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W. O. No. 92-11002

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: \_\_\_\_\_

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REPORT ON

SUBSURFACE INVESTIGATION

FOR

SOUTHEAST TRANSITWAY - CN BEACHBURG

BANK STREET

TO PROPOSED

CNR GRADE SEPARATION

OTTAWA, ONTARIO

TO

MCCORMICK RANKIN

AND THE

REGIONAL MUNICIPALITY OF OTTAWA-CARLETON

REPORT NO. SF-3062  
August 29, 1990

**McROSTIE GENEST ST-LOUIS**

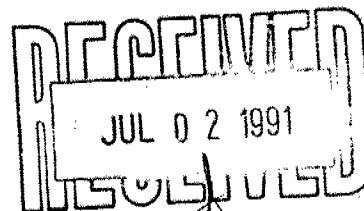
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June 27, 1991



McCormick & Rankin  
Associates Ltd.  
1145 Hunt Club Road  
OTTAWA, Ontario  
K1V 0Y3

Attention: Mr. A. Wing, P.Eng.

RE: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

Further to our recent telephone discussions and review of sketches related to temporary shoring for the above project, we would like to record the recommendations that we have discussed.

The earth pressure diagram on Plate No. 52 of our report SF-3062 is purposely conservative because of the restriction of limiting the settlements of the existing railroad to negligible amounts.

In order to increase the deadman anchor capacity by a significant amount, we suggest that they be deepened by about one (1) metre. At this depth they should be set back at least fifteen (15) metres behind the face of the wall.

The following soil parameters can be considered applicable for the clays encountered at the site.

- soil density 16.5 kN/m<sup>3</sup>
- passive earth pressure coefficient 3.40

There will be a few anchors in compacted Granular 'B' backfill and these can be expected to provide at least as much resistance as those placed in clay soils. We still recommend that a reduction factor of at least 2.0 be applied to passive earth pressures.

A sloped cut (1:1) will need to be made from the top of the anchor block for stability reasons. A similar cut will be needed on the exposed side of the anchor block.

Although there are some edge effects that would increase the available passive resistance, it is normal and conventional practice to design the anchors on the basis of the block area only.

Should any questions arise from the above, we would be happy to discuss the details with you.

Yours very truly,



M.W. St-Louis, P.Eng.  
McRostie Genest St-Louis  
& Associates Ltd.

MWS/nd

## McROSTIE GENEST ST-LOUIS

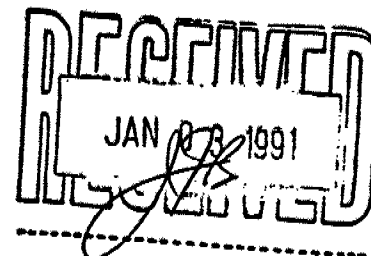
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December 31, 1990

Our Reference: SF-3062C

McCormick Rankin Associates Ltd.  
1540 Raven Avenue  
OTTAWA, Ontario  
K1Z 7Y9



Attention: Mr. A. Wing, P.Eng.

RE: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

Further to your recent request, we have carried out stability analysis of the wall/slope/backfill system between Stations 10+011 and 10+040 at the above site.

Boreholes made by ourselves and others at the site, indicate that the section presently under study is affected by significant quantities of unselected fill.

The lowering of the foundations for the retaining wall onto natural undisturbed soils would assure proper bearing and because of the depths involved would also assure adequate margins of safety for overall stability.

An alternative would be to subexcavate and replace the unselected fill on-site with compacted selected granular fill such as MTO - Granular 'B' (type II). The footings could then be made to bear on the selected fill layer at a depth where suitable frost protection is provided.

In order to provide lateral resistance, the backfill adjacent to and below the toe of the retaining wall should be extended at least 0.5 m from the edge of the footing and then sloped 1 on 1 (45 degrees) or flatter. On the side of the heel, the unselected fill can be removed to the natural soil and the backfilling made against an undisturbed face. The same bearing pressure can be used for both the natural soils and man-made backfill.

Should any questions arise from the above, or should you require further information, we would be happy to discuss the details with you.

Yours very truly,



A handwritten signature in cursive script, appearing to read 'M.W. St-Louis'.

M.W. St-Louis, P.Eng.  
McRostie Genest St-Louis  
& Associates Ltd.

MWS/nd

**McROSTIE GENEST ST-LOUIS**

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TEL: (613) 228-7088  
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DATE: November 19, 1990

TO: Mr. P. Wing OF: McBarnick Rankin

FAX NO. 728-6241 SENDER: M. W. St Louis

PROJECT S.E. Transitway PROJECT NO. \_\_\_\_\_

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November 19, 1990

Our Reference: SF-3062B

McCormick Rankin Associates Ltd.  
1540 Raven Avenue  
OTTAWA, Ontario  
K1Z 7Y9

Attention: Mr. A. Wing, P.Eng.

Re: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

Further to our recent meeting at your office, we would like to confirm certain recommendations pertaining to the above project, in view of certain site-specific conditions and restrictions.

Between Stations 10 + 300 and 10 + 334, where the height of fill to be retained by the north retaining wall reduces from about 2.1m to 0.2m, the overall stability of the wall/slope system can be assured by providing adequate frost cover only. The underside of footing at the extreme end of the toe for the retaining wall should be established from a line drawn perpendicular to the slope in front of the wall. In other areas, the general criterion given in our report SF-3062 should be followed to assure the overall stability of the wall/slope system.

The lateral stability of the retaining walls at the location of proposed utility pipes needs to be assured. The edge of trench needs to be at a minimum



distance such that it falls beyond an imaginary line drawn at two (2) horizontal to one (1) vertical units from the underside of footing as shown on the accompanying plate.

Between Stations 10 + 100 and 10 + 220, the use of a "Jersey" barrier wall is being considered at the toe of the slope south of the transitway route. The small wall could possibly be designed as a large curb without frost protection. In this case, the designers must be prepared to accept frost related movements. Otherwise, a suitably designed insulation scheme equivalent to a soil cover of 1800mm would be required.

Relatively steep backslopes are being considered behind these retaining walls (ie. 1.4:1 and 1.7:1) on the south side of the transitway route and special recommendations are needed to maintain the margins of safety of sloping ground within acceptable limits.

We understand that the entire wall will be backfilled with Granular 'B' (type II) material connected to an underdrainage system. Between Stations 10 + 100 and 10 + 220, partial removal and replacement of the clay soil will be required.

As a clay slope gets steeper, such as the inclinations being considered above, surface sloughing due to freezing/thawing and wetting/drying become more important. Furthermore, the effect of train loads on the steeper slopes has a greater impact on reducing the factors of safety.

Our analysis shows that by replacing the clay to at least 0.5m behind the retaining wall and into the slope cut at 1:1, the overall stability of the wall/slope/backfill system will have adequate margins of safety. Of course, this backfill wedge will need to be connected to an underdrainage layer, in order to reduce groundwater pressures within the slope.

Our analysis included a parametric study of both drained and undrained conditions with train loads present on a track with centreline at 1600mm from crest of slope. Some assumptions were necessary in order to analyse a 3-D load from the train wheel assembly using conventional 2-D computer techniques.

The steep backslope behind the retaining walls will result in a significant increase in earth pressure. The equivalent coefficients of active earth pressure recommended for this project are based on both theoretical and semiempirical methods for estimating the pressure of drained backfill on low retaining walls.

#### ACTIVE PRESSURE COEFFICIENT (Ka)

##### Granular 'B' (type II)

Backslope	SLS-11	ULS
1.4:1	0.67	0.81
1.7:1	0.45	0.76

The earth pressure distribution can be assumed to be triangular with the direction of resultant parallel to the inclination of the backslope.

The coefficient of friction under the cast-in-place retaining wall resting on a layer of Granular 'B' (type II) backfill can be taken as follows:-

## COEFFICIENT OF FRICTION

Granular 'B' (type II)

SLS II	0.70
UIS	0.56

The use of an artificial erosion protection layer should be considered in the steep backslope areas, specially if plans call for a surficial layer of topsoil.

Special measures may be required to resist the large earth pressures. If the sliding resistance is found to be inadequate, the lateral forces acting on the wall could be reduced by the use of light weight fill. This light weight fill is apparently available from Canadian sources in the Hamilton area. We understand that the material has a unit weight of about  $11 \text{ kN/m}^3$  and is now being estimated at \$40.00/tonne on an Ottawa area MTO project.

Should any questions arise from the above, or should you require further information, we would be happy to discuss the details with you.

