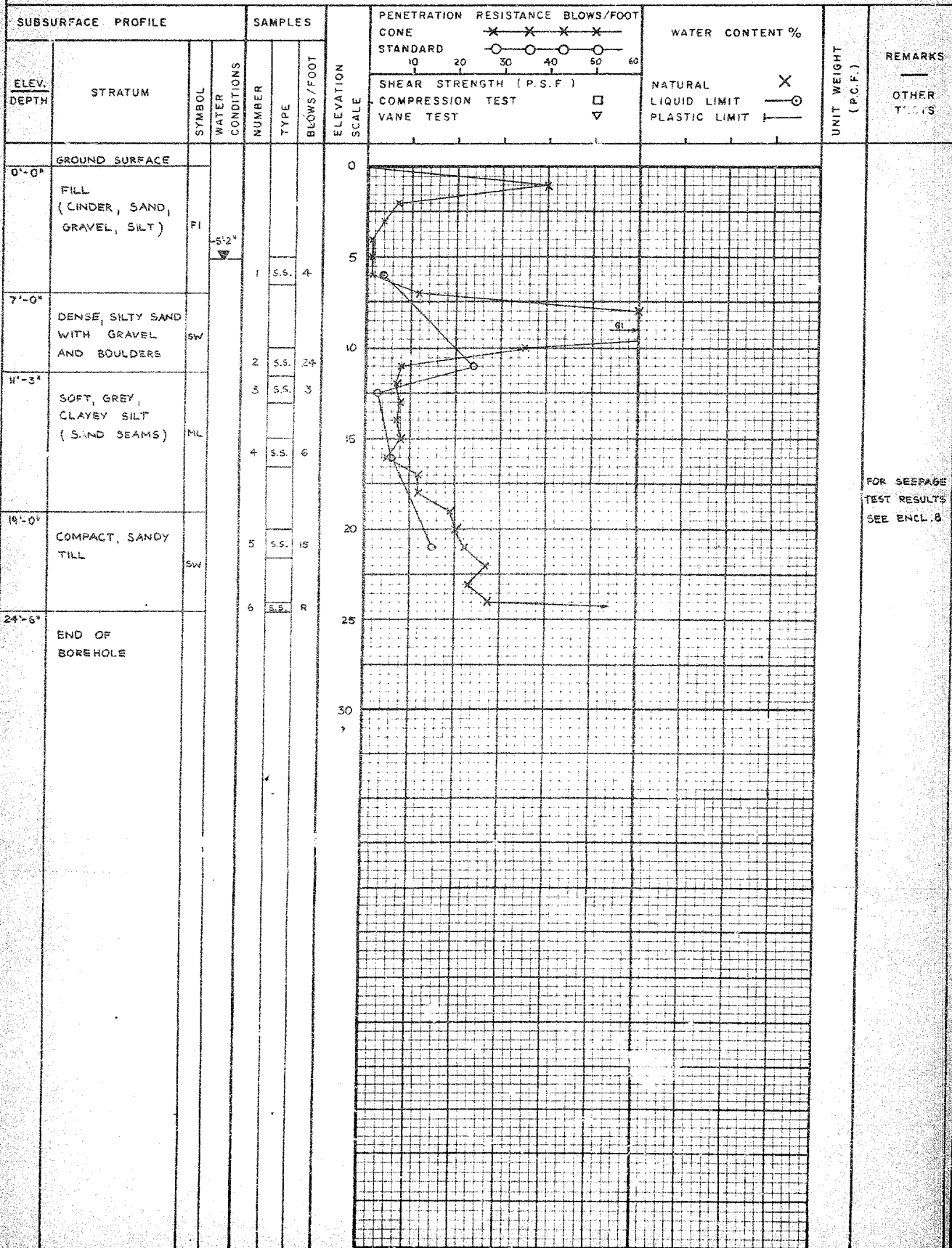
BORING METHOD

WASHING WITHIN Bx CASING

FIELD SUPERVISION BY H.A.G. DATE DEC/15/61

LOG COMPILED BY I.G.B. DATE

LOG CHECKED BY I.P.L. DATE



RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 15

ORDER NO. S-652 / T-3432

PROJECT KAPPAAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 190.9

DATUM GEODETTIC

BORING METHOD

WASHING WITHIN 8" CASING

FIELD SUPERVISION BY H.A.G. DATE
LOG COMPILED BY I.G.B. DATE
LOG CHECKED BY I.P.L. DATE

SUBSURFACE PROFILE			SAMPLES			PENETRATION RESISTANCE BLOWS/FOOT		WATER CONTENT %		UNIT WEIGHT (P.C.F.)	REMARKS OTHER TESTS
ELEV. DEPTH	STRATUM	SYMBOL	WATER CONDITIONS	NUMBER	TYPE	BLOWS/FOOT	ELEVATION SCALE	CONE STANDARD	NATURAL LIQUID LIMIT PLASTIC LIMIT		
190.9 0'-0"	GROUND SURFACE						0				
	BROWN SAND AND GRAVEL WITH BOULDERS (POSSIBLE FILL)	FI	-2' 6"	1	S.S.	36					
				2	S.S.	120					
				3	S.S.	R	5				
				4	S.S.	R					
180.4 10'-6"	SOFT TO FIRM GREY CLAY, TRACE OF SILT	CL		5	S.S.	8	10				
					VANE						
				6	T.W.	-	15				
					VANE						
169.9 21'-0"	VERY DENSE, SILTY TILL, SOME SAND & GRAVEL	ML		7	S.S.	1	20				
165.2 25'-9"	END OF BOREHOLE (POSSIBLE BEDROCK)			8	S.S.	140	25				

PENETRATION RESISTANCE BLOWS/FOOT

CONE STANDARD

SHEAR STRENGTH (P.S.F.)

COMPRESSION TEST

VANE TEST

WATER CONTENT %

NATURAL LIQUID LIMIT

PLASTIC LIMIT

CU = 2200 PSE

CR = 480 PSE

CU = 1550 PSE

CR = 370 PSE

Cu = 2200 P.S.F.
Cr = 480 P.S.F.

