

W.O. 70-F-211 M

VICTORIA STREET

BRIDGE

REPLACEMENT

40116-14

~~401-190~~



40116-14
GEOCRESS No.

70-F-211M

FOUNDATION INVESTIGATION FOR
PROPOSED VICTORIA STREET BRIDGE REPLACEMENT
SIMCOE, ONTARIO

COUNTY OF NORFOLK

STRUCTURE SITE No. 20-73

Prepared for:

JEWITT, DODD AND VALLEE

WILLIAM TROW ASSOCIATES (HAMILTON) LTD.

Project: H761
February 5th, 1970

198 Superior St.
Hamilton, Ontario
547-6385

STRUCTURE SITE No. 20-73

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SUMMARY

A foundation investigation was performed at the site for the proposed new Lynn River Crossing in Simcoe, Ontario. The purpose of the investigation was to assess the subsurface conditions, to provide foundation design criteria and recommendations regarding foundation construction procedures.

The soils encountered were quite variable, but generally may be grouped into:

1. Fill - consisting of silt, sand and gravel, contaminated with pieces of wood, construction materials and peat.
2. Silts and Sands - dense to very dense, stratified, occasionally containing gravel and thin layers of clay.
3. Silty Clay - firm to very stiff, stratified.

The water levels observed within the project area generally varied from Elevation 680 to 684 feet.

The new proposed river crossing may be constructed at the location of the existing structure, or at the new location, as shown on Drawing 1. It may be founded on spread footings at either location at Elevation 670 feet, or at Elevation 664 feet. The allowable bearing capacity of the soil at either elevation is 3 tsf at the existing structure, and 4 tsf at the new structure. Closed sheeted excavations will be required if the structure is founded at Elevation 664 feet. Alternatively, artificial scour protection must be provided if it is founded at Elevation 670 feet.

If the proposed crossing is constructed at the new location, the Lynn River will be diverted. Excavation for the new river channel can be accomplished by digging a pilot trench and gradually lowering the water level in it. The trench may then be extended to within 25 feet of the existing river in the dry. The river banks may be protected with rubble-stone walls.

Excavations for underground services may require closed sheeting or shoring, depending on their locations relative to the Lynn River or existing structures.

The retaining wall in front of the Brook Woollen Mill may be constructed on compacted fill at a safe bearing capacity of 1 t.s.f.

Foundation Investigation
Victoria Street Bridge, #6932
Board of Works Project 69-8, Simcoe, Ont.

INTRODUCTION

A foundation investigation was undertaken by William Trow Associates (Hamilton) Limited at the site of the proposed new Victoria Street Bridge across Lynn River in Simcoe, following authorization received from Jewitt, Dodd and Vallee Limited, in November of 1969.

It is understood that a proposed crossing will be a simply-supported structure located approximately 100 feet east of the existing structure, and that the new deck elevation will be raised to achieve a better vertical alignment. Associated with the new river crossing, will be the excavation for the river diversion, the relocation of underground services, and the construction of a small retaining wall in front of the Brook Woollen Mill, parallel to Victoria Street.

The purpose of the foundation investigation was to assess the soil and groundwater conditions at the proposed site, to provide geotechnical data for the preparation of foundation design criteria and recommendations regarding foundation construction procedures.

PROCEDURE

The fieldwork was performed by F.E. Johnston Drilling Co. Ltd., under the supervision of representatives of William Trow Associates (Hamilton) Limited, between November 27th and December 5th, 1969. The fieldwork was carried out with the use of diamond drilling equipment, and consisted of drilling nine boreholes, varying in depth from 12 to 80 feet, soil sampling at intervals as indicated in the borehole logs, making groundwater observations in the boreholes, and obtaining dynamic cone penetration resistance counts at two locations to depths of 19 and 23 feet. The locations of the boreholes and cone penetration tests are shown on

Drawing No. 1. A piezometer was also installed 5 feet west of Borehole 6 at Elevation 665.6 feet.

The soil samples were obtained using the Standard Method for Penetration Testing and Split-Barrel Sampling of Soils, ASTM Designation, D 1586-67. These samples were retained for visual and textural classification, and for water content and unit weight determinations in the laboratory.

The locations and groundsurface elevations of all boreholes and dynamic cone penetration tests were surveyed by Trow personnel. The elevations were referred to the geodetic surveys of Canada bench mark, located at the northwest corner of Pond and Victoria Streets.

SITE AND SUBSURFACE CONDITIONS

The flood plain of the Lynn River is a relatively flat tract of land south and west of the existing river crossing. The toe of the C.P.R. embankment, north of Victoria Street is located near the left bank of the river, from where the embankment rises rapidly to Elevation 692 feet approximately, of the order of 20 feet above the existing river bottom. Further northeast of the C.P.R. and Victoria Street, the groundsurface elevation rises again an additional 30 feet approximately.

The Lynn River is a meandering stream that has eroded and redeposited material within the reach of its flood plain in the geological past. The present river channel varies in width from approximately 40 to 60 feet, and its riverbed is at Elevation 672 feet approximately at the existing bridge. High water level has been observed above Elevation 682 feet.

The Brook Woollen Mill is located immediately south of Victoria Street between the existing river channel and the C.P.R.

The soil strata encountered in the boreholes were quite variable as anticipated from surficial evidence. These are detailed in the borehole logs, and in the interpreted subsoil stratigraphy shown on Drawing No. 1.

Except in Borehole 4 and 5, fill, consisting of loose silt, sand and gravel in varying proportions, occasionally also containing pieces of wood and brick, was found below ground surface in excess of 10 feet. Immediately underlying the fill, peat was found in Holes 3, 6 and 8 to approximately Elevation 670 feet.

Below the fill and peat, various stratified deposits of dense to very dense silts and sand, occasionally containing gravel and thin layers of clay, were encountered. In Holes 1, 2 and 3, grey, firm to very stiff stratified and laminated silty clay was encountered, and in Borehole 3, grey, fine sand with traces of silt beneath the silty clay.

In the immediate vicinity of the new proposed Lynn River crossing, Boreholes 1 and 2 were advanced to 80 and 40 feet respectively. Below the fill and peat, extending to approximately Elevation 675 feet, various strata of silts and sands were encountered to Elevation 648 feet on the west side of the river, and to a higher elevation of 657 feet on the east side. The average standard penetration resistance in the silts and sands below Elevation 670 is approximately 40 blows per foot, and in the silty clay, is approximately 35 blows per foot.



The dynamic cone penetration tests confirm the very significant increase in density of the strata at approximately Elevation 664.

Groundwater observations, made in the various boreholes, indicate a phreatic line which rises in elevation with distance away from the existing river. At the time the field exploratory work was performed, the groundwater level within the proposed project area was located at approximately Elevation 680 to 684 feet.

DISCUSSION

As a result of the explorations and fieldwork performed, it is concluded that the new proposed Lynn River crossing may be constructed at its present location or at the proposed new location as shown on Drawing 1. However, the proposed new location is preferred because of higher bearing capacities under spread footings, because of the likelihood of more similar foundation conditions under either bridge abutment, and because of certain construction advantages. The founding level of spread footings at either location, could be at Elevation 670 feet or at Elevation 664 feet. At Elevation 670 feet, scour protection would have to be provided, whereas at Elevation 664 feet the excavation for the footings will have to be dewatered.

The allowable bearing pressure of the soil is 3 tsf at both Elevation 670 and 664 feet at the existing location, and is 4 tsf at these elevations at the proposed new position. Under these bearing pressures, the total and differential settlements should be well within limits tolerated for simply-supported structures.

Relocating the new river crossing to the east of the existing crossing will necessitate the permanent diversion of the Lynn River channel as shown on Drawing 1. This will require excavations for the new channel, a certain amount of bank protection, and depending on the founding level of the bridge abutments, scour protection.

(a) Excavations

(i) New Proposed Lynn River Channel

The excavations for the new proposed Lynn River channel should be carried out to at least the same width and depth as that of the existing river channel. The excavation should commence at least 200 feet downstream of the centreline of the new structure. Initially the channel should be opened to its complete width and for a length of at least 100 feet. This excavation will have to be carried out with a backhoe or dragline since it will extend below the water table. The side slopes should be allowed to cave to their natural slopes. When the pilot excavation has been completed, water should be gradually pumped from it to lower the surrounding water table. The excavation should then be extended upstream at the full channel width beyond the new proposed structure, and pumping of water should continue. A berm should be left in place at the U/S and D/S limit of the excavation. To ensure a stable channel bed for possible construction and excavation traffic, ditches should be excavated parallel to and adjacent to both channel banks, to lower the water level below the traffic lanes. A typical channel cross-section is shown on Drawing 13.

(ii) Bridge Abutment Foundations

The groundwater table in the vicinity of the proposed bridge abutment foundations will be significantly lower, after the channel excavations have been carried out as discussed in the previous section. In fact, if the bridge abutment foundations are located at Elevation 670 feet, the excavations for the footings can be accomplished by deepening the general channel depth in the vicinity of the bridge. Care must be taken in this case to keep construction traffic off the footing bed once the excavation is below Elevation 672 feet. Also the excavation should be carefully dewatered with ditches similar to those indicated in Drawing 13, before the final excavation is carried out.

However, if the founding level for the bridge abutments is at Elevation 664 feet, or the foundations at Elevation 670 feet are placed before the new channel is excavated, closed-sheeting will be required. The design of closed sheeting is discussed in Appendix A. For this purpose, the groundwater level should be taken at least 5 feet above the general excavation base, since at depth below the foundation level, water pressure will be influenced by the higher water table at distance away from the excavation.

If the excavations for the bridge abutment foundations are carried out prior to the river-channel excavations, the groundwater level for the design of the closed-sheeting should be taken at Elevation 680 feet. Water that collects within the confines of the closed sheeting can most likely be removed by pumping from sumps in the base of the excavations.

(iii) Underground Services

Excavations for the proposed underground services, everywhere within the project area, except for the sanitary sewer, immediately west of, and parallel to the C.P.R., can most likely be carried out without special shoring. However, the work must be carried out according to regulations stipulated in the 'Trench Excavators' Act', and according to local requirements if any are available.