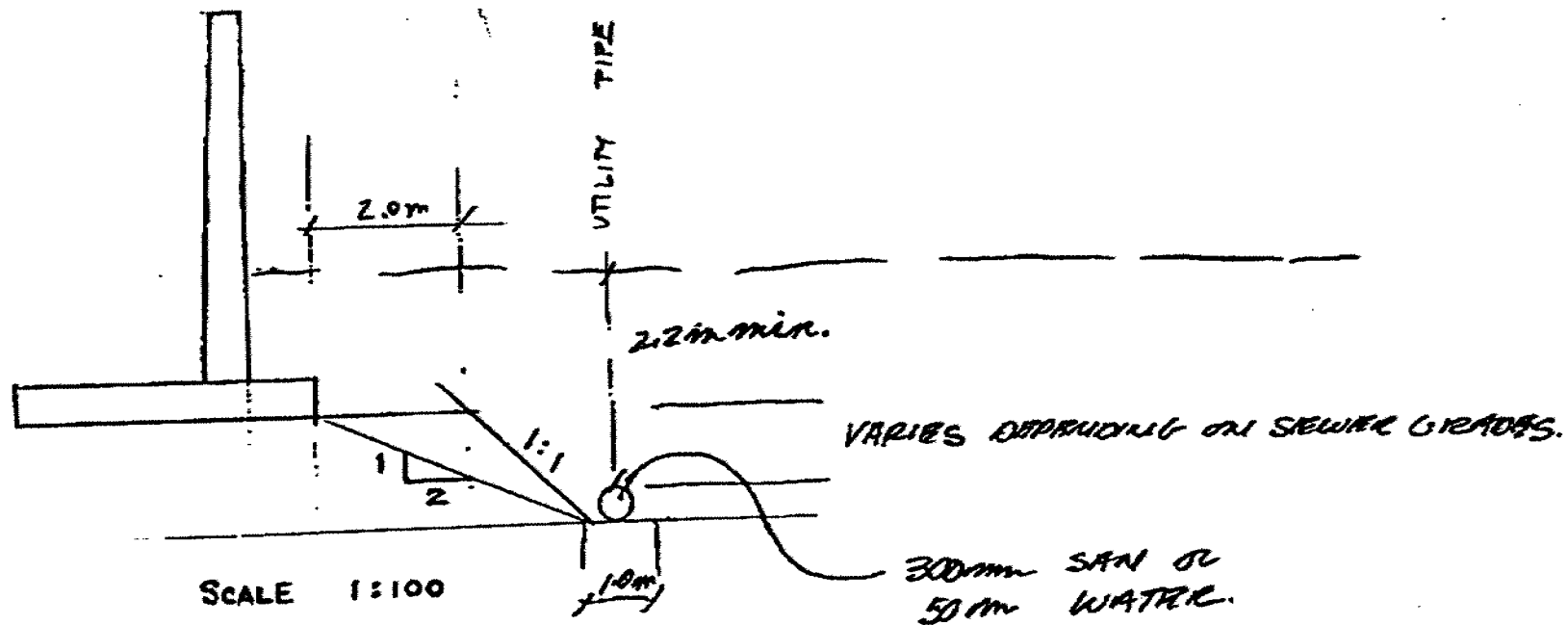


MWS/gl

Yours very truly,

A handwritten signature in cursive script, appearing to read "Michel Allard".

M.W. St-Louis, P.Eng.  
McRostie Genest St-Louis  
& Associates Ltd.



**McROSTIE GENEST ST-LOUIS  
& ASSOCIATES LTD.  
CONSULTING ENGINEERS**

Ste 201-1755 Woodward Dr., Ottawa, Ont. K2C 0P9

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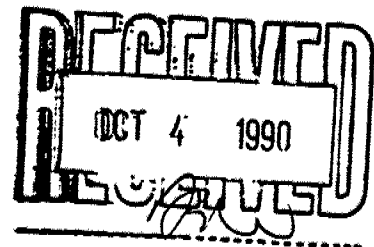
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DATE: October 04/1990TO: Mr. A. Wing OF: McBormick RankinFAX NO. 728 6241 SENDER: MW St LouisPROJECT SE Transitway PROJECT NO. SE 3062ANUMBER OF PAGES INCLUDING THIS TRANSMITTAL 5ORIGINAL FORWARDED: MAIL C COURIER    HAND    NO   

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TEL: (613) 228-7088  
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October 04, 1990  
Our reference SF-3062A

McCormick Rankin  
Associates Ltd.  
1540 Raven Avenue  
OTTAWA, Ontario  
K1Z 7Y9

Attention: Mr. A. Wing, P. Eng.

RE: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

Further to our recent meeting at your office, we would like to confirm certain recommendations pertaining to the above project.

The project will include several shelters (12 X 3m) on a concrete platform 6m wide. These platforms can be constructed as floating slabs but will need to be designed to accommodate differential settlements.

Differential settlements are somewhat proportional to the amount of backfill above the compressible clay layer. Our best estimate of the time required for these consolidation settlements to occur would be 25% after 3 months, 50% after 1 year with near completion after 5 years. Estimates of the magnitude of settlement where the backfill will be 4.5m to 5m above the present ground surface are given in section 3.11 of our report SF-3062.

Several parameters affect the amount of settlement that will occur under fill weight, including the thickness and density of fill, the thickness of the compressible clay as well as variations in soil strength and compressibility characteristics. Although, theoretically incorrect, a linear relationship from zero settlement at zero fill to the maximum values given above would be reasonable in judging the probable amounts of future settlements and their effect on surface drainage.

As discussed, some consideration could be given to localized excavations in clay, in order to get a more uniform thickness of granular fill and thus more uniform settlement.

There are shelters and platforms in a clay cut section that would result in exposing these underlying soils to a first cycle of freezing and thawing. Significant differential heaving would occur and special treatment would be necessary. We would recommend that subexcavation and replacement of the clay with non frost acting free drainage granular soils be made at these locations. A thickness of 1800mm is recommended.

In other locations, where the grade will not be changed significantly or will remain unchanged, some protection against frost heave will also be necessary even though the clays have been subjected to freeze-thaw cycles. These clays are also frost acting although they tend to heave somewhat less than deeper higher water content clays never exposed to these conditions.

Generally, frost protection can be accommodated by replacement or by means of an equivalent insulation scheme. The frost line can be assumed to be at 1800mm from grade throughout the project.

Of course, the selected granular fill placed below shelters and platforms should be drained to a suitable outlet.

Shelters 6 and 7 and others with geometries typical of these, are in a condition where no change of grade will occur at the front of the platform but a wedge of soil with a maximum height between 1.0 and 1.5m will be placed behind the platform. These soil wedges are not expected to induce more than 5 to 10mm of settlement behind the platforms.

Based on soil parameters obtained in the boreholes made at the site, including the statistical analysis of the variations in strength between boreholes, we recommend the following for adhesion under footings subjected to lateral loading conditions.

<u>ULS</u>	<u>SLS II</u>	
50kPa	100kPa	above el. 70m.
25kPa	50kPa	above el. 70m.

The above are generalized parameters. Should the stability become critical using these values, further refinements would become necessary including review of closest borehole information.

We were requested to review the feasibility of anchoring the soldier pile and lagging wall system by means of a deadman block approach.

We feel that this temporary retaining system would be possible at the site. The resisting block could be attached at a depth of about 2500mm but should be set back at least 15m behind the face of wall. The H-piles should also be driven at least 5m below the bottom of excavation and would not be required to reach the till layer as no vertical downward force component results from this type of bracing system.



The deadman anchor block and the portion of pile below the base of the excavation will be under conditions of passive restraint. We recommend that the following passive earth pressure coefficients be used.

	<u>ULS</u>	<u>SLSII</u>
Kp	2.70	3.40

In accordance with the CFEM, the piles below the excavation can provide passive resistance over a distance equivalent to three (3) times the pile width.

Again in accordance with the CFEM, passive pressures should be computed using a coefficient of passive pressure reduced to account for movements required to develop these. We would recommend that a reduction factor of at least 2.0 and preferably 2.5 be applied to the Kp factors in order to reduce excessive movements in SLSII. Passive earth pressures considered as a resistance should be factored in accordance with section 6 of the OHBDC under ULS conditions.

Should any questions arise from the above, or should you require further information, we would be happy to discuss the details with you.

Yours very truly,



*Michael Allard*

M.W. St-Louis, P.Eng.  
McRostie Genest St-Louis  
& Associates Ltd.

MWS/evv

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# McROSTIE GENEST ST-LOUIS

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## 1. TERMS OF REFERENCE

We were requested and authorized by McCormick Rankin Associates Limited on behalf of RMOC (Regional Municipality of Ottawa-Carleton), to carry out a subsurface investigation for the Southeast Transitway between Bank Street and the proposed CNR grade separation, in Ottawa, Ontario.

The study was to be made with new boreholes, auger holes and test pits. Pertinent subsurface information from previous studies was also to be included in this report.

Special tests were also carried out at the proposed Billings Bridge Station, using a Norwegian corrosion sounder probe.

## 2. GENERAL GEOLOGY

The surficial topography at the site consists of natural sloping land to the southeast resulting from the entrenched Sawmill Creek. The land to the northwest has been artificially reworked and sloped during the last few decades.

The area is underlain by Champlain Sea deposits consisting of clays, silts and silty clays. These deep water clay sediments are underlain by glacial deposits resting on bedrock. The glacial tills consist of a heterogeneous mixture of soil material ranging from clay to large boulders. The underlying bedrock is geologically mapped as black shale of the Billings Formation.

In general, bedrock is closer to the ground surface in the northeastern portion of the site and gradually gets deeper in a southwesterly direction.

### 3. CONCLUSIONS AND RECOMMENDATIONS

#### 3.1. Foundation Type

The most suitable and economical types of foundations at the Billings Bridge Transitway Station and approaches would be soil supported footings and/or reinforced earth walls for the more flexible approaches to the station and the ancillary structures. The more rigid glass covered structures should be supported on piles.

#### 3.2. Soil Bearing Pressures

Foundations for local shelters or platforms, other small ancillary structures and the retaining wall would need to be lowered through the fill layers and onto undisturbed natural clay soils as encountered in the boreholes and test pits.

For design purposes, the allowable bearing pressures that we recommend would be 150 kPa under SLS II conditions and 300 kPa under ULS conditions. These values are based on a statistical analysis of the variations in strength that were found in widely spaced boreholes.

The central portion of the pedestrian tunnel link could also be soil supported. At this location and at the depths being considered, the clay is somewhat softer but the load applied below the base of the open box structure under the relocated tracks is about equal or slightly less than the present overburden pressure.

## 3.2. cont'd

The relatively deep excavation required to construct the tunnel will result in an elastic response of heaving of the bottom of the excavation due to stress relief. Based on a comparison of site specific soil properties with those from another site where heave gauges were actually installed, we have evaluated the amount of rebound and recompression due to the excavation followed by backfilling.

We expect that the recompression of the clay below the future tunnel box structure will be almost equal to the amount of rebound because the state of stress will have changed only slightly. We estimate that about 30mm of recompression will occur under the tunnel base after the backfilling is completed. The settlement response is elastic or immediate in nature and is expected to occur over a period of a few hours to a few days.