Cu = 1550 P.S.F.
Cr = 370 P.S.F.

RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 16

ORDER NO. S-632/T-3432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 199.1

DATUM GROUND

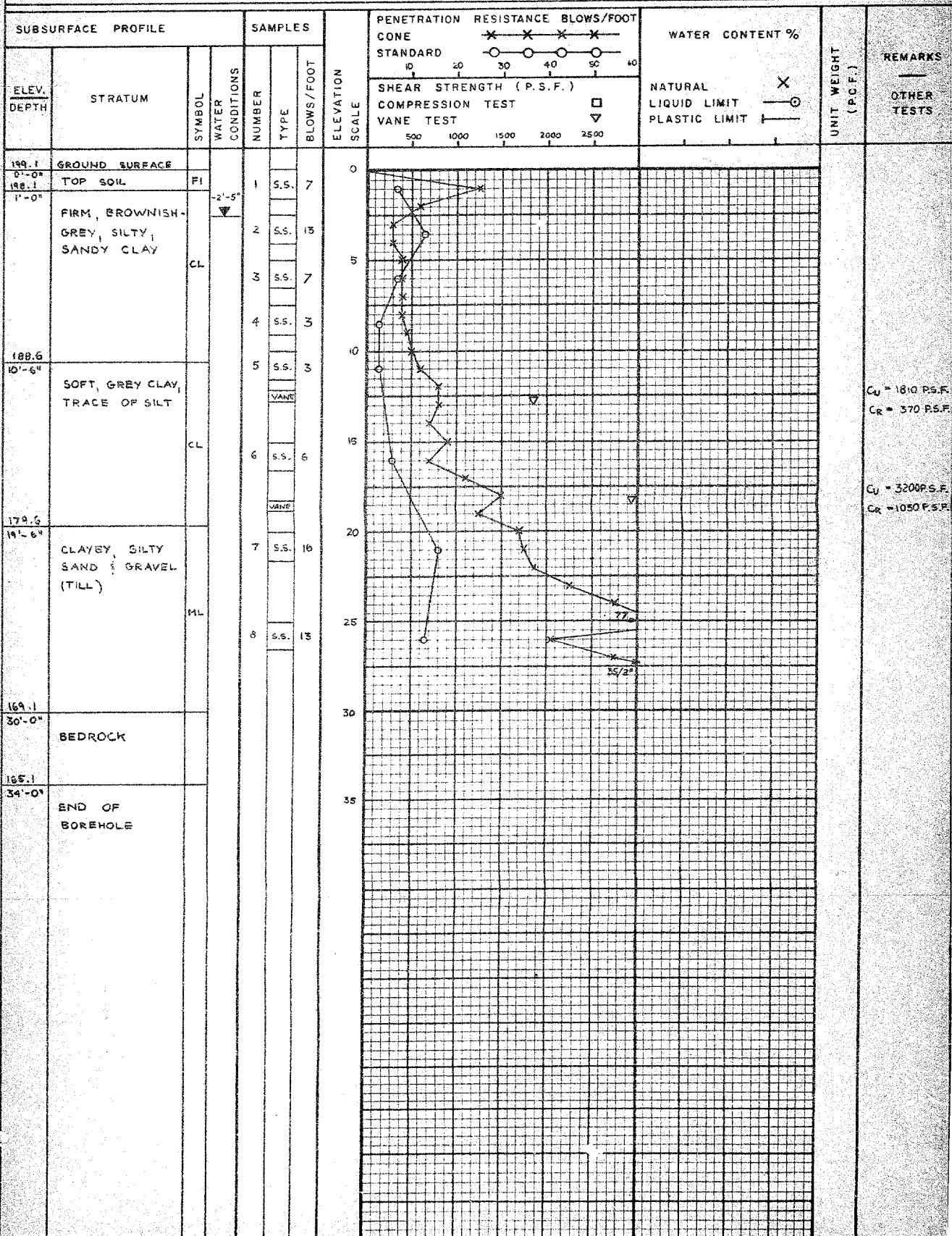
BORING METHOD

WASHING WITHIN 8" CASING

FIELD SUPERVISION BY J.M.G. DATE NOV. 13/62

LOG COMPILED BY I.P.L. DATE NOV. 27/62

LOG CHECKED BY DATE



RACEY, MacCALLUM AND ASSOCIATES LIMITED

LOG OF BOREHOLE NO. 17

ORDER NO. 5-632 / T-3432

PROJECT NEPEAN BAY, OUTFALL SEWER, OTTAWA

LOCATION SEE LOCATION PLAN

GROUND ELEVATION 201.0

DATUM GEODETIC

BORING METHOD

WASHING WITHIN 8 X CASING

FIELD SUPERVISION BY J. McG. DATE NOV/12/62

LOG COMPILED BY I. P. L. DATE NOV/27/62

LOG CHECKED BY DATE

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE BLOWS/FOOT		WATER CONTENT %		UNIT WEIGHT (P.C.F.)	REMARKS — OTHER TESTS
ELEV. DEPTH	STRATUM	SYMBOL	WATER CONDITIONS	NUMBER	TYPE	BLOWS/FOOT	ELEVATION SCALE	STANDARD 10 20 30 40 50 60	SHEAR STRENGTH (P.S.F.) COMPRESSION TEST VANE TEST	NATURAL LIQUID LIMIT PLASTIC LIMIT		
201.0	GROUND SURFACE						0					
0'-0"	ASPHALT											
0'-9"	FILL (SILTY SAND)	FI		1	S.S.	17						
198.5				2	S.S.	14						
2'-6"	GREY CLAY, SOME GRAVEL	CL		3	S.S.	8						
193.5				4	S.S.	3						
7'-6"	SOFT, GREY CLAY	CL		5	T.W.	1						
				6	S.S.	3						
				7	T.W.	1						
178.5												
22'-6"	SHALE BEDROCK				AX CORING							
173.2												
27'-10"	END OF BOREHOLE											

PENETRATION RESISTANCE BLOWS/FOOT

STANDARD 10 20 30 40 50 60

SHEAR STRENGTH (P.S.F.)