The work can probably be carried out in such a manner that any water that may seep into the trenches will accumulate at the location of the actual excavation, from where it can be removed by conventional means. Excavations adjacent to the C.P.R. and adjacent to the Lynn River will require shoring if:

1. the base of the excavation is closer to the C.P.R. than a point as defined by the intersection of a line drawn at 2:1 from the shoulder of the C.P.R. embankment, with the elevation line of the base of the excavation.
2. the trench wall on the side of the river is closer to the river than six times the difference in elevation between the level of the water in the river and the base of the excavation below river level.

If the trench excavation is closer to the C.P.R. as defined above in (1) shoring must be designed to support the C.P.R. embankment and any traffic that may be on it. The design of shoring for this purpose is illustrated in Appendix B.

If the excavations are carried out closer to the river, than as defined above in (2), significant amounts of water may seep into the excavations, which may be more than can be removed efficiently by conventional means. In this respect, closed-sheeting may be required. The design of closed sheeting is illustrated in Appendix A.

(b) Bridge Abutment Footings Scour Protection

Artificial scour protection will be required for the bridge abutment footings if they are located at Elevation 670 feet. Spread footings at this elevation will not have sufficient soil cover to provide natural scour protection. The artificial scour protection could be installed after the excavation for the river channel has been essentially completed, but prior to the diversion of the Lynn River. This protection could consist of a type of pavement or a closed-sheet pile wall along the front of the bridge abutment footings. Rip-rap or rubble-stone scour protection may be less preferable since ice action could possibly remove it and therefore allow undermining of the footings, if continued maintenance is not undertaken.

Artificial scour protection in terms of a pavement should consist of a nominally-reinforced concrete blanket, approximately 6 inches thick, perforated with 4-inch holes at 5 foot centres. This pavement should extend across the entire width of the channel, within a reach beyond the upstream and downstream limit of the bridge abutment footings of at least 20 feet. The upstream end of the blanket should be keyed into the riverbed channel to a depth of at least 3 feet.

Alternatively, if scour protection is provided in terms of closed sheeting, it should consist of a wall in front of the bridge abutment footings, driven to at least Elevation 655 feet. It should consist of sufficient capacity sheeting to support the soil under the weight of the bridge abutment footings, should scour take place to a maximum depth at approximately Elevation 664 feet.



(c) Lynn River Bank Protection

It is understood that the Town of Simcoe is quite successful in protecting the banks of the Lynn River Channel with fairly steep rubble-stone walls. Such walls, therefore, should also serve adequately as bank protection for the new river channel.

The construction of these walls should be carried out prior to the diversion of the Lynn River. They should be founded approximately 1 foot below the elevation of the existing riverbed and should be backfilled with granular material of such gradation that it will not be washed out from between the stones used in the construction of these walls. The final surface of the backfill material should be protected from erosion due to surface runoff by sodding with a deep-root type of grass.

(d) Retaining Wall-Brook Woollen Mill

Loose granular fill was encountered in Borehole 9 to a depth greater than 12 feet. Because of the variability in bearing capacity of fill, it is not generally good practice to found a structure in it. However, the retaining wall in front of the Brook Woollen Mill may be founded in the fill, provided the following recommendations are implemented whenever loose fill is encountered below the founding level of the retaining wall. Regardless whether or not fill is encountered, the wall should be founded at a minimum depth of 4 feet below final surface elevation in front of the wall for frost protection.

1. Excavate all fill to a minimum of 2 feet below the proposed founding level, and at least two feet beyond the front of the strip footing for the wall, to ensure that no fill exists within this zone.
2. Compact the foundation soils with five to six passes of a heavy compactor.
3. Backfill to the proposed foundation level with basecourse material conforming to Department of Highways of Ontario specifications for Granular Basecourse Class 'B', and compact to 100 percent of the Standard Proctor Maximum Dry Density.
4. Recommendations about lateral earth pressure, drainage facilities, and the effect of eccentric loading on the bearing capacity, provided in Appendix C should be implemented in the design.

The allowable bearing capacity for the strip footing for the retaining wall should not exceed 1 tsf.

(e) Backfill and Drainage

(i) Bridge Abutment

Backfill behind the bridge abutment should consist of free-draining granular soil compacted to 95 percent of the Standard Proctor Maximum Dry Density. The granular material should conform to the Department of Highways of Ontario specifications for Select Granular Material.

To prevent the build-up of significant hydrostatic pressures behind the bridge abutments, permanent drainage holes should be provided through the bridge abutments at ' ' centres at an elevation approximately 2 feet above low river level. These drainage holes should be protected from being washed out by river action by placing at least 12 inches of 2-inch crushed stone immediately behind the drain holes and surrounded by at least 6 inches of pea gravel everywhere except on the side of the abutment wall. The 2-inch crushed

stone and pea gravel should connect all drain holes.

(ii) Retaining Wall

Backfill and drainage requirements behind the retaining wall should be similar to those recommended for the bridge abutments, except that the centre-to-centre spacing of the drain holes may be increased to 10 feet, and that they should be located approximately 1 foot above the final surface elevation in front of the wall.

(iii) Underground Services

The natural soils excavated for the underground services are quite variable, but may be suitable as backfill. However, this should be checked by performing Standard Proctor Density Tests to ensure that the natural soils can be compacted with reasonable effort. Peat, fill and silt should not be used as backfill material.

If it is required that vehicular traffic be routed over the backfill material in the trenches, periodic sprinkling with water may be required to control dust. During periods of wet weather, the backfill materials may become quite soft under the action of vehicular traffic. Should this be the case, a running surface of at least 12 inches of gravel should be placed over these areas.

All material backfilled in the trenches should be compacted to a minimum of 95 percent of the Standard Proctor Maximum Dry Density. To achieve this, the addition of some water may be required to facilitate compaction.

(f) Road Design

It is understood that Victoria Street will be raised in elevation in the vicinity of the proposed new Lynn River crossing to achieve a better vertical alignment. This may be achieved by placing compactible fill to within $28\frac{1}{2}$ inches of the final grade. The fill will, in general, be compactible if it is within 3 per cent of the optimum moisture content determined by the Standard Proctor Density test. It should be compacted to 95 percent of the Standard Proctor value. The basecourse should consist of 18 inches of Sand Cushion, followed by 6 inches of Granular 'A' crushed stone, and $4\frac{1}{2}$ inches of asphaltic concrete. All basecourse material should conform to the relevant Department of Highways of Ontario specifications, and should be compacted to at least 100 percent of Standard Proctor Maximum Dry Density.



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C.D. Thompson, P.Eng.

DS-gk

Dist: Jewitt, Dodd and Vallee (5)



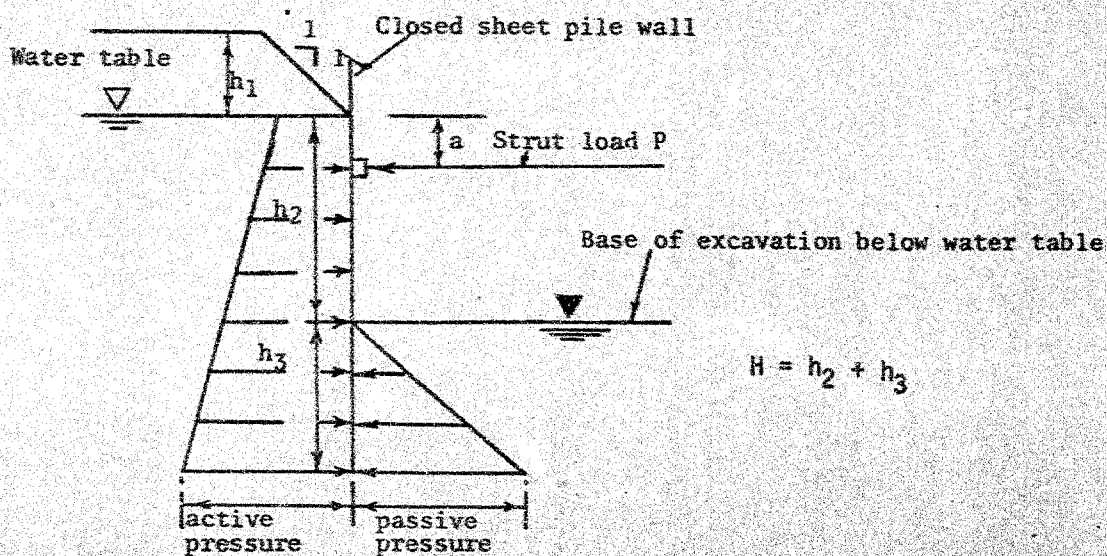
APPENDIX 'A'

DESIGN OF TEMPORARY FLEXIBLE CLOSED SHEET PILE WALL

The construction of spread footings for the abutments of the proposed Lynn River Crossing will require excavations significantly below the water table. These, and other excavations below the water table for this project can be carried out within the confines of closed sheet pile walls.

The design of sheeted excavations requires two major considerations, namely, the earth and water pressures acting on the sheeting, and the seepage pressure tending to boil up the base of the excavation. To ensure the stability of the base of the excavation, it is recommended that the sheet pile wall be driven to a depth at least equal to twice the depth of the excavation below the water table. The design of the sheeting to resist the earth and water pressures may be performed with reference to the diagram and equations below. The most severe criterion should govern the design.

DIAGRAM ILLUSTRATING STRUTTED SHEET PILE WALL





Criterion 1. $h_3 \geq h_2$

Criterion 2.
$$\left\{ \begin{aligned} & q H K_a \left(\frac{H}{2} - a \right) \\ & + \frac{1}{2} H^2 (K_a \gamma'_s + \gamma_w) \left(\frac{2H}{3} - a \right) \\ & - \frac{1}{2} h_3^2 (K_p \gamma'_s + \gamma_w) \left(\frac{2h_3}{3} + h_2 - a \right) \end{aligned} \right\} \leq 0$$

Criterion 3.
$$P \geq q H K_a + \frac{1}{2} H^2 (K_a \gamma'_s + \gamma_w) - \frac{1}{2} h_3^2 (K_p \gamma'_s + \gamma_w)$$

The bending moments and shear in the sheet pile wall may be computed by assuming that the earth and water pressures are linearly distributed on both sides of the wall, as shown in the foregoing diagram, i.e., the active pressure = $K_a (q + \gamma'_s h)$

the passive pressure = $K_p (\gamma'_s h)$

where: q = the value of surcharge, in psf, which may be applied to the ground surface near the wall; for the case illustrated in diagram above,

$$q = \frac{1}{2} \gamma_s h_1$$

H, h_1, h_2, h_3, a = dimensions as defined in the diagram, in feet

P = strut force, in pounds, required to support the sheet pile wall

K_a = the active coefficient of earth pressure
= 0.30.