No consolidation settlements are expected because the state of stress below the future tunnel base will not exceed present conditions. Furthermore, transient loading from the train above will not induce any consolidation settlements in the underlying clay.

The base slab of the proposed tunnel will need to be designed using an elastic model. Based on the geometry of the proposed structure, the shear strength of the underlying clay and the expected amount of soil recompression, we have evaluated the modulus of subgrade reaction (ks). We suggest that a value of 4000kPa/m be used for design purposes.

### 3.2. cont'd

A special structural feature may be required where the soil supported structure joins the pile supported structure.

### 3.3. Pile Types and Capacities

The most suitable type of pile to support any rigid structure on this project, in our opinion, would be thick wall pipe piles driven closed end and filled with concrete after driving. The closed end pipe can be inspected after driving to ensure that it has not been damaged by obstructions or otherwise, thus allowing a greater degree of confidence in the piles. An end plate about 50mm thick should be welded on the pipes piles in order to reduce the risk of damage.

H-piles fitted with a driving shoe can also be considered by the designers but these should be expected to penetrate deeper into the underlying fractured shale bedrock and they cannot be inspected after driving.

We have reviewed design capacities for different pile types driven to bedrock. These values given below are for both SLS II and ULS capacities with an allowance for corrosion which explains their reduction from conventional capacities. The ULS capacity is governed by the structural strength of the pile rather than the ground resistance, which is very high in the bedrock.

## 3.3. cont'd

<u>PIPE PILE SIZE</u>	<u>PILE CAPACITY (kN)</u>	
	<u>SLS II</u>	<u>ULS</u>
244mm dia X 13mm wall	740	960
273mm dia X 13mm wall	830	1,130
324mm dia X 14mm wall	1,100	1,575
356mm dia X 15mm wall	1,275	2,175
<u>H-Pile Size</u>		
310 HP 79	390	550
310 HP 110	700	1,000

We have examined typical energy requirements and the resulting driving criteria necessary to develop the capacity of the pile types given above. The results are as follows: -

<u>PILE SIZE</u>	<u>BLOWS/LAST 12mm</u>	<u>ENERGY(kJ)</u>
244mm dia X 13mm wall	4	40
273mm dia X 13mm wall	8	40
324mm dia X 14mm wall	9	50
356mm dia X 15mm wall	8	60
310 HP 79	3	30
310 HP 110	4	30

The pile capacities that we have recommended are purposely conservative and can be achieved with the above. These are based on simplified assumptions such as hammer efficiency and driving block and may need to be modified to meet actual site conditions if found to be unusually different. Where the designer chooses to specify the driving criteria, the Owner then becomes responsible for achieving the required pile capacity and in this case the criterion needs to be conservative.



## 3.3. cont'd

We feel that it is in the best interest of the Owner to specify typical pile types and capacities and be prepared to accept alternatives proposed by The Piling Contractor. The pile capacities can then be maximized within accepted standards resulting in minimizing the cost of piling.

There are boulders in the glacial till underlying the thick clay deposit. Some piles may be damaged when driving through these obstructions and some piles will need to be replaced. There needs to be some provision in the contract documents to accomodate the replacement of damaged piles.

We suggest that the Owner will likely pay the least total dollars for a driven pile foundation if he accepts the cost of reject piles as a risk, rather than having a piling contractor include this cost as an uncertainty in his bid. A contract which is based on unit length prices for the piles plus payment for rejected piles should result in a lower bid price, although some of this benefit is reinvested as payment for additional piles and varying lengths.

On similar projects, the percentage of rejection of piles which are damaged or bent in attempting to penetrate to an adequate depth has been about five (5) percent.

### 3.4. Pile Lengths

It is difficult to be certain regarding the elevation at which piles would be driven to achieve design capacities. In some instances, piles may even penetrate the underlying shale rock, that required diamond core drilling in the boreholes, to depths of a few metres.

For the above reason, we would suggest that a piling contract contain certain provisions for adjustment of compensation, depending on the actual lengths found necessary. For contract estimating purposes, however, an assumption regarding length needs to be made and hence, we are suggesting that the pile lengths be estimated using the average bottom of pile as the elevation corresponding to about two (2) metres below the bedrock surface found in the boreholes.

Also, the adjustment of cost for the actual pile lengths driven should be based on the average of all piles within the contract or on the total length of piles for the contract and not on an individual pile basis.

### 3.5. Relaxation of Pile Support

In the Ottawa area, a phenomenon of relaxation at the pile tip has occurred on some construction sites underlain by shale rock and/or glacial till. Changes in porewater pressure with time, in the rock or soil prism, below the pile tip, are believed to be the cause of a reduction in the carrying capacity of the pile, only a few days after driving to adequate refusal. Restriking of piles is usually made on sites where relaxation could be expected, in order to verify its possible occurrence and to assure adequate resistance to penetration.

## 3.5. cont'd

We therefore, recommend that the costs of a pile restriking program be included in the piling contract. This could be based on a price per pile for a specified number of returns to its location and restriking, since 2 or 3 restriking returns have been found necessary on some recent projects. It is however, also possible that no relaxation will occur. We can suggest that a unit price be obtained for at least one restrike for each pile in the contract bids.

3.6. Lateral Resistance of Piles

The soil-structure response of laterally loaded piles is complex. The following recommendations are based on both widely accepted empirical methods and results from actual horizontal load tests performed by us in the Ottawa Area by jacking two (2) adjacent piles apart. Our examples of pile behavior under horizontal loading conditions were spread across a wide range of pile sizes and soil conditions.

Vertical and steep batter piles can provide lateral resistance by passive earth pressure adjacent to the pile. As the batter of a pile increases, the lateral resistance is provided only by the structural horizontal force component.

In accordance with the Canadian Foundation Engineering Manual (CFEM), the passive soil resistance should not be accounted for in the design of piles battered at more than eight (8) vertical units to one (1) horizontal unit.

## 3.6. cont'd

From our review of past records, and considering the subsurface conditions at the site, we judge that under ULS conditions, conservative values of passive soil resistance must be used for the range of pile sizes that could potentially be used. These values were calculated using OHBDC section C6.8.3.8. and are listed as follows: -

<u>PILE SIZE</u>	<u>LATERAL RESISTANCE (kN)</u>
244mm dia X 13mm wall	15
273mm dia X 13mm wall	20
324mm dia X 14mm wall	35
356mm dia X 15mm wall	45
310 HP 79	30
310 HP 110	30

The above values should only be used for vertical and steep piles within the limits of batter recommended in the CFEM.

Previous test results have shown that pipe piles of similar diameter to the larger ones listed above could tolerate about 10mm of movement at the pile top at a load of about 120 kN without damage to the piles.

The conservative values that we have recommended could likely be increased on the basis of future field testing, if the lateral resistance of piles is a limiting and important part in the design.

Battered piles can resist lateral loads quite effectively through the structural force component and we recommend that this alternative be considered. The

## 3.6. cont'd

slope of inclined piles should be limited to four (4) vertical units to one (1) horizontal unit in order to limit installation problems.

3.7. Downdrag Forces on Piles

The Billings Bridge station will be supported on piles. The fill within the station will cause settlements to occur due to the consolidation of the underlying compressible clay layer. This settlement will transfer downdrag forces onto the piles. An allowance for downdrag equivalent to about 20% of the total pile loading in SLS II should be included in sizing the design piles. In areas where there is no proposed grade change, no account for downdrag is needed. The above recommendation is based on past experience and practice but recent and current research suggests that a different approach to account for downdrag will likely be used in the near future.

The possibility of using a bituminous slip layer to reduce the amount of downdrag has been considered but at this time, its use is not obviously more economical than the 20% capacity allowance, but the option can be further studied.

### 3.8. Uplift Capacity of Piles

The allowable uplift on the piles was calculated using fifty (50%) percent of the factored axial skin friction capacity in accordance with the OHBDC. Piles at this site will be driven through clay and till soils. The soil "freeze-up or set-up" against the piles will take a few months in the clay and only a few days in the tills. Based on the subsurface stratigraphy at the site, we would recommend that a value of 5 kPa be used for the average allowable skin friction acting on the pile perimeters.

As indicated in the CFEM, the most reliable way of designing piles subjected to uplift forces is by means of uplift testing in accordance with ASTM-D3689. Otherwise, conservative values such as the one recommended above should be used.

### 3.9. Depth of Frost Penetration

The depth to which frost will penetrate into soils is influenced by several factors including grain size, density and water content. For practical purposes, however, the depth of frost penetration can be adequately estimated for roadway design regardless of subgrade type by using a correlation between thermal characteristics such as the freezing index and actual field observations made under highways and airport runways.

Based on a normal freezing index of about 1,000 degree - days ( $^{\circ}\text{C}$ ) for Ottawa, derived from the average of thirty (30) years of weather records, a depth of frost penetration ranging between 1.8m and 2.0m would be expected under snow cleared areas. We recommend that the value of 1.8m be used for design purposes.

## 3.9. cont'd

Special frost protection will be required for the unheated pedestrian tunnel. Polystyrene foam will need to be placed below the base and up the exterior sides of the box structure in order to avoid frost action in the adjacent clay soils. The insulation scheme and dimensions need to account for a heat loss path 1.8m long. This path needs to recognize that the zero-isotherm (frost) will extend through the concrete wall and granular backfill. The granular backfill behind the insulation near the edge of this insulation should also be expected to freeze. We recommend that a rigid insulation 75mm thick used.

3.10. Widening at former Railway Location

Normal roadway widening construction techniques such as OPSD - 208.01 should be followed on this contract. In compliance with the Ontario Provincial Standards, benching is expected to reduce the effect of differential subgrade reaction. A 1,200mm dimension can be used for the depth of transition point treatment for earth cut to earth fill in accordance with OPSD - 205.01.