COMPRESSION TEST

VANE TEST

500 1000 1500 2000 2500

WATER CONTENT %

NATURAL LIQUID LIMIT

PLASTIC LIMIT

UNIT WEIGHT (P.C.F.)

REMARKS

OTHER TESTS

CU = 2530 PS

CR = 520 PS

CU = 2530 PS
CR = 520 PS

APPENDIX "A"

November 29, 1962.

FIELD PROCEDURE

Investigation in the field is carried out by means of a diamond drill rig or a truck mounted power auger unit. The holes are advanced either by augers, tubes or by wash water. If holes cannot be kept open because the soil collapses, casing is driven.

Standard sampling procedures are followed. Samples are recovered ahead of the casing at frequent intervals, generally 5 feet, with either a 2-inch or 3-inch O.D. split barrel sampling tube or Shelby tube.

The standard penetration test results are recorded when sampling with the regular 2-inch O.D. split spoon, these being the number of blows of a 140-pound hammer falling 30 inches, required to drive the sampling tube a distance of one foot into undisturbed soil. The standard penetration resistance "N" bears an empirical relationship with the relative density or the consistency of the soil. Dynamic cone probes are made by using a 2-inch O.D. 60-degree cone point attached to the end of the drilling rods. The cone is advanced into the soil by ramming, using a 140-lb. hammer falling freely 30 inches. The number of blows for each foot of penetration is recorded. The dynamic cone test provides a continuous picture of variation of soil densities.

In soft or firm cohesive soils, undisturbed samples are obtained by pushing a thin-walled Shelby tube into the undisturbed soil. The samples are returned to the laboratory for later examination and testing as required. Where required, the in-situ shear strength of the soil is determined by field vane tests. Disturbed samples are visually classified in the field, sealed in air-tight jars, and are re-examined, and tested as necessary in the soils laboratory.

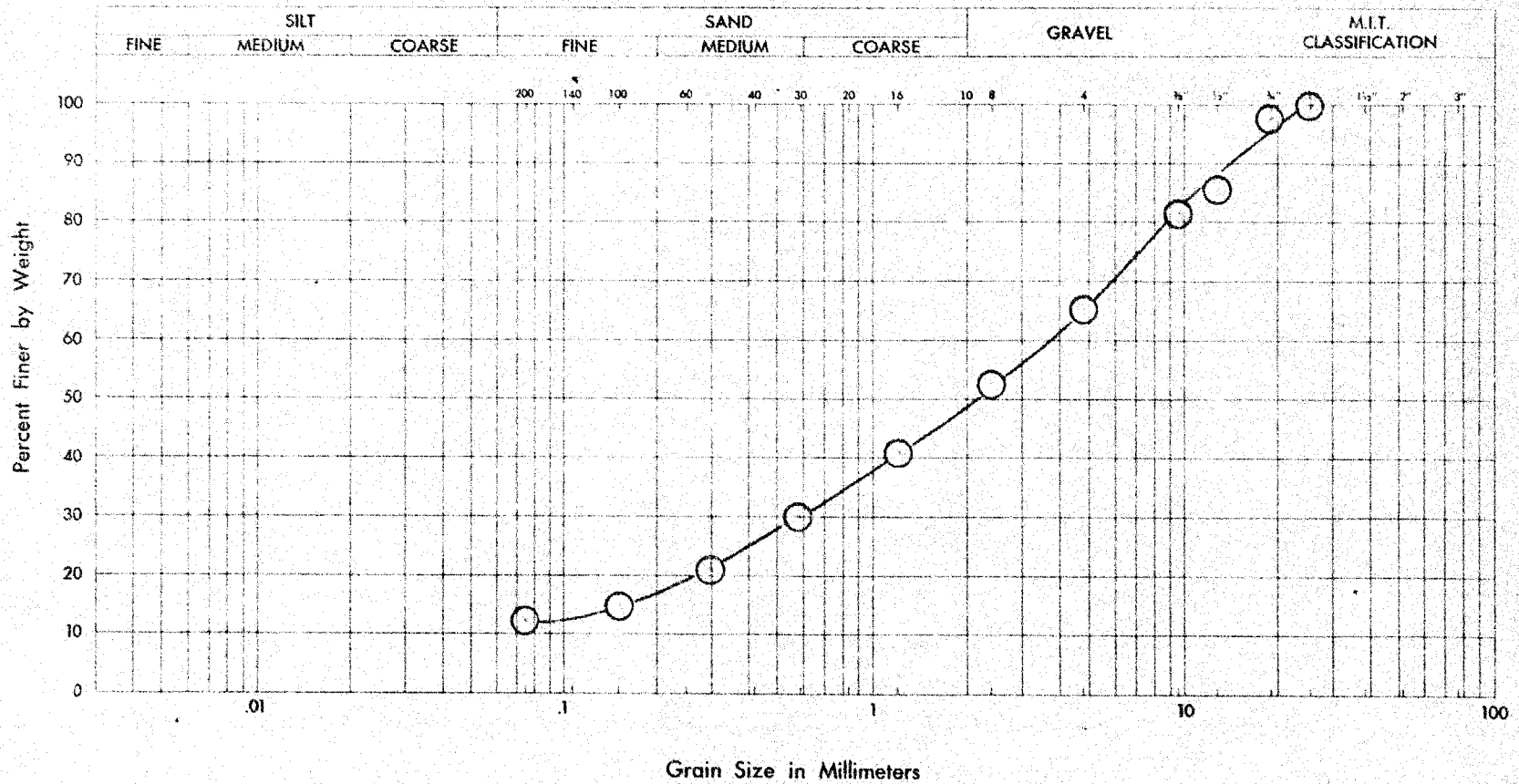
Ground water conditions are observed as follows:

The depth where ground water was encountered is recorded, and the change in water level observed. In case of wash water used the test holes are bailed or pumped out, during the work as necessary, at the end of the day and on completion. Subsequent water level readings are taken for the duration of the field work. When artesian water conditions are encountered the head of water in feet is recorded.