K_p = the passive coefficient of earth pressure
= 3.0



γ_s = unit weight of soil, in pcf,
above the water table
= 135 pcf

γ_s' = submerged or buoyant unit
weight of soil
= 70 pcf

γ_w = unit weight of water
= 62.4 pcf

APPENDIX 'B'DESIGN OF TEMPORARY SHORING

Temporary shoring for excavations for underground services will be required if they are carried closer to the C.P.R. embankment than a point as defined by the intersection of a straight line, drawn from the shoulder at the top of the embankment with the elevation line at the base of the excavation. This condition is illustrated in the figure below. The earth pressures exerted on the shoring may be computed with the following equation:

$$p = 0.8K (\gamma h + q)$$

where:

it is assumed that the shoring is supported by 2 or more struts, the soil is drained to at least the level of the base of the excavation

and

γ = the unit weight of soil being retained, in pcf
= 135 pcf

q = equivalent unit vertical pressure, in psf, of any surcharge adjacent to the excavation

h = depth, in feet, at which pressure, p , is to be computed

K = the coefficient of earth pressure considered applicable under the conditions defined above
= 0.75

If excavations require shoring, but are not subjected to the weight of the C.P.R. embankment, or any other sloping backfill, the value of the coefficient of earth pressure, K , may be reduced to 0.3.

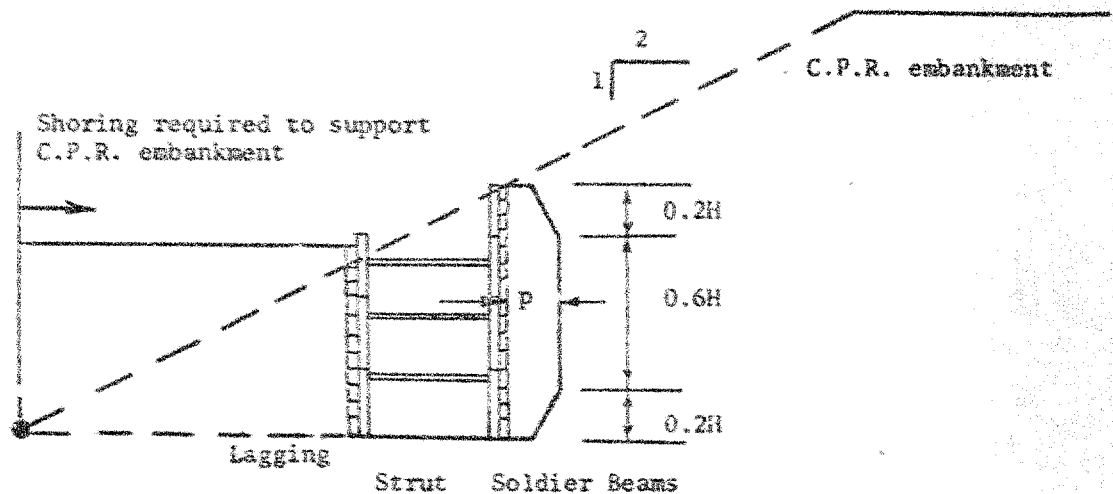


DIAGRAM ILLUSTRATING SUPPORT SYSTEM FOR
BRACED EXCAVATIONS

The pressure should be assumed constant (at value, p) between $0.2H$ and $0.8H$ below the top of the excavation. Above and below the central zone, the pressure, p , is assumed to decrease linearly to zero or to a value equal to $0.8K_q$ if a surcharge is present.

The retained soil must be permitted to drain freely if the above relationships are to remain valid in practice. Spaces should be kept between lagging members to allow drainage of the soil behind the shoring. If soil flows through these spaces, straw or burlap should be inserted which will

hold the soil in place but still permit drainage. If drainage of the soil is not permitted, the earth pressure relationship should be adjusted to allow for the possible build up of hydrostatic pressure behind the shoring.

APPENDIX 'C'Design of Rigid Earth Retaining Structure

The proposed retaining wall in front of the Brook Woollen Mill, and the bridge abutments for the proposed Lynn River crossing will be subjected to lateral earth pressures. If small lateral movements of the structures can be tolerated, the earth pressure behind the walls will reduce to the active case and in front of the wall, increase to the passive case, as the shear strength of the soil is mobilized. Schematic sections of a typical bridge abutment and retaining wall are illustrated in the diagrams below. Both walls are assumed to be backfilled with well-compacted free-draining granular soils. It is also assumed that drainage of the backfill is achieved to the levels shown with the use of drainage tiles.