3.11. Settlements of Fills

We have carried out three (3) laboratory consolidation tests in order to determine the compressibility characteristics of the clay underlying the site. On the average, these tests show that the clay has been overconsolidated by at least 200 kPa by a combination of factors such as past loading, cementation, etc.

## 3.11. cont'd

The maximum thickness of backfill behind the proposed retaining walls will be between 4.5m and 5m above present ground surface. The amount of settlement induced in the clay layer from the surficial filling is dependent on the amount of fill and the thickness of clay. We expect that the maximum settlements at the wall facing will be of the order of 50mm where the underlying clay is thickest.

In areas where there is a transverse cut and fill section, the portion in cut will have zero settlement and the maximum settlement will occur at the wall. Benching will help to make a smoother transition or settlement taper.

3.12. Pipe Bedding and Backfill

The natural subgrade beneath the proposed transitway consists of marine clay, and service trenches will need to be dug into the latter. We recommend that excavations be carried out to at least 150mm below the planned underside of utilities. The preferred bedding material is Granular 'A' (O.P.S.S. 1010) compacted to at least 95% Standard Proctor Density (S.P.D.). The pipe surround, to at least 300mm over the pipe, can be the same material or an approved sand, again compacted to 95% S.P.D.



## 3.12. cont'd

Under favourable weather conditions, some of the native clays can be used as trench backfill. These clays will however become unworkable if exposed for more than a short period to rain. The clays should be compacted with a sheeps foot roller by backfilling from the top of pipe surround to the underside of the granular road subbase. Lift thicknesses such as 200mm are usually necessary to achieve uniform compaction. The clay backfill should be compacted to 95% S.P.D.

The natural clay soils to be used as backfill should be taken within the surficial weathered crust from the upper few metres in the soil profile and have a natural water content below 35%.

There is a special benefit resulting from the re-use of the excavated clay crust as trench backfill, and therefore the practice can be encouraged. In a clay soil environment, the seasonal frost related movements of the pavement surface are more uniform if trench backfill zones and undisturbed subgrade zones are both

## 3.12 cont'd

clay soil. Unfortunately, it may be unwise to make the reuse of clay backfill compulsory since during the wet seasons of the year, the excavated clay cannot be satisfactorily compacted and its use could cause more future difficulties than the use of free draining, compactible backfill.

Should it be necessary to use granular fill above the pipe surround, frost tapers from the frost line to the underside of pavement structure would be necessary in order to provide a transition to accomodate frost related movements. The frost line can be assumed to be at 1.8m from the finished grade.

3.13. Pavement Structure

An economic balance between present or construction costs and such factors as rideability and maintenance costs must be incorporated in any design of pavement structures. Empirical thicknesses of various elements based on past performance combined with an analysis of the factors given above tend to result in a higher degree of confidence.

Based on pavement structures that have generally been successful on similar subgrades, for similar loads and frequencies, we recommend that the following be considered: -

## 3.13. cont'd

Asphalt Wear Course (HL-3)	40mm
Asphalt Binder Course (HL-8)	100mm
MTO - Granular 'A'	150mm
MTO - Granular 'B' (Type II)	<u>450mm</u>
TOTAL	740mm

In order to reduce the amount of permanent surficial deformation or rutting of the asphalt during the summer months, more stable mixes of the above with modified coarse aggregates should be considered.

This pavement structure has also been recommended by others as part of preliminary studies made for costing purposes.

In areas where the excavation into natural soils will be extended a few metres, the subgrade below the future pavement structure will be exposed to frost for the first time. We therefore recommend that the Granular 'B' thickness in these locations be increased to 600mm.

There are areas with significant quantities of unselected fill at the site. All the unselected fill should be removed and replaced beneath the future transitway. The space between the proposed pavement structure and the natural clay can be replaced with MTO - Granular 'B' (type I) and/or suitable clay backfill taken from the upper few metres in the soil profile and having a natural water content below 35%.

## 3.13. cont'd

In areas where native clay is used to backfill between the natural clay and the pavement structure, we recommend that the Granular 'B' thickness forming part of the pavement structure be increased to 600mm.

The wear course should be substituted to a more stable mix with 100% crushed aggregate, such as DFC (dense friction coarse), as a means of further reducing surficial rutting in the station areas.

3.14. Composite Pavement Design

The design of rigid pavements is a function of subgrade type, drainage, loading and frequency, concrete characteristics, thickness of base course, etc. In areas where differential movements are expected due to a combination of settlements, frost heave and other factors, we would not recommend a rigid pavement structure.

Differential movements would result in cracking of concrete whereas flexible asphalt concrete would be more appropriate under these conditions.

3.15. Groundwater

Because of the natural subsurface stratigraphy at the site, groundwater is not expected to result in significant construction problems.

Groundwater levels, however, should be expected to be closer to the ground surface during the wetter seasons of the year. Normal construction groundwater pumping may be required at some locations.

### 3.16. Groundwater Chemistry

The possibility of groundwater containing chemicals in sufficient concentrations to attack buried concrete and steel at unusual rates was examined by conventional methods. Groundwater samples for this purpose were taken from several boreholes.

The concentrations of soluble sulphates, which determine the potential attack on buried concrete indicate that no special problem is likely to exist, therefore, no special cement requirement is recommended for any buried concrete. The concentrations of chlorides in groundwater samples which are a rough guide to the strength of potential attack on buried steel objects tended to show that no special problem was likely to exist but more sophisticated testing has shown otherwise, as reported in a following paragraph of this report.

The above reported chemical results should not be considered as an "Environmental Audit". Concerns such as potential toxicity, biological contamination, chemical composition, presence of oils, greases, phenols, DDT's or PCB's, radio-active materials including Radon gas, are beyond the scope of this study.

Our securing of some soil samples from test pits for EPA - type environmental testing of contaminants should not be considered as an "Environmental Audit".

### 3.17. Soil Corrosivity

The Norwegian Geotechnical Institute (NGI) corrosion sounder was used adjacent to two (2) boreholes at the proposed Transitway Station in order to evaluate the soil corrosiveness with special reference to steel piles. The sounding device consists of a steel tube with a magnesium point acting as an anode. The probe is pushed into the ground and both the resistivity and the galvanic current between the point and the tube are measured with a Wheatstone bridge and a sensitive ammeter. Both sets of readings were taken within the clay deposit at one (1) metre depth intervals from the surface and are recorded on accompanying plates.

Based on the results obtained at the two (2) locations tested, the classification of the clay at the site falls into Group 3 which is "medium corrosivity". This classification system is based on Norwegian experience with a calibration made by comparing probe results to the condition of old steel piles that were pulled out for measurement of actual corrosion losses.

During the present investigation, the maximum readings that were recorded correspond to a corrosion velocity of about 0.02mm per year. A previous study had shown a maximum corrosion velocity of about 0.05mm per year, at the nearby proposed bridge structure around chainage 10+600. We therefore recommend that a three (3) mm layer of sacrificial steel be provided in pile selection to account for losses due to corrosion. This is based on a design life of sixty (60) years and the higher values of corrosion rates that were measured at the adjacent site. The above recommendation is purposely conservative but is made

## 3.17. cont'd

after serious consideration of several aspects including possible variations and reductions in construction costs.

In the case of H-piles, all faces are exposed to soil and therefore, peripheral sacrificial steel needs to be considered. In the case of concrete filled pipe piles, only the outer face of the pile will be exposed to corrosion.

We have also considered alternatives such as passive or active cathodic protection and/or epoxy coatings and sleeves but these corrosion prevention techniques are expensive and may not be considered practical at this site. We recommend that the piles be driven with sacrificial steel included in addition to the steel required in the structural design.

3.18. Roadway Subdrainage

The performance of a roadway can be seriously affected by groundwater and the saturation of the pavement structure usually results in a significantly reduced subgrade support causing irrecoverable strains in the asphalt and thus cracking. The presence or the availability of groundwater below the pavement structure during the winter months allows for capillary action to occur, thus stimulating the growth of ice lenses and subsequent alligator cracking of the asphalt surface.

## 3.18. cont'd

The performance of roadways is a function of subgrade type, pavement structure and proper drainage for a given set of environment and traffic load conditions. Selected granular materials to form the pavement structure will perform adequately on the natural subgrades provided proper drainage is assured. In the depressed areas, a subdrainage system is recommended.

3.19. Stability of Clay Slopes

We have examined both the short and long term stability of the proposed modifications to the existing clay slopes by cutting into and/or by adding vertical retaining walls. The short term stability is controlled by the undrained shear strength of the underlying clay beneath the retaining walls and within the slopes. The long term stability is controlled by drained parameters of cohesion and friction in a condition of maximum porewater pressure caused by groundwater recharge usually corresponding to spring thaw when rapid snowmelt coincides with heavy rainfall.

We have made a statistical analysis of the probable average undrained shear strength profiles of the clay soils at the site based on vane shear strength measurements taken in boreholes from the present and previous studies. The drained shear strength parameters used in our long term slope stability calculations are based on a slope stability study of the R.M.O.C. prepared by Klugman and Chung in 1976. Their study included both man-made cuts in clay slopes and natural slopes.



## 3.19 cont'd

As part of the present study, all the assumptions and parameters were used in circular arc slope stability analysis by using a modern computer program including the Bishop formula.

Two main factors influence the choice of calculated margins of stability needed. First is the various uncertainties involved in the stability analysis of slopes. Second is the consequences of failure.

In an attempt to satisfy ourselves of the applicability or the validity of using the Klugman and Chung regional parameters for this site, we have made some inquiries with regards to previous site specific work that was carried out. We understand that for a 7.5m high slope cut at slightly less than 2:1 (actually 1.9:1), there were signs of surface sloughing only with no indication of rotational failure occurring.