APPENDIX "B"

RACEY MacCALLUM AND ASSOCIATES LTD.

GRAIN SIZE DISTRIBUTION



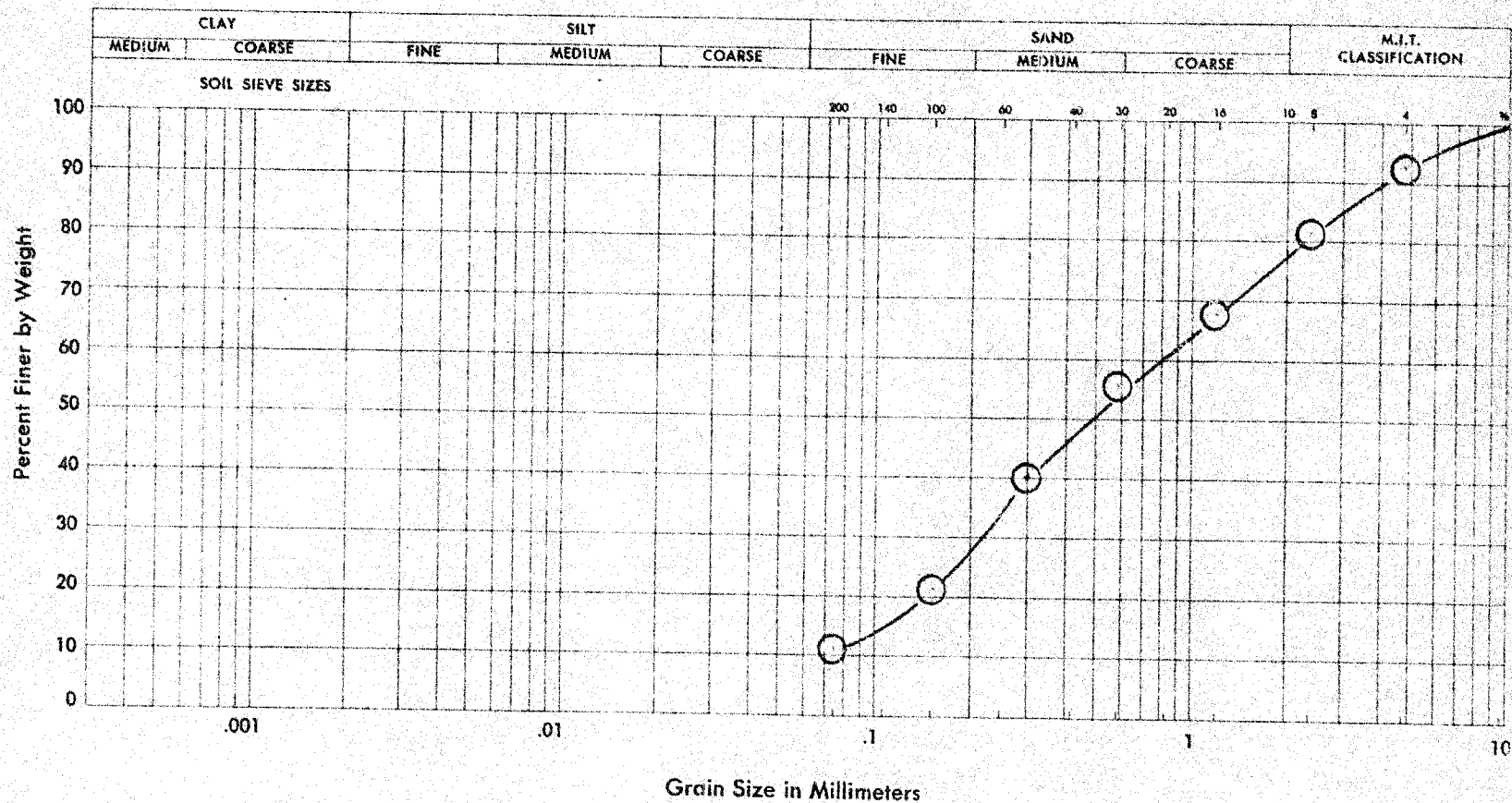
Project NEPEAN BAY OUTFALL SEWER

Legend. BH N° 5 SAMPLE SS-4
DEPTH: 11' 0"

Order No. S-632/T-3432

RACEY MacCALLUM AND ASSOCIATES LTD.

GRAIN SIZE DISTRIBUTION



Project NEPEAN BAY OUTFALL SEWER

Legend BH. #16 SS-7
DEPTH: 20'-21'-6"

Order No. S-632/T-3+32

Enclosure No.

Prep. By

SEEPAGE TESTS - BOREHOLE 5A

(Casing pumped out - water rises in casing)

<u>BX casing depth (feet)</u>	<u>Water depth (feet)</u>	<u>Time (Minutes)</u>	<u>k cm/sec.</u>	<u>BX casing depth (feet)</u>	<u>Water depth (feet)</u>	<u>Time (Minutes)</u>	<u>k cm/sec.</u>
5.0	5.0	0		10.0 (cont'd)	9.2	5	
	4.6	1			9.2	10	1.3×10^{-4}
	4.5	3			9.1	20	
	4.5	4			9.1	30	
	4.5	5	5×10^{-4}	12.5	12.5	0	
	4.4	10			11.4	1	
	4.3	20			11.3	2	
	4.0	30			11.2	3	1.6×10^{-4}
7.5	7.5	0			11.2	4	
	6.8	1			11.1	5	
	6.7	2			11.0	10	
	6.6	3			10.9	20	
	6.6	4	5×10^{-4}		10.9	30	
	6.5	5		15.0	15.0	0	
	6.2	10			13.9	1	
	5.9	20			12.8	2	
	5.5	30			12.3	3	3×10^{-3}
10.0	10.0	0			11.5	4	
	9.6	1			10.8	5	
	9.4	2			7.7	10	
	9.3	3			4.2	20	
	9.3	4			3.0	30	

Casing at 15.0 feet, borehole at 20.5 feet: pump (25 gpm)
unable to lower water below 6 feet depth in casing.