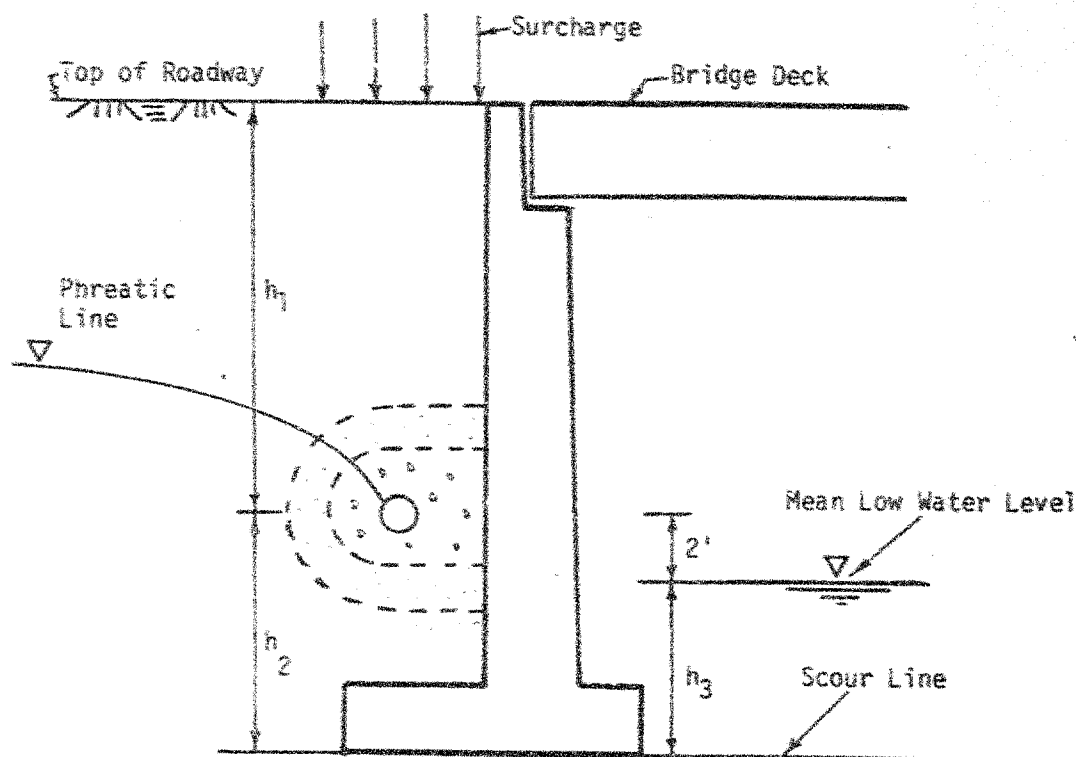


DIAGRAM ILLUSTRATING TYPICAL BRIDGE ABUTMENT

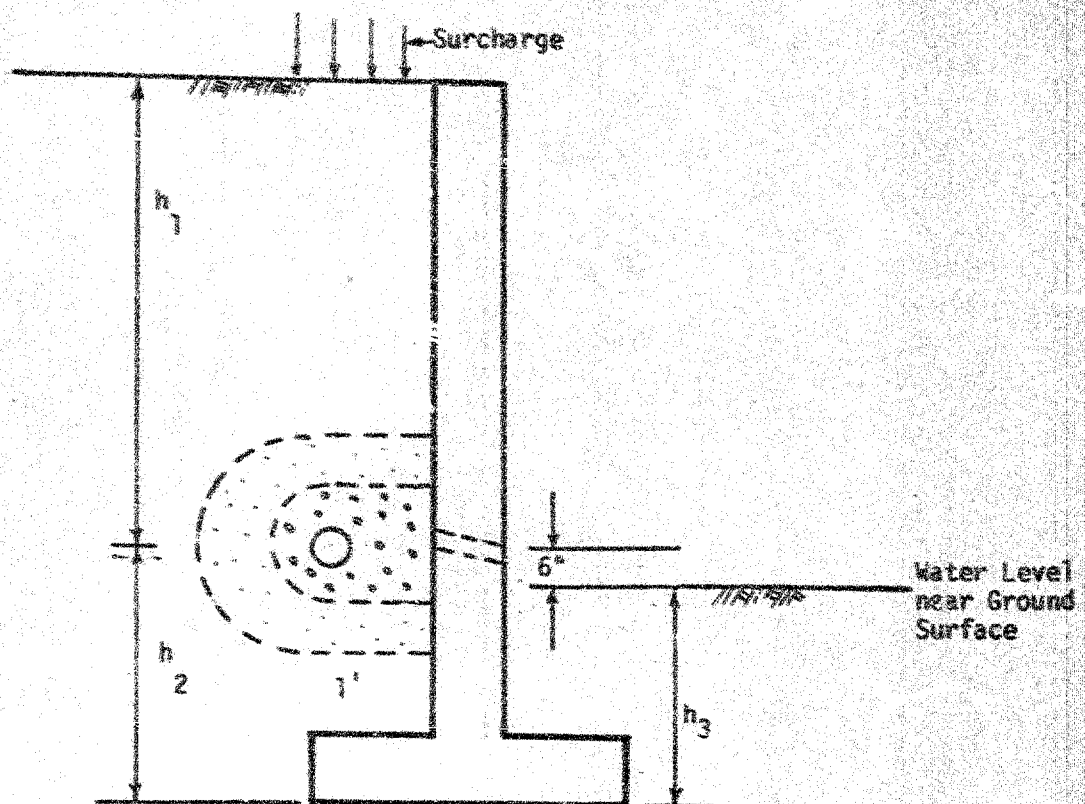


DIAGRAM ILLUSTRATING TYPICAL RETAINING WALL

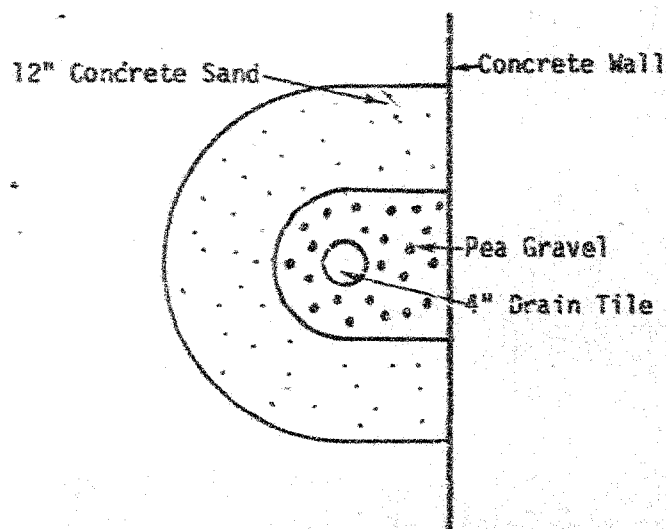


DIAGRAM ILLUSTRATING DRAINAGE DETAILS

The active and passive earth pressures may be assumed to be hydrostatically distributed, and may be computed with the use of the equations below.

$$P_a = K_a (q + h_1 \gamma_1 + h_2 \gamma_2') + h_2 \gamma_w$$

$$P_p = K_p (h_3 \gamma_2) + h_3 \gamma_w$$

where: in the case of the bridge abutments, it is assumed that the soil will be removed to the scour line.

and: K_a = active coefficient of earth pressure considered applicable for this case.

$$= 0.30$$

K_p = passive coefficient of earth pressure considered applicable for this case.

$$= 3.0$$

q = equivalent surcharge in psf.

h_1, h_2, h_3 = dimensions, in feet, as illustrated, in feet, in the diagrams above.

γ_1 = total unit weight of soil above the drain tile.

$$= 130 \text{ pcf.}$$

γ_2' = submerged or buoyant unit weight of soil below the drain tile.

$$= 70 \text{ pcf.}$$

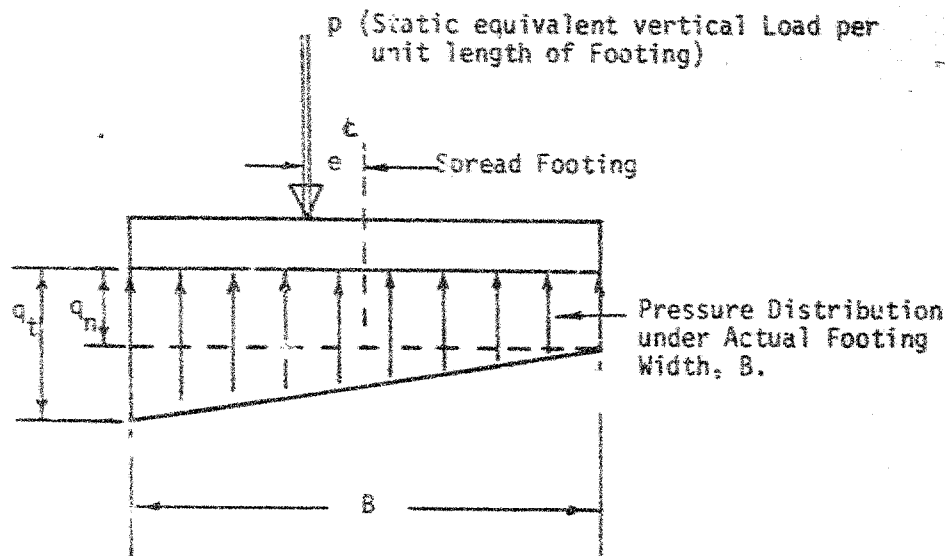
γ_w = unit weight of water.

$$= 62.4 \text{ pcf.}$$

For stability against sliding, the footing should be proportioned such that the 'driving' active pressure is balanced by the 'resisting' passive pressure and the frictional resistance at the base of the footing. The frictional resistance to sliding at the base of the footing may be computed by assuming the angle of static friction to be equal to 40 degrees. A factor of safety of two should be used.

The lateral and vertical loads acting on the footing base may be replaced by an inclined resultant force, or by a statically-equivalent vertical force, P , per applied unit length eccentrically with respect to the width of the footing. As a result, the soil under the footings will be subjected to a trapezoidal pressure distribution, or in the limiting case, to a triangular pressure distribution, as illustrated below.

DIAGRAM ILLUSTRATING METHOD FOR
ESTIMATING ALLOWABLE SOIL PRESSURE BENEATH
ECCENTRICALLY-LOADED FOOTINGS












The limiting contact pressure that may be applied to the foundation soil may be analysed by the 'useful width concept', by which the portion of the footing which is symmetrical about the load, P , is considered useful, and the remaining portion is simply considered superfluous for the convenience of computations.

If the eccentricity of the applied load is e , the useful width is $(B-2e)$. The bearing pressure exerted on the soil under this useful width should not exceed the allowable bearing pressure, q_a .

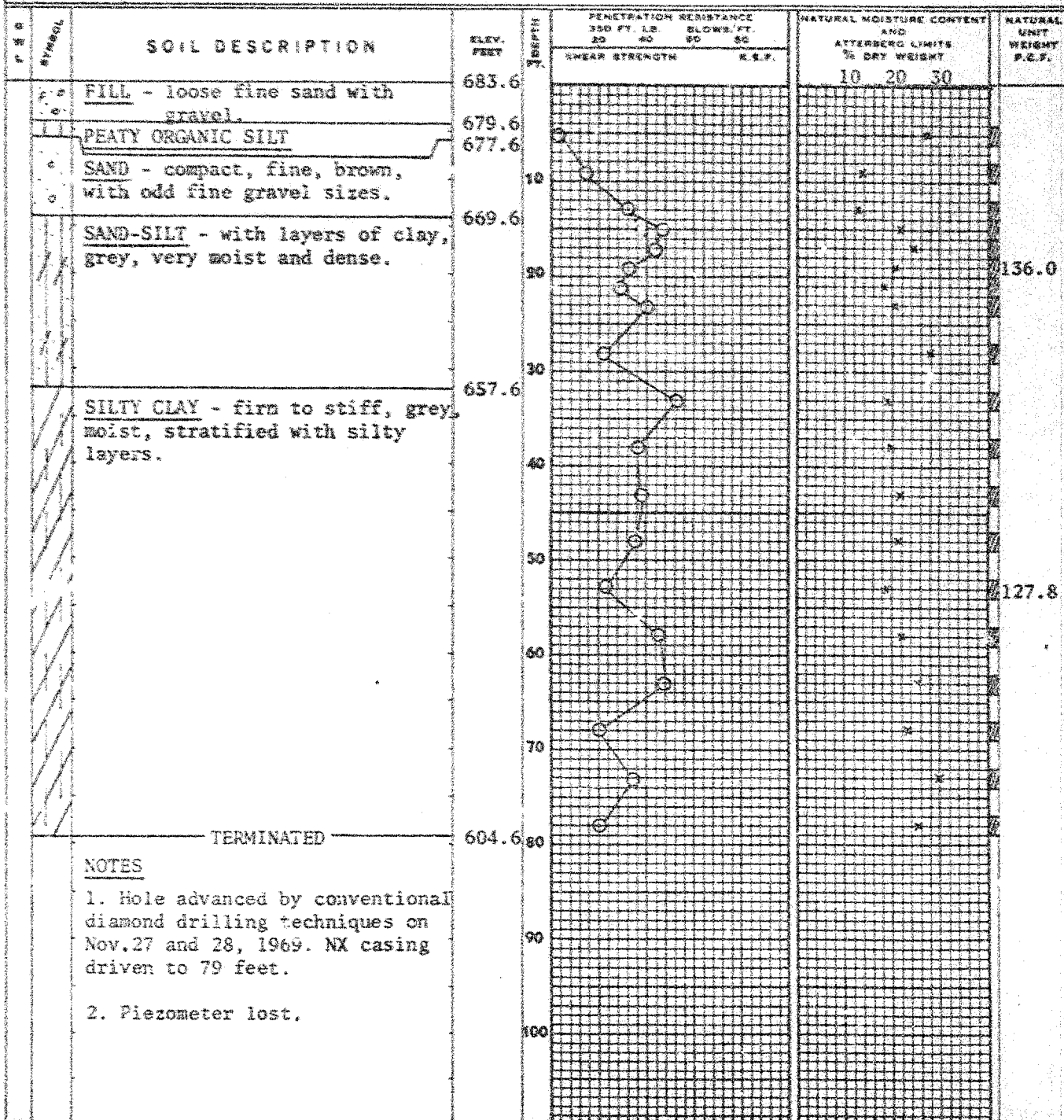
BOREHOLE LOG

JOB No. H76LBOREHOLE No. 1DRAWING No. 2PROJECT Victoria Street BridgeLOCATION Simcoe, Ontario