Based on the Klugman and Chung parameters, the margins of safety of the latter observed slope would have been about ten (10%) percent above unity. For the same height of slope, the margins of safety would become about forty (40%) percent above unity, for an inclination of 2.5 horizontal to 1 vertical.

As a result of the above calculations, and based on past performance for a relatively marginal condition of stability, we have judged that the requirement of a calculated factor of safety of 1.4 would be essential when using these parameters.

## 3.19. cont'd

In order to obtain this level of factor of safety at the site, the clay slopes should not be cut steeper than 2.5 horizontal to 1 vertical. For slopes with a height of five (5) metres or less, the inclination could be 2.0 horizontal to 1 vertical.

In the case of retaining walls, the base of the wall should be lowered onto natural clay, through the fill, at an elevation corresponding to the flatter land adjacent and to the north. In the case where the land continues sloping downward to the north, the underside of footing can be limited to a depth corresponding to the elevation of the land 7.5 metres away. We have enclosed a small sketch on a plate accompanying this report. A review of the final design as well as during construction would be needed in case modifications have occurred since the preliminary drawings from which we have been working.

A surficial layer of vegetation should be placed on the natural and/or clay fill exposed by cutting into the slopes. This layer would reduce the amount of erosion and rilling caused by heavy rainfalls and snowmelt water.

There are two (2) areas that required analysis in order to evaluate how feasible it would be to produce relatively steep construction cuts. In order to construct the pedestrian tunnel, an eight (8) metre deep excavation is planned with a 1.2m side wall and a backslope of one on one (1:1). Such excavations have been carried out successfully in the Ottawa Area in the past, in similar clays for reasonable construction periods such as three (3) to six (6) months. Actually,

## 3.19 cont'd

the calculated factors of safety in the short term based on measured shear strengths are about 2.0 but are less than unity (1.0) in the long term. The margins of safety against instability are time dependent but the change from a short to a long term condition can also be significantly influenced by factors such as wetting and drying, freezing and thawing, heavy rainfalls and snowmelt as well as undercutting or oversteepening. The contractor should therefore be responsible for performing the work within a time frame compatible with adequate margins of safety. He should also be required to protect and maintain the slopes under the professional advice of a geotechnical engineer.

Another area of concern with regards to temporary excavations in clay slopes, is between stations 10+400 and 10+600, where a sewer trench about 2.5m deep needs to be dug close to the toe of a clay cut at 2.5 horizontal units to 1 vertical (2.5:1). The calculated factors of safety in the short term, based on measured shear strengths are about 1.8 but again less than unity (1.0) in the long term. We recommend that the length of trench open at any time be limited to ten (10) metres in order to maintain adequate margins of safety against instabilities. The sewer trench backfill should be compacted to 95% Standard Proctor Density, so that the lateral restraint of the soil adjacent to the trench will be restored. The bulk excavation that will be required to remove the unselected fill will increase the total height of cut in the clay slopes. At some locations, these cuts may be more critical than the effect of the sewer trench

## 3.19. cont'd

cut on the adjacent cut slope. The construction may require staging in order to maintain adequate margins of safety. Because of all the factors that affect the stability of construction cuts in clay, the contractor should retain a geotechnical engineer prior to starting the excavation in this area.

3.20. Reinforced Earth Walls

The transitway route can be designed to tolerate some movement and therefore, we are recommending that the relatively more flexible "reinforced earth" systems be considered. Reinforcing strips are placed in the granular backfill and mechanically attached to the facing units of the wall. The two (2) likely types of granular backfill suitable for reinforced earth walls would be Granular 'B' (type I and type II). The properties of these materials are given in following section of this report. The final selection of the backfill should be based on a cost comparison since the lower cost material would require a slightly higher cost of reinforcement.

The wall should be designed to resist active earth pressures as well as any vehicular surcharge loading. Drainage should be provided at the base of the wall and a geotextile layer should be installed behind the wall to avoid loss of ground between the facing units.

In some locations, in order to maintain adequate margins of stability of the slope and wall system, a concrete retaining wall will need to be lowered to match adjacent lower ground.

### 3.21. Lateral Earth Pressures

The magnitude of earth pressures acting or capable of providing lateral resistance on retaining walls is a function of the backfill material used adjacent to these walls. The other major factor to be recognized is the rigidity of the walls.

Where reinforced concrete walls are unyielding, at-rest earth pressures will be developed rather than active earth pressures.

Design values can be recommended for free draining non-frost susceptible materials such as: -

Granular 'A' or	-soil density $22.5 \text{ kN/m}^3$
Granular 'B' (type II)	-at-rest pressure co-efficient $K_0 = 0.43$ (SLS II) $0.51$ (ULS)
	-active pressure co-efficient $K_a = 0.27$ (SLS II) $0.34$ (ULS)
	-triangular distribution
Granular 'B' (type I)	-soil density $21.0 \text{ kN/m}^3$
	-at-rest pressure co-efficient $K_0 = 0.50$ (SLS II) $0.58$ (ULS)
	-active pressure co-efficient $K_a = 0.33$ (SLS II) $0.41$ (ULS)
	-triangular distribution

The above values are based on both measured and estimated values of regional parameters. No values for passive resistance are given because large movements are required to develop these.

## 3.21. cont'd

The values given for the coefficient of active earth pressures given above are for horizontal backfill behind a vertical wall. We have considered various inclinations for the backslope with the results as follows: -

Active Pressure Coefficient ( $K_a$ )		
<u>Granular 'A' or Granular 'B' (Type II)</u>		
Backslope	SLS II	ULS
2.0:1	0.43	0.64
2.5:1	0.36	0.49
3.0:1	0.33	0.44

<u>Granular 'B' (Type I)</u>		
Backslope	SLS II	ULS
2.25:1	0.52	0.79
2.5:1	0.47	0.65
3.0:1	0.42	0.55

In the case of a backslope behind a retaining wall, the earth pressure distribution is also triangular but the direction of the resultant is parallel to the inclination of the backslope.

It is assumed that adequate drainage will be provided behind the retaining walls and that hydrostatic pressures will be relieved.

3.22. Excavation Bracing

Temporary shoring will be required in order to maintain the existing railroad service in operation while the southerly portion of the pedestrian tunnel is being constructed.

## 3.22. cont'd

The shoring will need to support active earth pressures and the lateral forces resulting from E-85 railway loading with centreline of track 3.6m away.

A shoring system using soldier piles and lagging could be considered for soil support, provided that special care was used to backfill or grout behind the lagging boards, in order to keep the ground movements at the Railway to a minimum.

The movements at the train rails, if any, should be monitored constantly to check the performance of the bracing system and to detect the need for shimming or other adjustment of the rails, if this were to be found necessary.

A tie-back system could be considered for restraint of soldier piles, these tie-backs would be quite long, as would the soldier piles since both tie-backs and pile will need to achieve their resistance in the underlying till and rock formations.

The loads for which the shoring system should be designed can be specified so that appropriate margins of safety will be included in all the designs. In view of the railway operations close to the top of the excavation, suitably conservative design pressures have been calculated for the soil loadings and train loadings. These are shown on Plate No. 52 included in this report.

## 3.22. cont'd

The above pressures do not contain any allowance for frost pressures that can be significant. Should the work be carried out during the winter months, provisions for heating or allowances for frost loads would need to be included in the design.

3.23. Variations in Subsurface Conditions

Variations in subsurface conditions between locations tested may be found at the time of construction. Significant variations should be reported to the supervising authority for suitable action.

3.24. Contractor's Use of this Report

Our report contains engineering recommendations on geotechnical aspects of the project based on our interpretation of subsurface information obtained and present project requirements. It must be stressed that all recommendations in our report are provided for the guidance of the design engineers as they pertain to this project. Contractors bidding on, or undertaking any work on this project should examine the factual results of the investigation, satisfy themselves as to the adequacy of this information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, safety, schedule and equipment capabilities.



#### 4. DETAILS OF THE INVESTIGATION

Fifteen (15) boreholes, eight (8) test pits and three (3) hand auger holes were made as part of the present study in the locations shown on Plate No. 1 of this report. The boreholes were performed by both truck and track mounted CME-55 drill rigs equipped with hollow stem augers and operated by experienced drillers under constant direction from our technicians. The test pits were dug by an excavating contractor using a tire mounted backhoe with extendable bucket. The hand auger holes were done by our forces with manual gas operated equipment.

In general, at all boreholes, split barrel samples were recovered in the overburden soils above bedrock. Standard penetration resistance tests were carried out simultaneously with the split barrel sampling. Some undisturbed clay samples were recovered by means of a special piston tube sampler. The undrained shear strength of the clay was determined by means of a field vane. The underlying bedrock was core drilled with BX-size diamond bits. Some boulders also required diamond drilling.

A careful watch was kept during drilling for drops of drill rods and loss of drill water in order to detect any discontinuities and evaluate the soundness of the bedrock.

## 4. cont'd

Corrosion sounder testing was carried out one (1) metre away from two (2) boreholes in the vicinity of the Transitway Station. The results of these special tests are included in this report.

All soil samples and rock cores were brought to our laboratory to be examined and tested. Moisture content determinations and visual classifications were made on all soil samples. Three (3) drained consolidation tests were carried out on undisturbed clay samples. Routine chemical tests were carried out on groundwater samples. Two (2) grain size analysis and three (3) Atterberg limits were performed. The rock cores were examined and described in the laboratory.