 6.6×10^{-2}

Prep. By

SEEPAGE TESTS - BOREHOLE 5B

(Water supplied - level drops in casing)

<u>BX casing depth (feet)</u>	<u>Water Level (feet)</u>	<u>Time (Minutes)</u>	<u>$\frac{k}{cm/sec.}$</u>
3	1.3 above ground (casing top)	0	
	1.8 below casing top	1	
	2.5 " " "	2	
	2.8 " " "	4	
	3.1 " " "	8	
6	2.7 above ground (casing top)	0	
	8.3 below casing top	1	
	8.3 " " "	2	
9	1.2 above ground (casing top)	0	
	1.3 below casing top	1	
	1.8 " " "	2	1.8×10^{-3}
	2.5 " " "	4	
	2.9 " " "	8	
12	0.7 above ground (casing top)	0	
	6.7 below casing top	1	
	7.7 " " "	2	
	7.9 " " "	4	
	7.9 " " "	8	
15	1.3 above ground (casing top)	0	
	1.6 below casing top	1	
	2.9 " " "	2	7×10^{-3}
	3.7 " " "	4	
	5.1 " " "	8	
18	0.3 above ground (top of casing)	0	
	1.0 below casing top	1	
	1.6 " " "	2	
	2.3 " " "	4	2×10^{-3}
	2.9 " " "	8	
19.2 (casing refusal)	-- Water outflow too rapid to measure.		

Prep. By

PUMPING TESTS - BOREHOLE 6A

<u>BX Casing Depth (feet)</u>	<u>Pumping Rate gpm.</u>	<u>Time (Min.)</u>	<u>Water depth in 6A</u>	<u>Water Depth in Observation Hole at 25 feet away</u>
10	1	0	10	4.1
		2	10	4.3
		10	10	4.35
15	5	0	5.5	5.2
		10	5.5	
		20	5.7	5.2
		30	5.7	
		90	6.0	

 $k = 1.4 \text{ cm/sec.}$

Could not lower water further.

Borehole advanced to 20.3 feet with casing to rock.
 AX core to 26.1 feet. Well-point was lowered to 20.7 feet at tip,
 with intent of pumping out hole with larger capacity. Attempted to
 pull casing to render well-point effective. Casing broke and 6 foot
 section lost. Well-point recovered by hammering out. Hole caved
 to 5 feet, trapping casing section at depth of 12 to 18 feet.
 Hole abandoned.

Prep. By

PUMPING TEST - BOREHOLE 6B (18"Ø)

(Hole pumped to 18.5 feet - water rises in casing)

<u>Date and Time</u>	<u>Water depth in 6B</u>	<u>Water depth in observation hole 10 ft. away</u>	<u>Water depth in BH.6 - 25 ft. away</u>
Dec. 19 - 5 p.m.	18.5	14.2	6.1
Dec. 20 - 9 a.m.	14.7	11.8	6.0
Dec. 22 - 9.35 a.m.	7.1	6.5	6.0

All three holes at same elevation.

12" diameter casing surrounded by gravel and slotted at bottom, with plastic mesh screen.

<u>Elapsed time (min.)</u>	<u>Water depth below GWT in BH - 6B</u>
0	15.8
2	15.6
4	15.6
6	15.5
8	15.5
10	15.5
15	15.4
20	15.4

Prep. By

SEEPAGE TESTS - BOREHOLE 6C

(Water supplied - level drops in casing)

<u>BX casing depth (feet)</u>	<u>Water level (feet)</u>	<u>Time (minutes)</u>
8	2.0 above ground (casing top)	0
	0.1 below casing top	1
	0.1 " " "	2
	0.1 " " "	5
	0.1 " " "	10
10	2.0 above ground (casing top)	0
	4.5 below casing top	1
	5.0 " " "	2
	5.3 " " "	5
	5.4 " " "	10
11	1.5 above ground (casing top)	0
	7.3 below casing top	1
	7.4 " " "	2
	7.4 " " "	5
	7.4 " " "	10

16 & 18.3 - 1200 gpm just kept casing filled 3 feet above ground, i.e. maintained at 9 foot head above water table.

$k = 20 \text{ cm/sec.}$

Originally, tests were attempted by pumping out the hole and letting water rise in casing, so as to be comparable to boreholes 13 and 14. The pump was too large however (due to expectations based on results of borehole 6A), and emptied the hole in intermittent blasts of water so that steady state conditions could not be reached. Hence infiltration tests were done instead.

Prep. By

WELL-POINT PUMPING TEST - BOREHOLE 6D.

Description: Wellpoint hole (6D) was 17.5 feet deep, 6" diameter, filled with sand. Top of wellpoint 13.4 feet below surface. Two 6" diameter observation holes were drilled, one 17 feet deep at 10 feet away and one 12 feet deep at 20 feet away from 6D. Each hole had BX casing in centre and was filled with sand.

Results:

	<u>Elapsed Time (hours)</u>	<u>Wellpoint Production</u>	<u>Water depth below top of casing</u>	
			<u>10 ft. away</u>	<u>20 feet away</u>
4.55 p.m., Jan. 24, 1962	0		6.4 feet	5.7 feet
	0.6	45 gal. in 3 min.	6.9	6.1
	4.0	" " " "	7.3	6.5
	6.2	45 gal. in 4.1 min.	7.5	6.7
	14.6	45 gal. in 8 min.	7.5	6.7
10.30 a.m., Jan. 25, 1962	17.6	45 gal. in 12 min.	7.5	6.7

Total drawdown 10 feet away = $13\frac{1}{2}$ inches.

20 feet away = 12 inches.

$$k = 2 \times 10^{-2} \text{ cm/sec.}$$

Note: A previous test not under our direct supervision, on Jan. 15, 1962, had been thought to be unsuccessful due to incorrect water level measurements. The results above were recorded by our Mr. I. G. Bowie with the test under his direct supervision.