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 

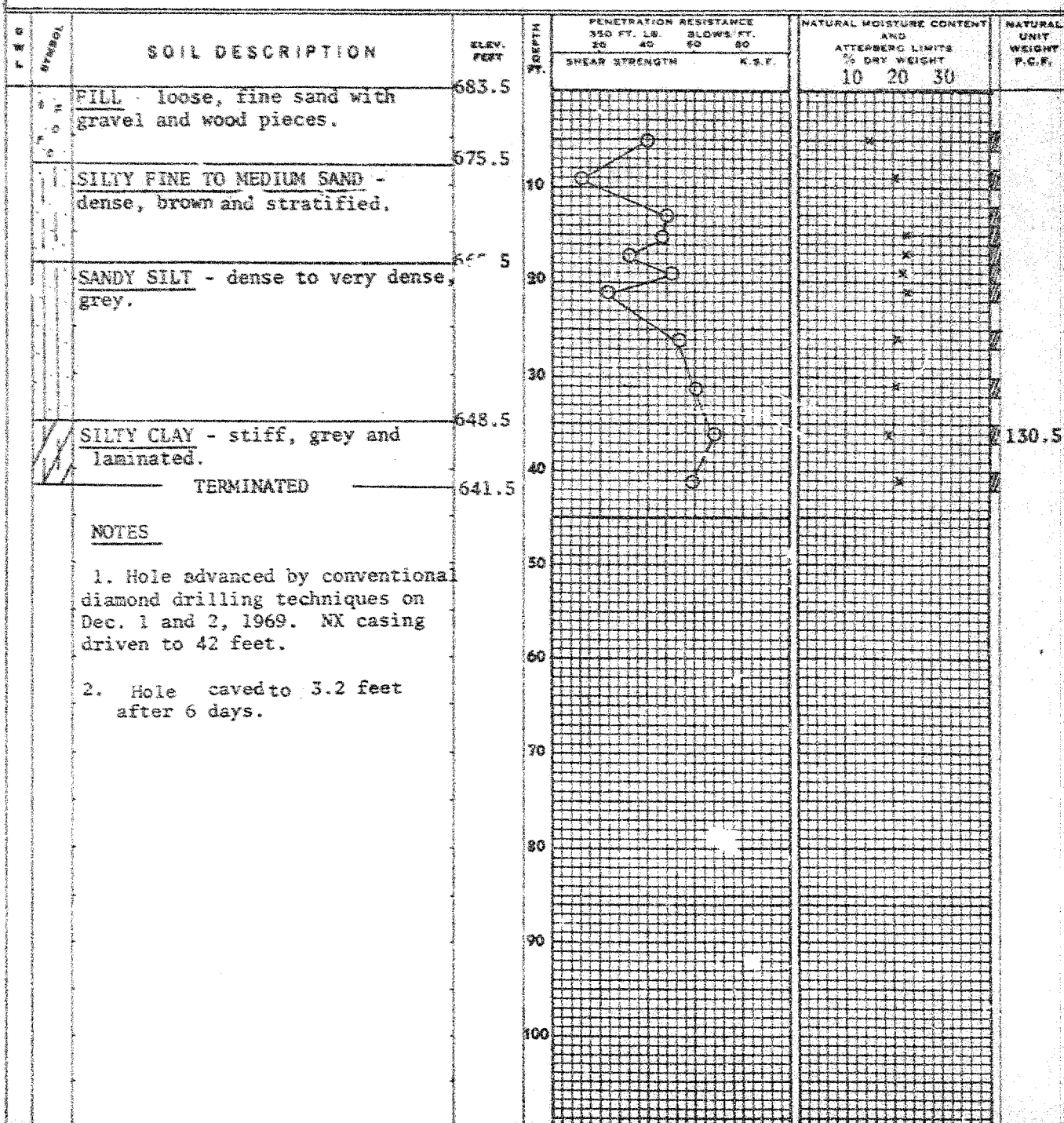
HOLE LOCATION AND DATUM SEE DRAWING NO. 1






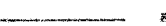

BOREHOLE LOG

JOB No. H761BOREHOLE No. 2DRAWING No. 3PROJECT Victoria Street BridgeLOCATION Simcoe, Ontario2" 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 

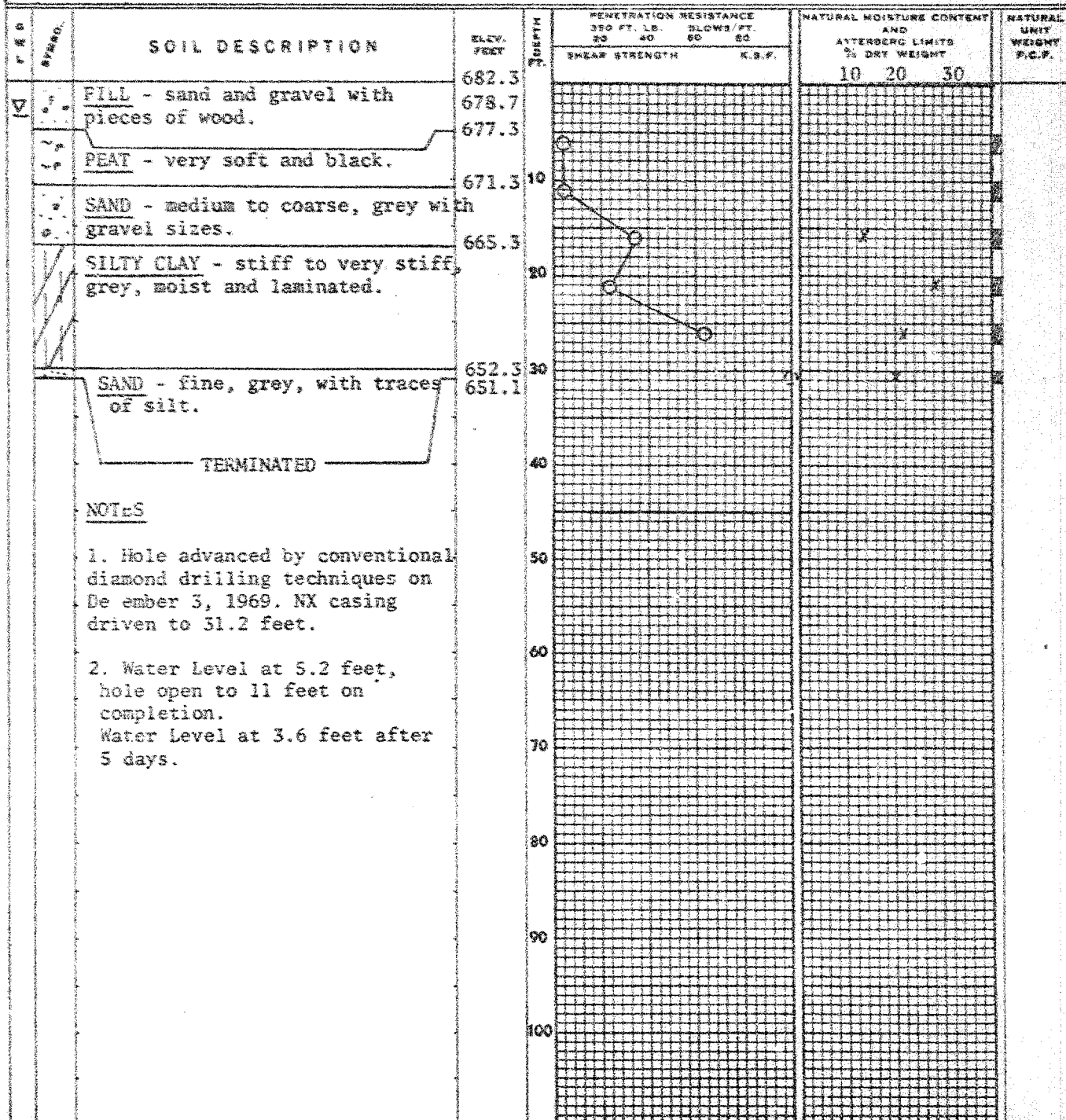
HOLE LOCATION AND DATUM SEE DRAWING No. 1



BOREHOLE LOG

JOB No. H761BOREHOLE No. 3DRAWING No. 4PROJECT Victoria Street BridgeLOCATION Simcoe, Ontario2" 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

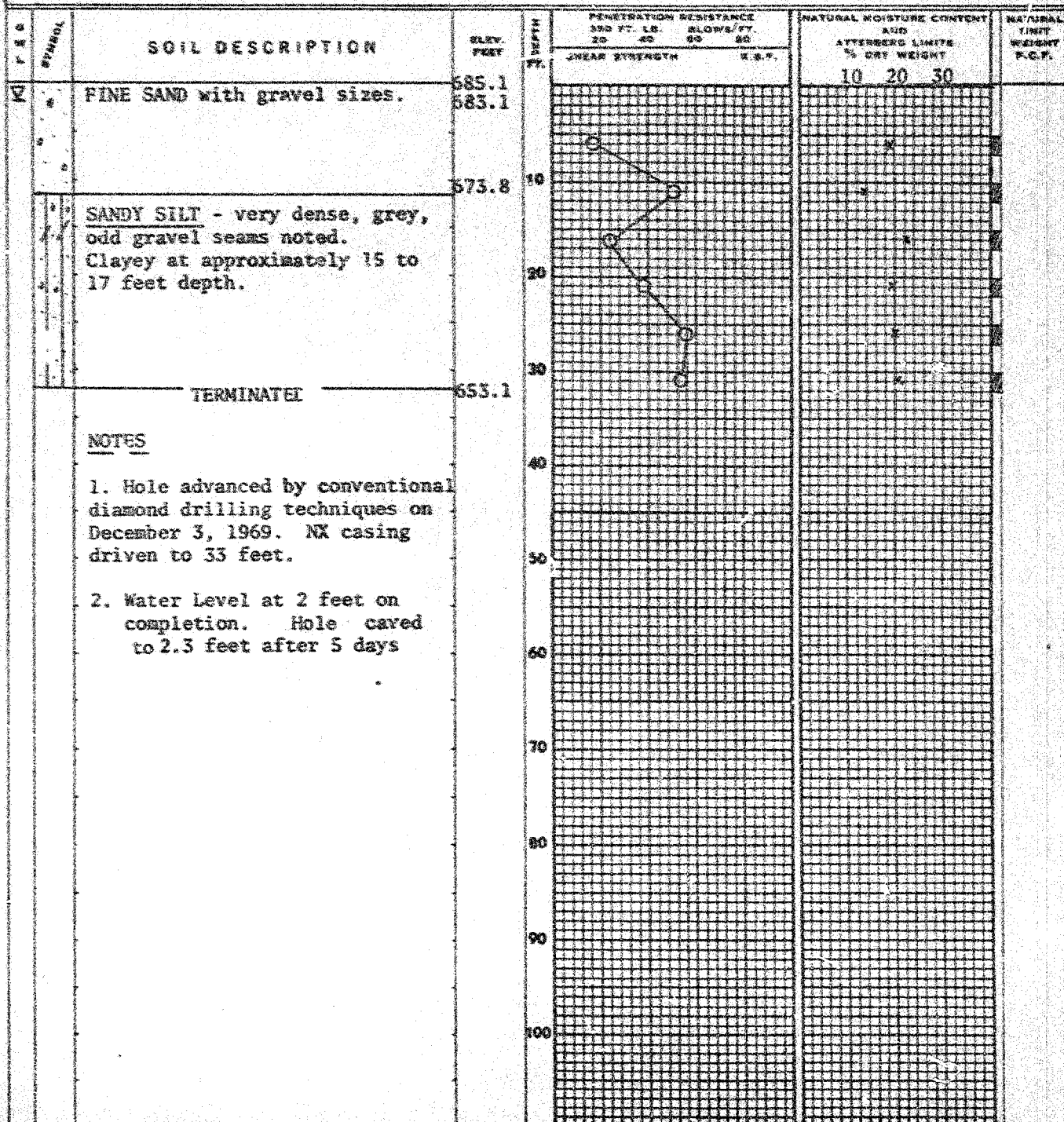
JOB No. H761

BOREHOLE No. 4

DRAWING No. 5

PROJECT Victoria Street BridgeLOCATION Simcoe, Ontario2" 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. H761