This report prepared by:



*Michel St-Louis*

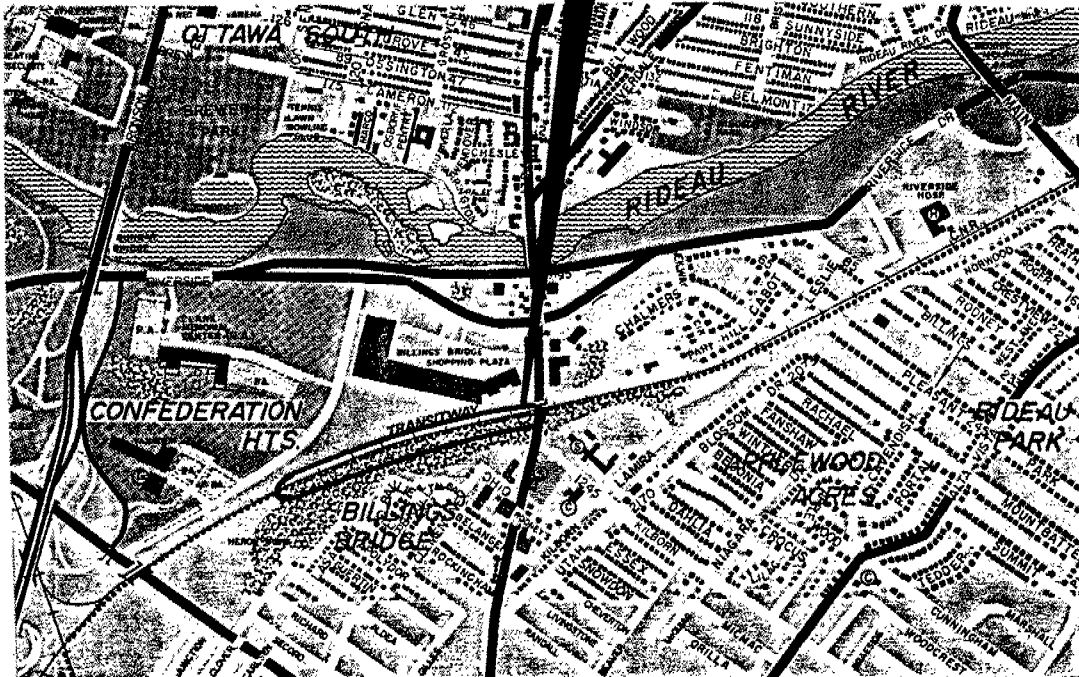
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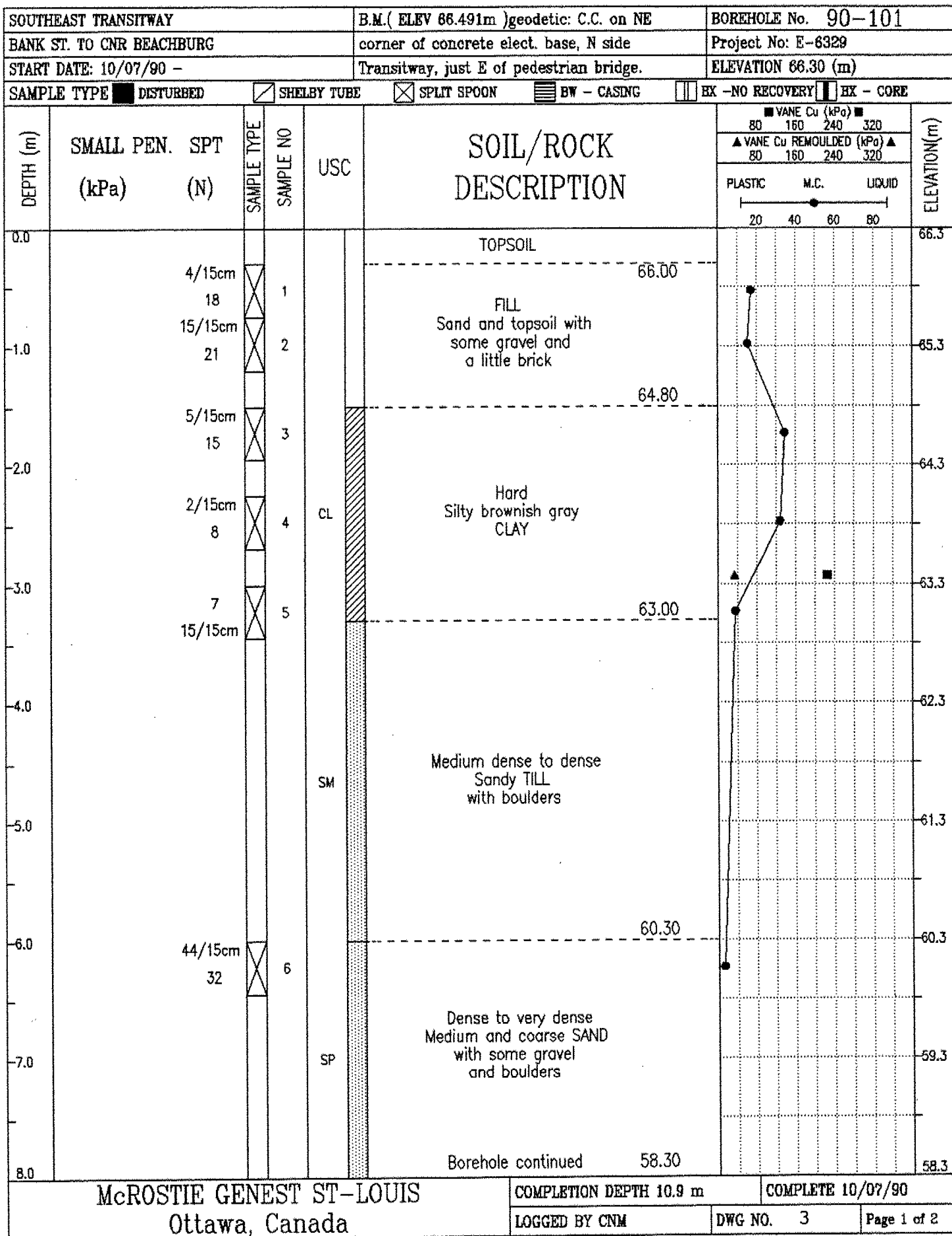
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KEY PLAN  
SOUTHEAST TRANSITWAY

SCALE 1:18,500

PLATE N° 1

# OVERSIZE DRAWING(S)



SOUTHEAST TRANSITWAY				B.M. (ELEV 66.491m) geodetic: C.C. on NE		BOREHOLE No. 90-101	
BANK ST. TO CNR BEACHBURG				corner of concrete elect. base, N side		Project No: E-6329	
START DATE: 10/07/90 -				Transitway, just E of pedestrian bridge.		ELEVATION 66.30 (m)	
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON		<input type="checkbox"/> BW - CASING	
				<input type="checkbox"/> BX - NO RECOVERY		<input type="checkbox"/> HX - CORE	

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)				VANE Cu REMOULDED (kPa)				ELEVATION (m)
							80 160 240 320				80 160 240 320				
							PLASTIC		M.C.		LIQUID				
							20 40 60 80		20 40 60 80		20 40 60 80				
8.0						Dense to very dense Medium and coarse SAND with some gravel and boulders								58.3	
						Water level July 27/90 - ELEV 57.66m Water level July 12&19/90 - ELEV 57.50m									
-9.0						Auger refusal -								57.3	
						Fractured Black SHALE (C.R. = 100%)									
-10.0														56.3	
						Fractured Black SHALE (C.R. = 99%)									
-11.0						Bottom of hole								55.3	
-12.0														54.3	
-13.0														53.3	
-14.0														52.3	
-15.0														51.3	
-16.0														50.3	

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 10.9 m		COMPLETE 10/07/90	
		LOGGED BY CNM		DWG NO. 4	
				Page 2 of 2	

SOUTHEAST TRANSITWAY			B.M.( ELEV 86.491m )geodetic: C.C. on NE			BOREHOLE No. 90-102		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 11/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 72.93 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input type="checkbox"/> BW - CASING			<input type="checkbox"/> BX - NO RECOVERY		
						<input type="checkbox"/> BX - CORE		

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)				ELEVATION(m)
							REMOULDED (kPa)				
							80	160	240	320	
0.0						FILL Sand with some gravel					72.9
-1.0											71.9
-1.83					CL	Hard Sandy brownish gray CLAY					70.9
-2.0						Bottom of hole					638+
-2.83											69.9
-4.0											68.9
-5.0											67.9
-6.0											66.9
-7.0											65.9
-8.0											64.9

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 2.1 m	COMPLETE 11/07/90
LOGGED BY CNM		DWG NO. 5	Page 1 of 1

SOUTHEAST TRANSITWAY		B.M.( ELEV 66.491m )geodetic: C.C. on NE		BOREHOLE No. 90-103		
BANK ST. TO CNR BEACHBURG		corner of concrete elect. base, N side		Project No: E-6329		
START DATE: 09/07/90 -		Transitway, just E of pedestrian bridge.		ELEVATION 65.67 (m)		
SAMPLE TYPE	DISTURBED	<input checked="" type="checkbox"/> SHELBY TUBE	<input checked="" type="checkbox"/> SPLIT SPOON	<input checked="" type="checkbox"/> BW - CASING	<input type="checkbox"/> BX - NO RECOVERY <input type="checkbox"/> BX - CORE	
DEPTH (m)	SMALL PEN. SPT (kPa) (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	ELEVATION(m)
0.0					FILL Clay with some topsoil and a little wood	65.7
-1.0	2 2/15cm	X	1			64.7
-2.0	2 2/15cm	X	2			63.7
-3.0	1/15cm 4	X	3	CL	Very stiff Silty brownish gray CLAY	62.7
-4.0	1/15cm 5	X	4	ML	Very loose to loose SILT	61.7
-5.0						60.7
-6.0	40	X	5	SP	Dense Medium SAND with some coarse sand and a little gravel with boulders	59.7
-7.0						58.7
-8.0	Auger refusal -				Limestone and granitic BOULDERS in sand	58.27
					Borehole continued	57.67