Prep. By

SEEPAGE TESTS - BOREHOLE 13

(Casing pumped out - water rises in casing)

<u>BX casing depth (feet)</u>	<u>Water depth (feet)</u>	<u>Time (Minutes)</u>	<u>k cm/sec.</u>	<u>BX casing depth (feet)</u>	<u>Water depth (feet)</u>	<u>Time (Minutes)</u>
20	19.0	0	4×10^{-4}	15	15.0	0
	17.3	1			12.3	1
	17.1	2			11.3	2
	17.0	3			10.1	3
	16.9	4			9.2	4
	16.7	5			8.8	5
	16.2	10			7.0	10
	15.8	15			6.6	15
25	25.0	0	3×10^{-4}	10	10.0	0
	22.3	2.5			6.7	1
	21.9	5			6.6	2
	21.2	10			6.4	3
	20.5	15			6.3	4
	20.0	0			6.2	5
20	17.5	1			5.9	10
	16.9	2			5.9	15
	16.5	3				
	15.9	4				
	15.4	5				
	13.3	10				
	12.0	15				

Prep. By

SEEPAGE TESTS - BOREHOLE 14.

(Casing pumped out - water rises in casing)

<u>BX casing depth (feet)</u>	<u>Water depth (feet)</u>	<u>Time (Minutes)</u>	<u>BX casing depth (feet)</u>	<u>Water depth (feet)</u>	<u>Time (Minutes)</u>
11	10.0	0	24	24.0	0
	7.1	2.5		19	1
	5.3	5		15.7	5
	5.3	10		13.7	10
	5.3	15	20	20.0	0
15	15.0	0		16.3	1
	13.4	2.5		14.1	2.5
	13.2	5		12.8	5
	13.0	10		11.2	10
	12.9	15	15	15.0	0
20	20.0	0		12.8	1
	17.2	2.5		12.0	2.5
	17.0	5		11.1	5
	16.3	10		10.1	10
	15.8	15			

Prep. By

WATER TABLE READINGS IN BOREHOLES

<u>Borehole No.</u>	<u>Water Surface</u>		<u>Remarks</u>
	<u>Depth</u>	<u>Elevation</u>	
5			
6	5.4	174.6	
7			
8	8.5	177.4	
9	dry		
10	dry		
11	dry		
12	dry		
5A	2.5	174.2	
5B	5.7	174.8	
6A	5.5	176.1	
6B			
6C	5.9		
6D			
13	7.8	175.0	
14	5.2		
C-17	7.5	172.8	Patterson Borehole
C-18	7.0	173.8	Patterson Borehole
15	2.5	188.4	
16	2.4	196.7	
17	20.0	181.0	
110		176.8	McRostie Borehole
109		177.5	McRostie Borehole
101		178.3	McRostie Borehole
117		181.9	McRostie Borehole

NO. 340R
EUGENE DIEZGEN GRAPH PAPER CO.
MADE IN U.S.A.

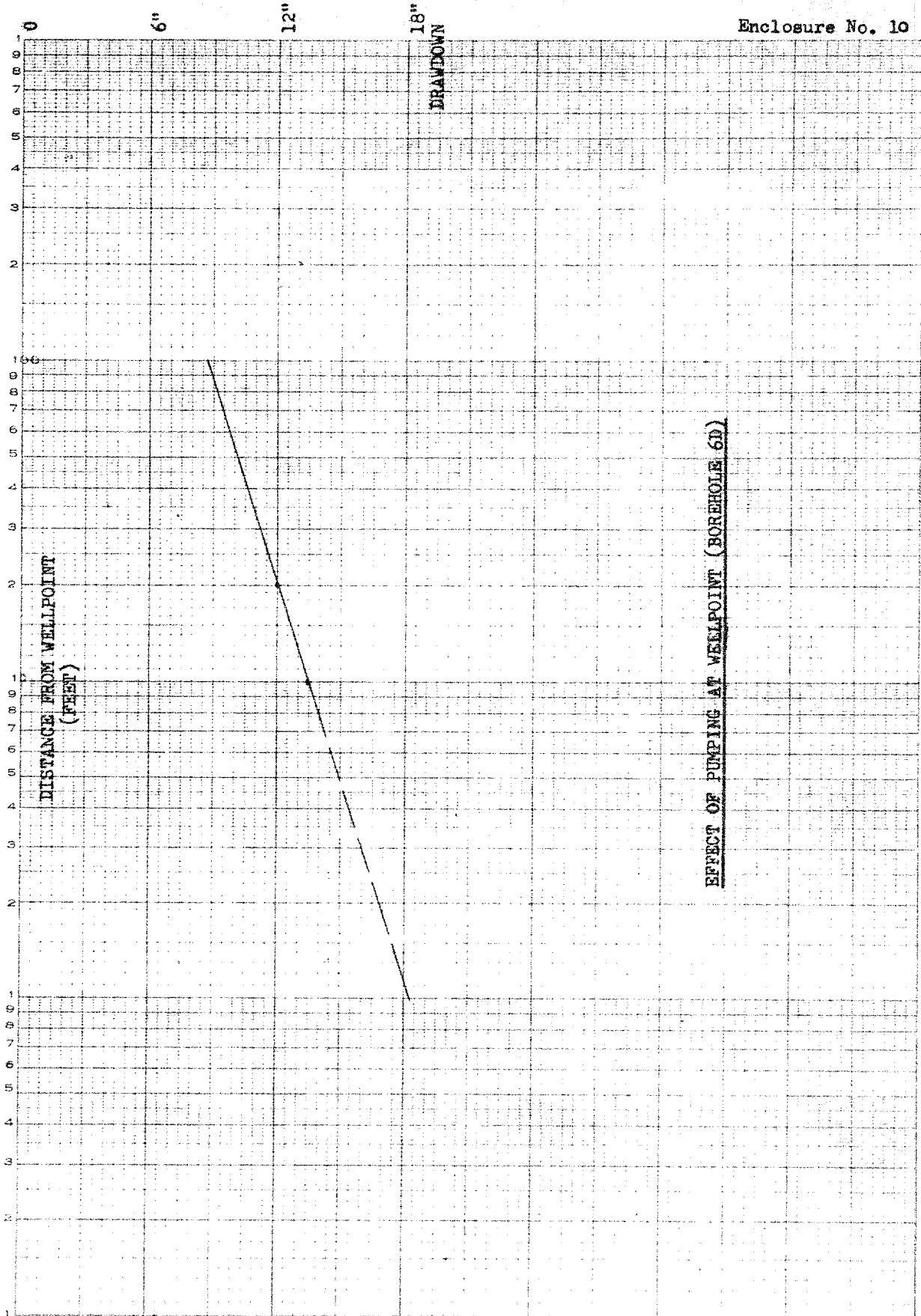
NO. 340R
EUGENE DIEZGEN GRAPH PAPER
MADE IN U.S.A.
4 CYCLES x 12 DIVISIONS PER INCH