BOREHOLE No. 5

DRAWING No. 6


PROJECT Victoria Street Bridge

LOCATION Simcoe, Ontario


2" O.D. SPLIT TUBE 


2" I.D. SHELBY TUBE 

2" DIA. CONE 

PUSHED 

VANE TEST AND SENSITIVITY (S) 

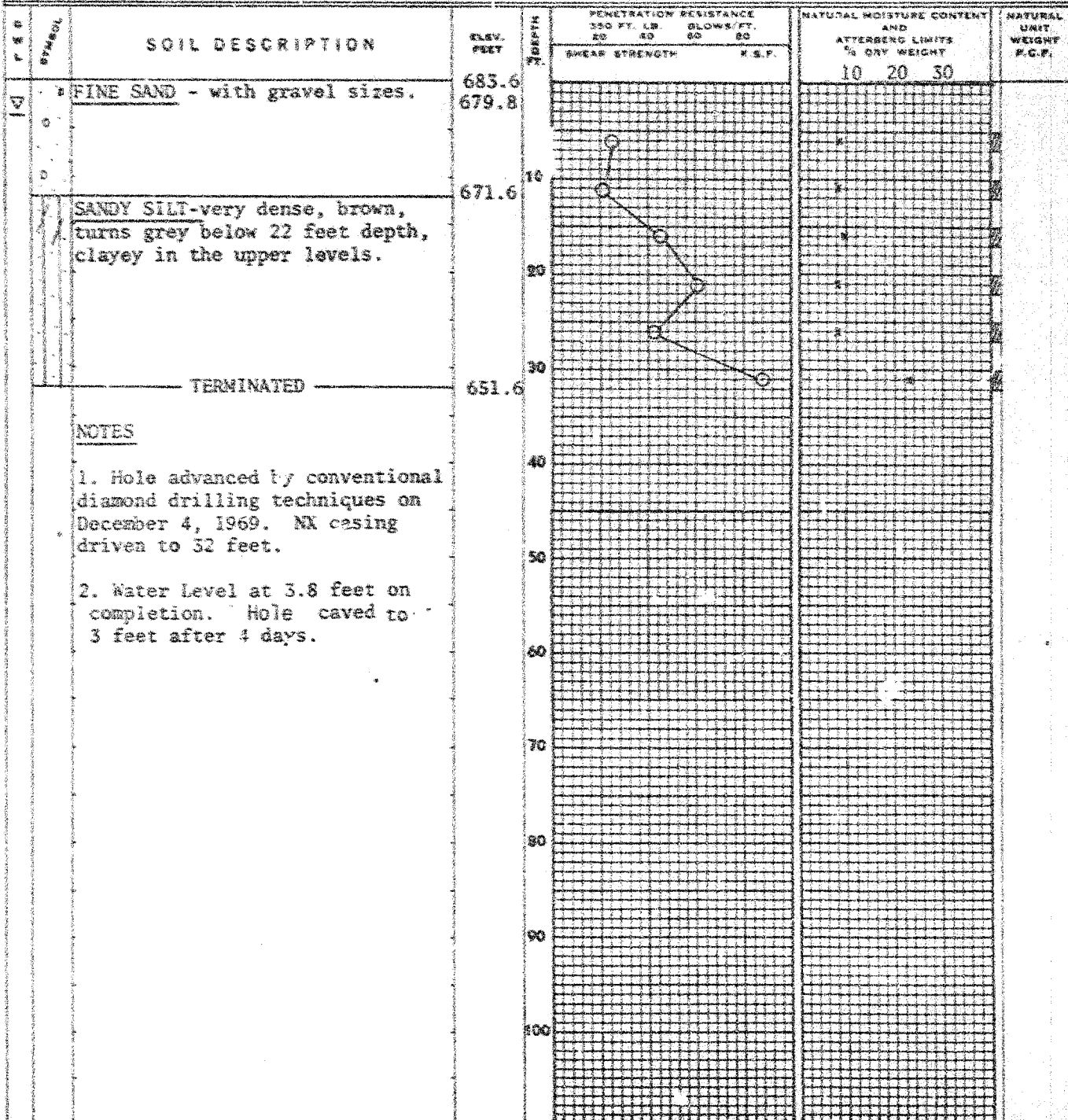
NATURAL MOISTURE 

PLASTIC AND LIQUID LIMIT 


UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE 

STRAIN AT FAILURE 

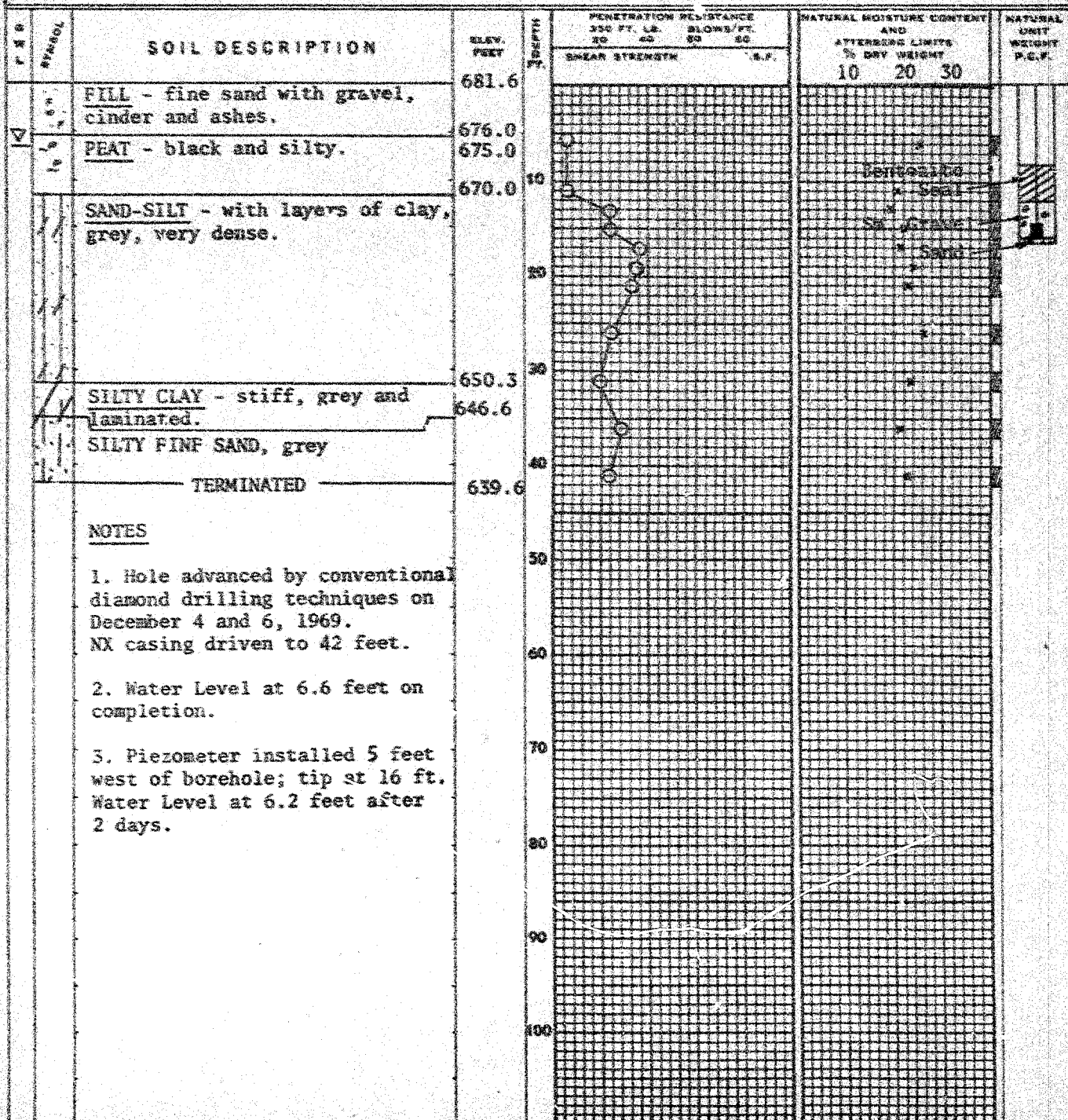
HOLE LOCATION AND DATUM SEE DRAWING NO. 1



BOREHOLE LOG

JOB No. H761BOREHOLE No. 6DRAWING No. 7PROJECT Victoria Street BridgeLOCATION Simcoe, Ontario2" 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 