■ VANE Cu (kPa) ■  
80 160 240 320  
▲ VANE Cu REMOULDED (kPa) ▲  
80 160 240 320

PLASTIC M.C. LIQUID  
|-----|  
20 40 60 80

McROSTIE GENEST ST-LOUIS  
Ottawa, Canada
COMPLETION DEPTH 11.6 m  
LOGGED BY CNM
COMPLETE 10/07/90  
DWG NO. 6
Page 1 of 2



SOUTHEAST TRANSITWAY		B.M. (ELEV 86.491m) geodetic: C.C. on NE		BOREHOLE No. 90-103	
BANK ST. TO CNR BEACHBURG		corner of concrete elect. base, N side		Project No: E-6329	
START DATE: 09/07/90 -		Transitway, just E of pedestrian bridge.		ELEVATION 65.67 (m)	
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON	
		<input type="checkbox"/> BW - CASING		<input type="checkbox"/> BX - NO RECOVERY	
				<input type="checkbox"/> HX - CORE	

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)			VANE Cu REMOULDED (kPa)			ELEVATION (m)		
							80	160	240	320	80	160		240	320
							PLASTIC M.C. LIQUID								
8.0						Limestone and granitic BOULDERS in sand							57.7		
9.0						Water level July 27/90 - ELEV 57.34m Water level July 19/90 - ELEV 57.07m Water level July 12/90 - ELEV 56.67m							56.7		
						Granitic and limestone BOULDERS, and fractured black SHALE (C.R. = 50%)							55.90 55.60		
10.0													55.7		
						Fractured and sound Black SHALE (C.R. = 96%)							54.7		
11.0													54.10		
						Sound and fractured Black SHALE (C.R. = 100%)							53.7		
12.0													52.7		
13.0						Bottom of hole							52.60		
													51.7		
14.0													50.7		
15.0													49.7		
16.0															

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 11.6 m	COMPLETE 10/07/90
LOGGED BY CNM		DWG NO. 7	Page 2 of 2

SOUTHEAST TRANSITWAY			B.M.( ELEV 86.491m )geodetic: C.C. on NE			BOREHOLE No. 90-104		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 11/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 72.88 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input type="checkbox"/> BW - CASING			<input type="checkbox"/> EX -NO RECOVERY <input type="checkbox"/> EX - CORE		

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	<div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>■ VANE Cu (kPa) ■</span> <span>80 160 240 320</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>▲ VANE Cu REMOULDED (kPa) ▲</span> <span>80 160 240 320</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>PLASTIC</span> <span>M.C.</span> <span>LIQUID</span> </div> <div style="text-align: center; font-size: 0.8em;"> <span>20</span> <span>40</span> <span>60</span> <span>80</span> </div>			ELEVATION(m)								
							0.0										72.9	
								HAND AUGERED BOREHOLE						FILL Sandy topsoil and gravel				
-1.0																		
					CL	Hard Brownish gray CLAY												
-2.0																		
						Bottom of hole												
-3.0																		
-4.0																		
-5.0																		
-6.0																		
-7.0																		
-8.0																		

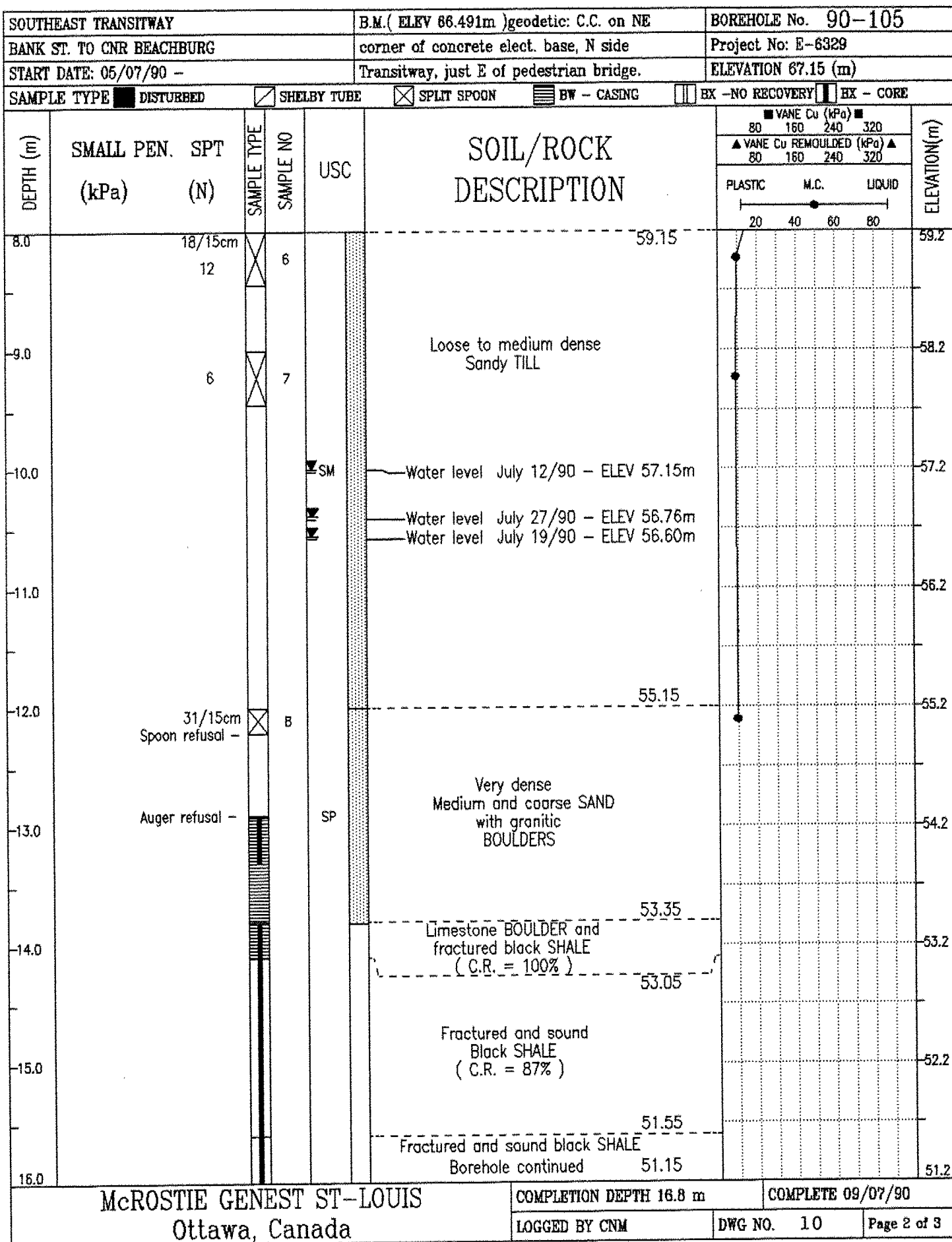
McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 2.1 m		COMPLETE 11/07/90	
		LOGGED BY CNM		DWG NO. 8 Page 1 of 1	

SOUTHEAST TRANSITWAY	B.M.( ELEV 66.491m )geodetic: C.C. on NE	BOREHOLE No.							
BANK ST. TO CNR BEACHBURG	corner of concrete elect. base, N side	Project No: E-6329							
START DATE: 05/07/90 -	Transitway, just E of pedestrian bridge.	ELEVATION 67.15 (m)							
SAMPLE TYPE    DISTURBED    SHELBY TUBE    SPLIT SPOON    BW - CASING    EX - NO RECOVERY    HX - CORE									
DEPTH (m)	SMALL PEN. SPT (kPa)      (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa) 80 160 240 320	VANE Cu REMOULDED (kPa) 80 160 240 320	PLASTIC M.C. LIQUID	ELEVATION(m)
						<div style="text-align:center;">20 40 60 80</div>			
0.0					TOPSOIL				67.2
-1.0	wt of hammer /30cm 1/15cm  2	X X	1 2						66.85
-2.0	wt of hammer /30cm 1/15cm	X X	3						65.2
-3.0	wt of hammer /30cm 1/15cm	X X	4						64.2
-4.0				CL	Stiff Silty gray CLAY				63.2
-5.0									62.2
-6.0	2 wt of hammer /15cm	X X	5						61.2
-7.0									60.2
-8.0									59.2
Borehole continued						59.15			59.2