Enclosure No. 10

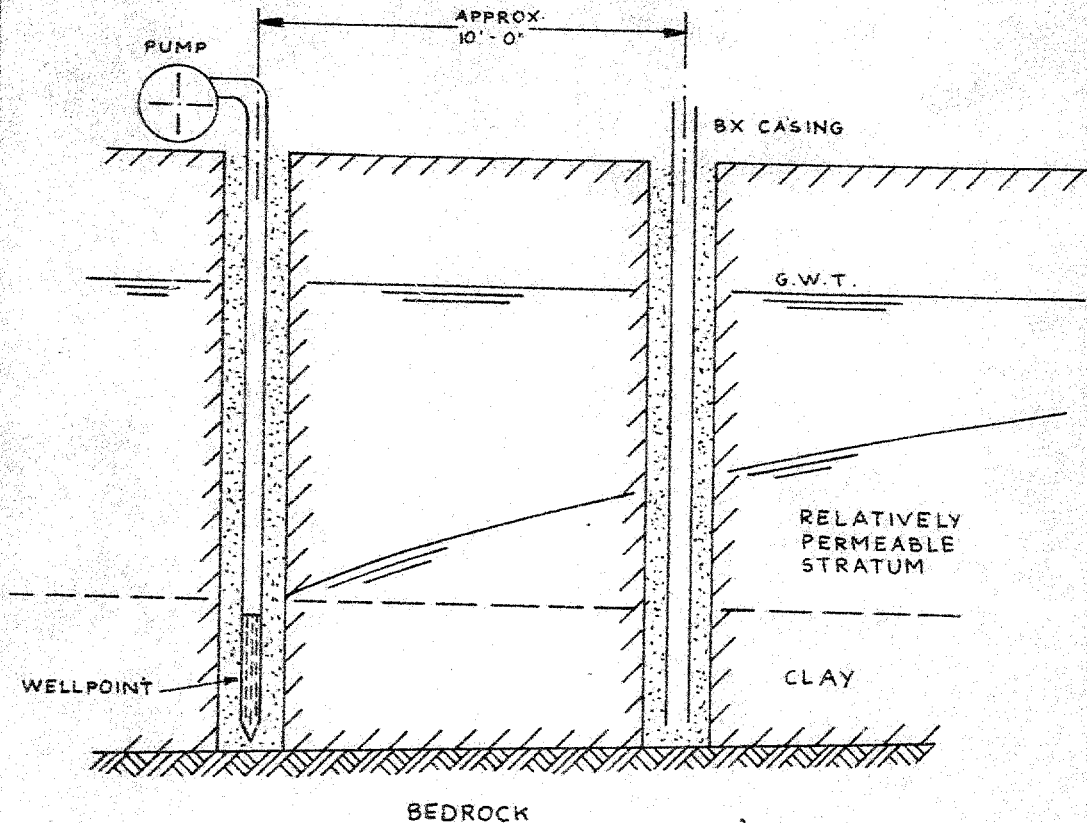
DISTANCE FROM WELLPOINT
(FEET)

18"
DRAWDOWN

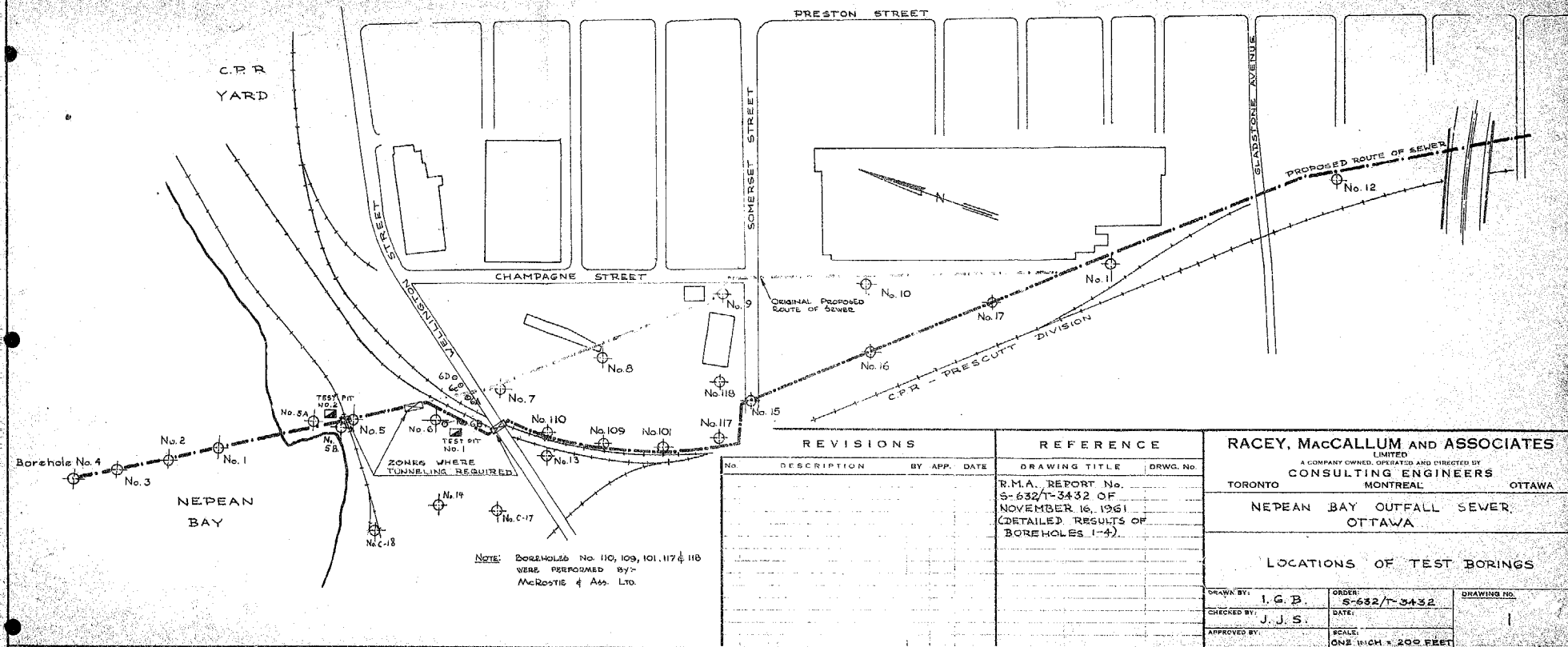
EFFECT OF PUMPING AT WELLPOINT (BOREHOLE 6D)