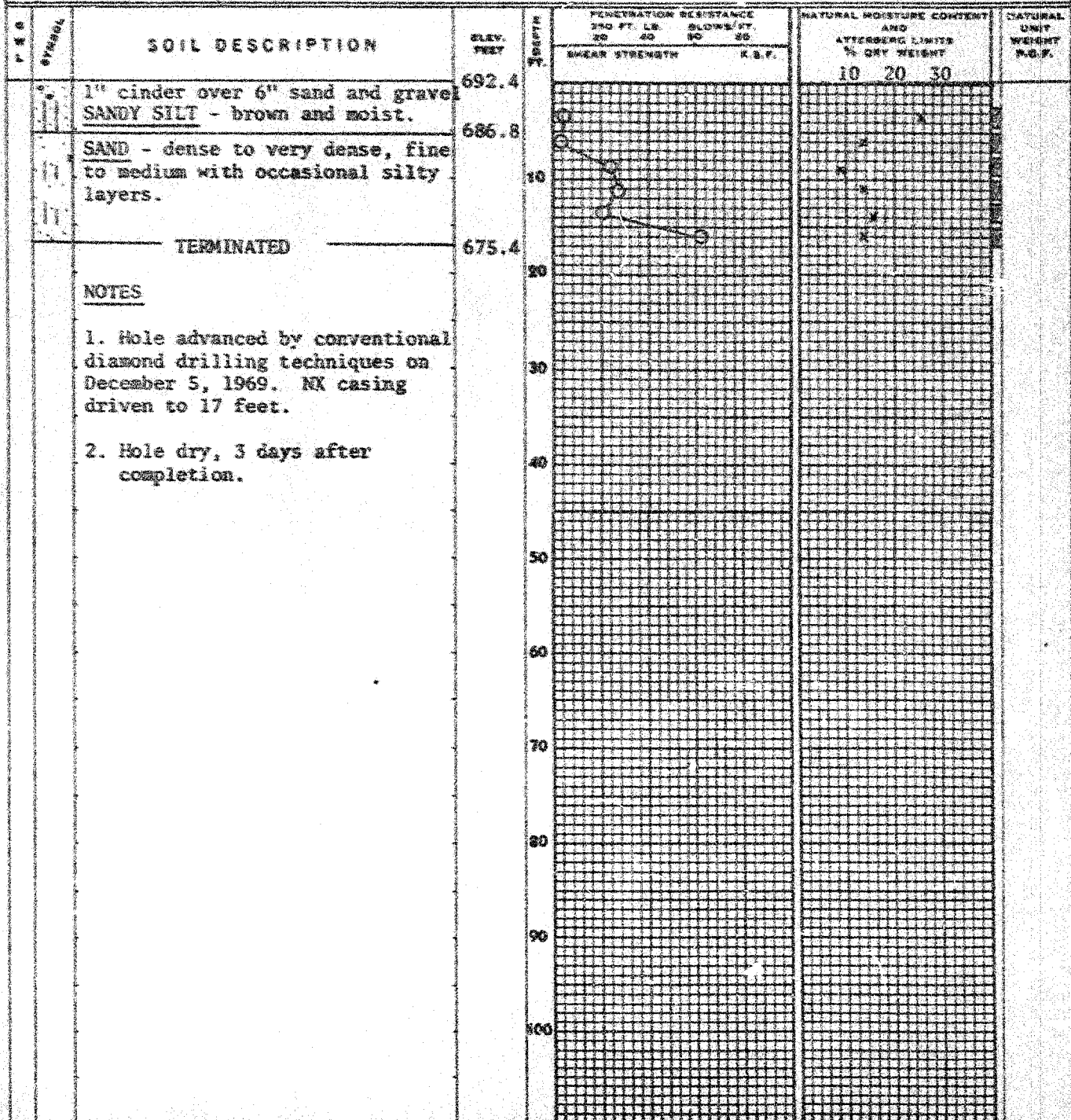
HOLE LOCATION AND DATUM SEE DRAWING NO. 1



BOREHOLE LOG

JOB No. H761BOREHOLE No. 7DRAWING No. 8PROJECT Victoria Street BridgeLOCATION Simcoe, Ontario2" 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. H761

BOREHOLE No. 8

DRAWING No. 9

PROJECT Victoria Street BridgeLOCATION Simcoe, Ontario2" O.D. SPLIT TUBE 2" I.P. SHELBY TUBE 2" DIA. CORE 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

P.C.	SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	PENETRATION RESISTANCE 200 FT. LB. BLOWS/FT. 20 40 60		NATURAL MOISTURE CONTENT AND ATTENBERG LIMITS % DRY WEIGHT			NATURAL UNIT WEIGHT P.C.P.
					SHEAR STRENGTH C.S.F.		10	20	30	
		FILL - mixture of silt sand, and gravel with cinders, wood pieces, brick fragments	692.9							
			683.9	10						
			681.3							
		PEAT to 13 feet depth then changing to sandy silt.								
		TERMINATED	675.9	20						
		NOTES								
		1. Hole advanced by conventional diamond drilling techniques on December 5, 1969. NX casing driven to 17 feet.								
		2. Water Level at 7 feet on completion. Water Level at 9 feet after 3 days.								






BOREHOLE LOG





JCS No. H761

BOREHOLE No. 9

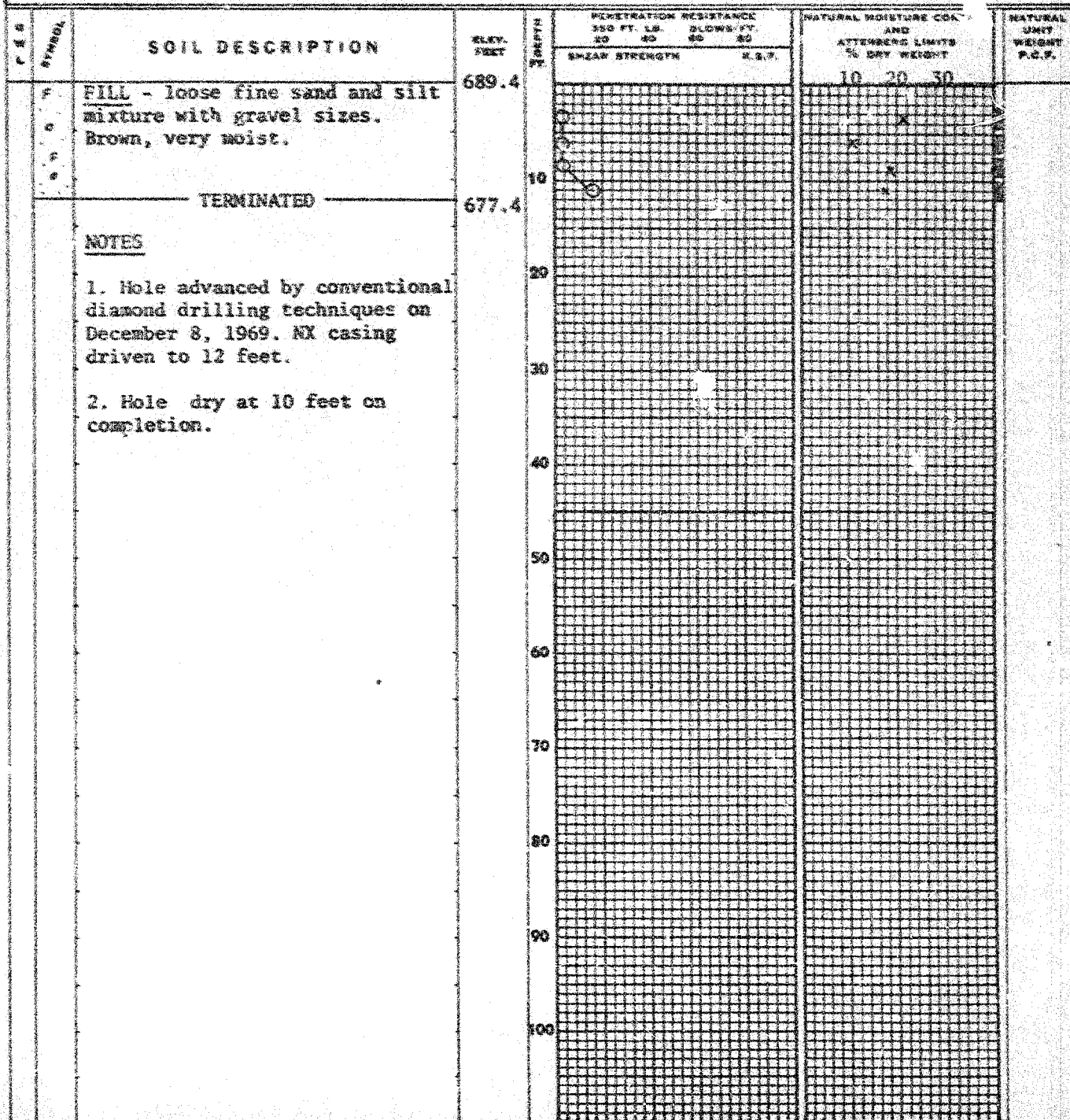
DRAWING No. 10

PROJECT Victoria Street Bridge
LOCATION Simcoe, Ontario


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 

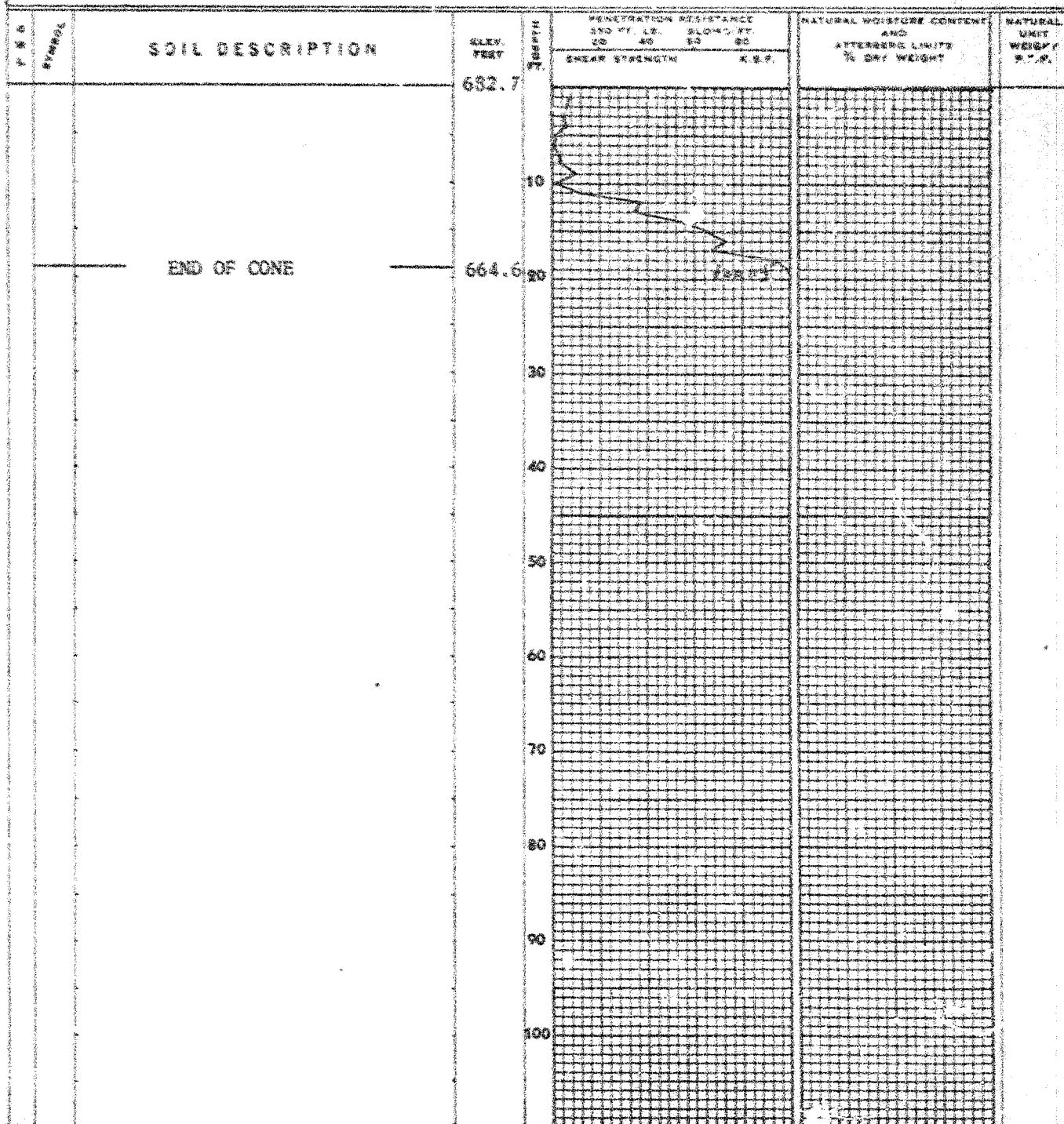
HOLE LOCATION AND DATUM SEE DRAWING No. 1







BOREHOLE LOG

JOB No. H761CONE ADRAWING No. 11PROJECT Victoria Street BridgeLOCATION Simcoe, Ontario2" O.D. SPLIT TUBE 2" I.D. SHELBY TUBE 2" DIA. CONE FURNACE VAPE TEST AND SENSITIVITY (SI) NATURAL MOISTURE PLASTIC AND LIQUID LIMIT UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE % STRAIN AT FAILURE 

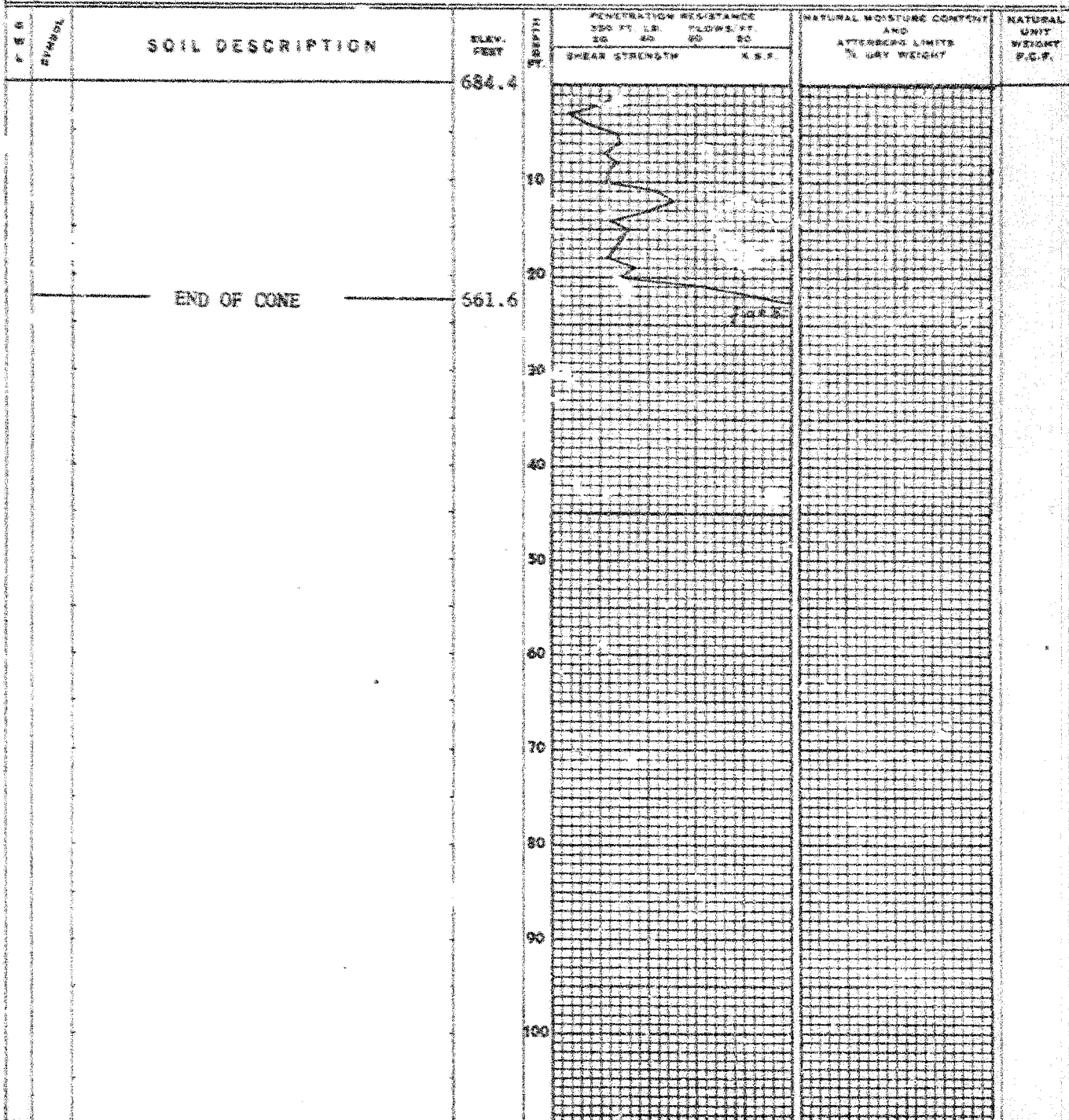
HOLE LOCATION AND DATUM SEE DRAWING NO. 1

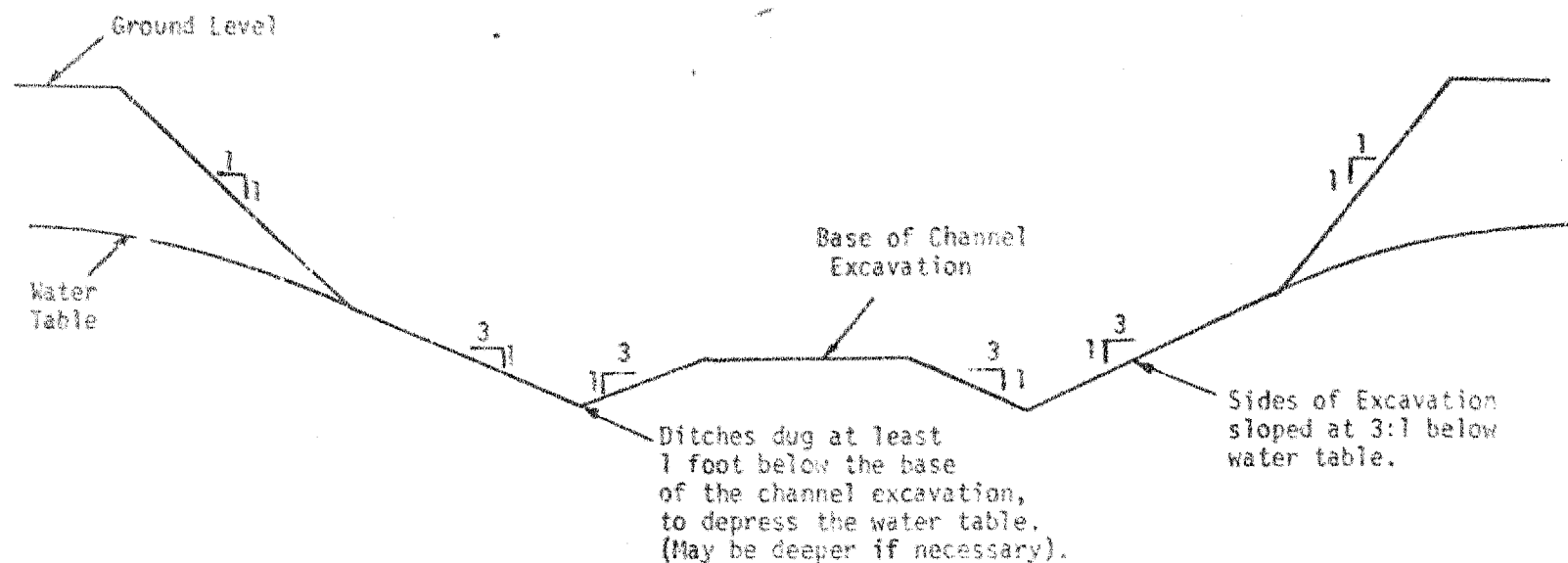


BOREHOLE LOG

JOB No. H761CONE BDRAWING No. 12PROJECT Victoria Street BridgeLOCATION Simcoe, Ontario2" 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





SKETCH OF RECOMMENDED DRAINAGE PROVISIONS FOR LYNN RIVER CHANNEL EXCAVATION
BELOW THE WATER TABLE