Prep. By



METHOD OF INSTALLING
WELLPOINT AND
OBSERVATION HOLE
NEPEAN BAY OUTFALL SEWER
OTTAWA



ELEV.

210

200

190

180

170

160

150

B.H. 5A.
176.7

B.H. 5B
B.H. 5C
B.H. 5
180.5

B.H. 6
180.0
B.H. 6B
180.5

B.H. 6A
181.6

WELLINGTON
STREET

B.H. 10
182.5
B.H. 13
182.8

B.H. 109
182.2

B.H. 101
189.5

B.H. 107
190.9

B.H. 15
190.9
SOMERSET
STREET

B.H. 16
191.1

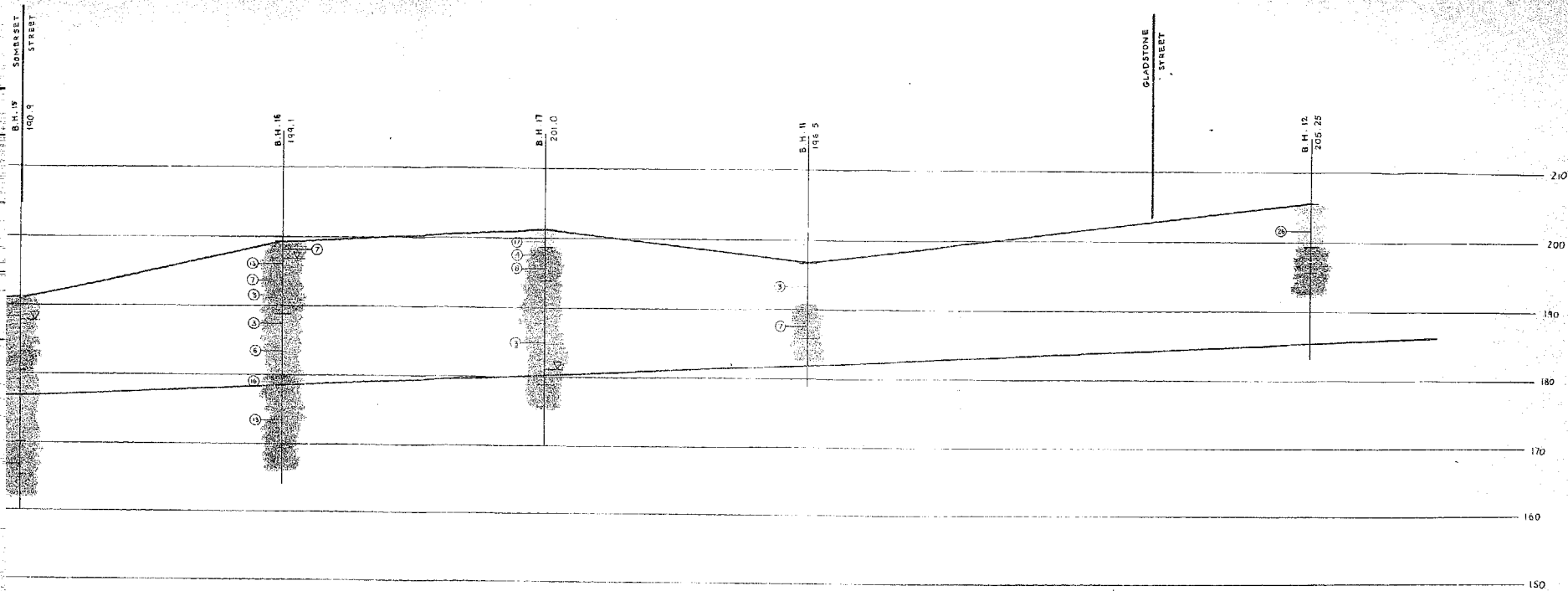
○ BLOWS / FOOT

▽ WATER ELEVATION

ORGANIC PEAT

GRANULAR FILL

2



LEGEND

- | | | | | | | | |
|--|---------------|--|----------------------------------|--|------------------|--|---------------------------|
| | ORGANIC PEAT | | DENSE SAND, GRAVEL & BOULDER | | FIRM, SILTY CLAY | | SHALE & LIMESTONE BEDROCK |
| | GRANULAR FILL | | LOOSE TO VERY DENSE GLACIAL TILL | | SOFT, SILTY CLAY | | |

**PROFILE OF BOREHOLES
NEPEAN BAY OUTFALL SEWER
OTTAWA**

DRAWING NO. : 2
ORDER NO. S-632/T-3432

SCALE : H. : 1" = 100'
V. : 1" = 10'

DATE : NOV. 29. 62