McROSTIE GENEST ST-LOUIS Ottawa, Canada

COMPLETION DEPTH 16.8 m COMPLETE 09/07/90

LOGGED BY CNM DWG NO. 9 Page 1 of 3

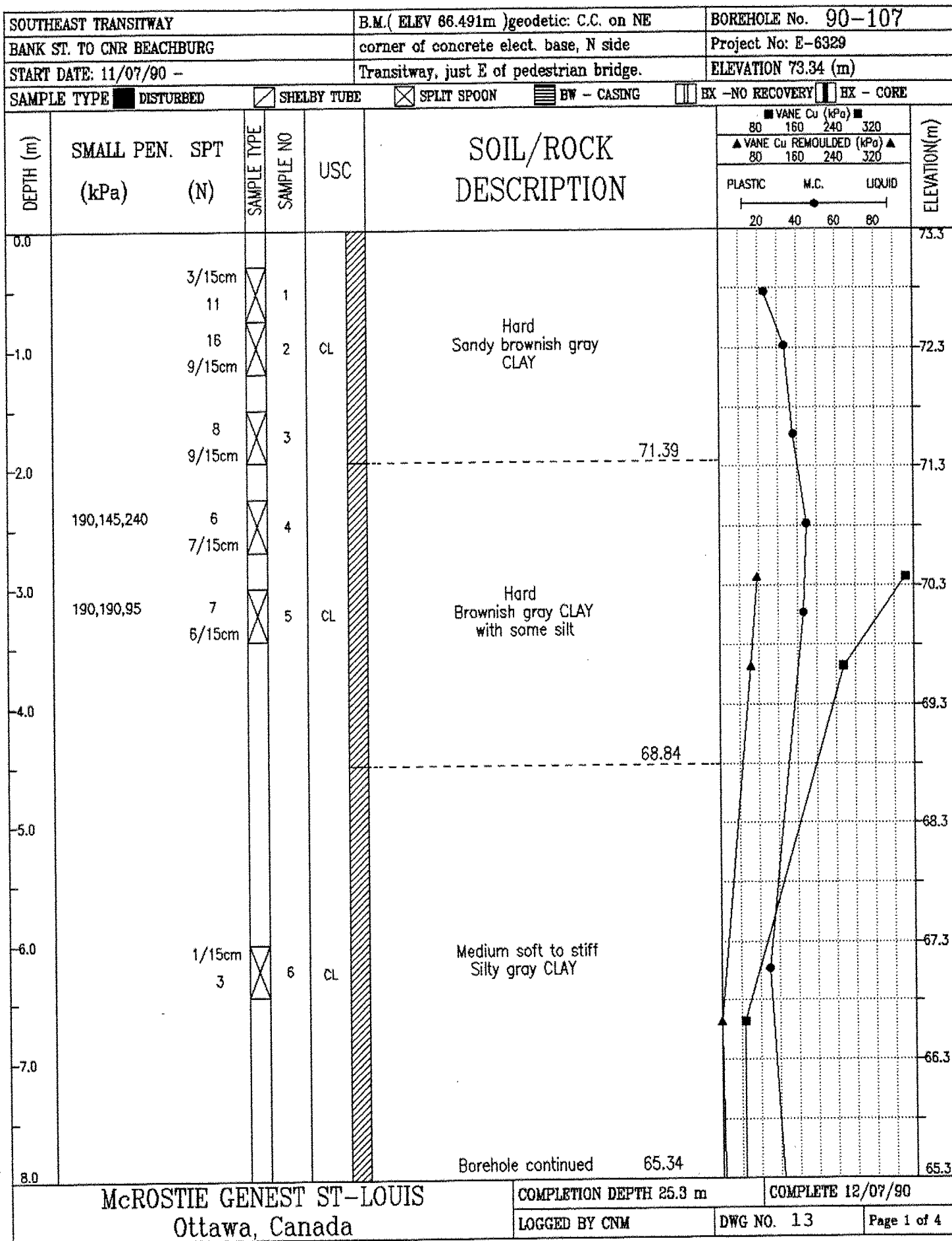


SOUTHEAST TRANSITWAY				B.M.( ELEV 66.491m )geodetic: C.C. on NE		BOREHOLE No. 90-105	
BANK ST. TO CNR BEACHBURG				corner of concrete elect. base, N side		Project No: E-6329	
START DATE: 05/07/90 -				Transitway, just E of pedestrian bridge.		ELEVATION 67.15 (m)	
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON		<input type="checkbox"/> BW - CASING	
				<input type="checkbox"/> EX - NO RECOVERY		<input type="checkbox"/> EX - CORE	

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)			ELEVATION(m)
							80	160	240	
							▲ VANE Cu REMOULDED (kPa) ▲			
							80	160	240	
							PLASTIC	M.C.	LIQUID	
							20	40	60	80
16.0						Fractured and sound Black SHALE ( C.R. = 100% )				51.2
						Bottom of hole				50.35
17.0										50.2
18.0						C.R. = Core Recovery				49.2
19.0										48.2
20.0										47.2
21.0										46.2
22.0										45.2
23.0										44.2
24.0										43.2

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 16.8 m	COMPLETE 09/07/90
LOGGED BY CNM		DWG NO. 11	Page 3 of 3





SOUTHEAST TRANSITWAY			B.M.( ELEV 66.491m )geodetic: C.C. on NE			BOREHOLE No. 90-107		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 11/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 73.34 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input type="checkbox"/> BW - CASING			<input type="checkbox"/> EX - NO RECOVERY		
						<input type="checkbox"/> HX - CORE		

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	<div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>■ VANE Cu (kPa) ■</span> <span>80 160 240 320</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>▲ VANE Cu REMOULDED (kPa) ▲</span> <span>80 160 240 320</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>PLASTIC</span> <span>M.C.</span> <span>LIQUID</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>20</span> <span>40</span> <span>60</span> <span>80</span> </div>			ELEVATION(m)							
							8.0										65.3
							9.0										64.3
							10.0										63.3
11.0									62.3								
12.0									61.3								
13.0									60.3								
14.0									59.3								
15.0									58.3								
16.0						Medium soft to stiff Silty gray CLAY			57.3								

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 25.3 m	COMPLETE 12/07/90
		LOGGED BY CNM	DWG NO. 14



SOUTHEAST TRANSITWAY			B.M. ( ELEV 68.491m )geodetic: C.C. on NE		BOREHOLE No. 90-107		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side		Project No: E-6329		
START DATE: 11/07/90 -			Transitway, just E of pedestrian bridge.		ELEVATION 73.34 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED <input checked="" type="checkbox"/> SHELBY TUBE <input checked="" type="checkbox"/> SPLIT SPOON <input checked="" type="checkbox"/> BW - CASING <input checked="" type="checkbox"/> BX -NO RECOVERY <input checked="" type="checkbox"/> RX - CORE							
DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	ELEVATION(m)
16.0						Water level July 27/90 - ELEV 56.55m Water level July 19/90 - ELEV 56.48m	57.3
17.0					CL	Medium soft to stiff Silty gray CLAY	56.3
18.0	26	14/15cm	<input checked="" type="checkbox"/>	8		55.34	55.3
19.0							54.3
20.0					SM	Medium dense Sandy TILL	53.3
21.0							52.3
22.0						51.14	51.3
23.0						Fractured and sound Black SHALE ( C.R. = 100% )	50.3
24.0						49.64 Fractured and sound black SHALE Borehole continued 49.34	49.3

**McROSTIE GENEST ST-LOUIS**  
Ottawa, Canada

COMPLETION DEPTH 25.3 m

COMPLETE 12/07/90

LOGGED BY CNM

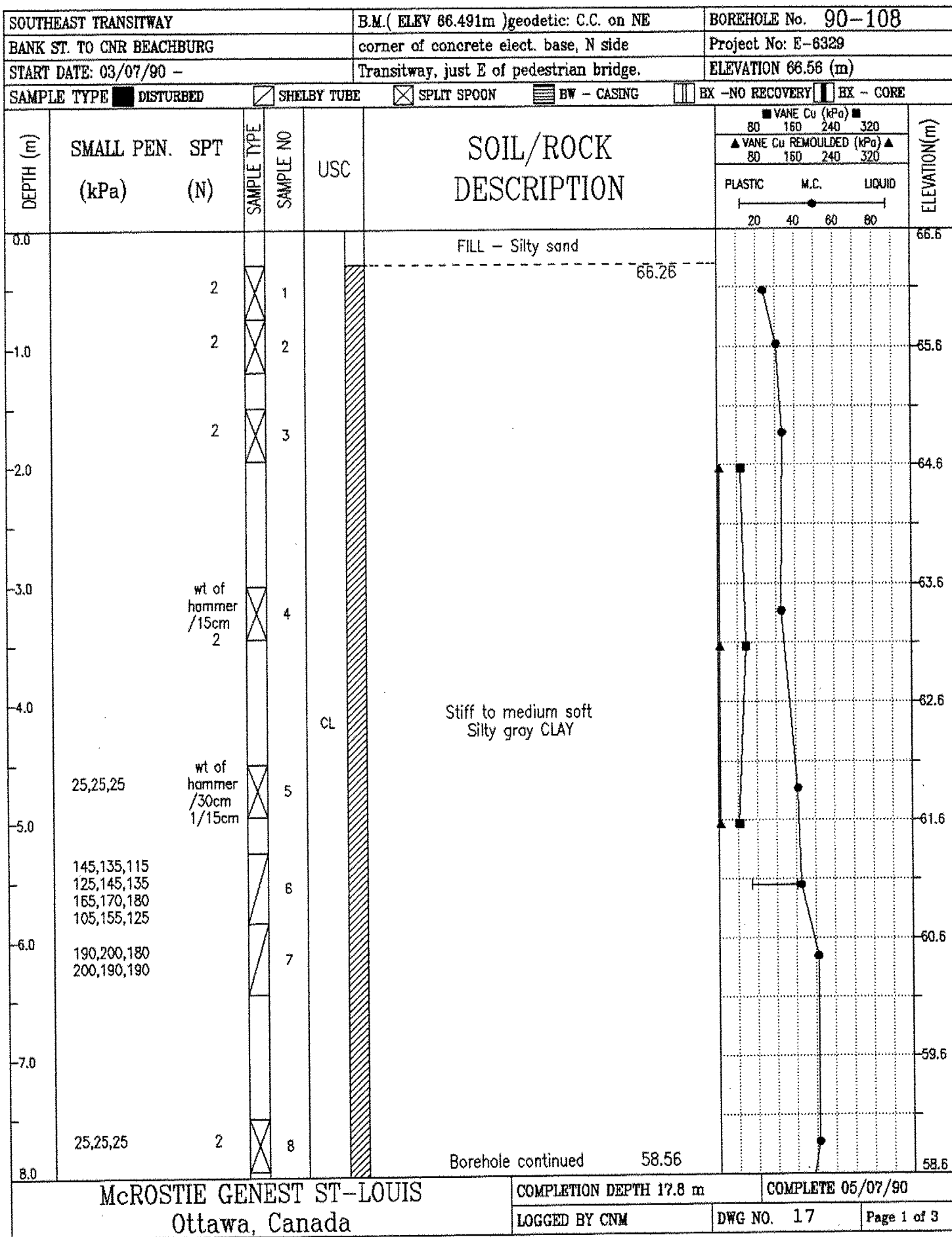
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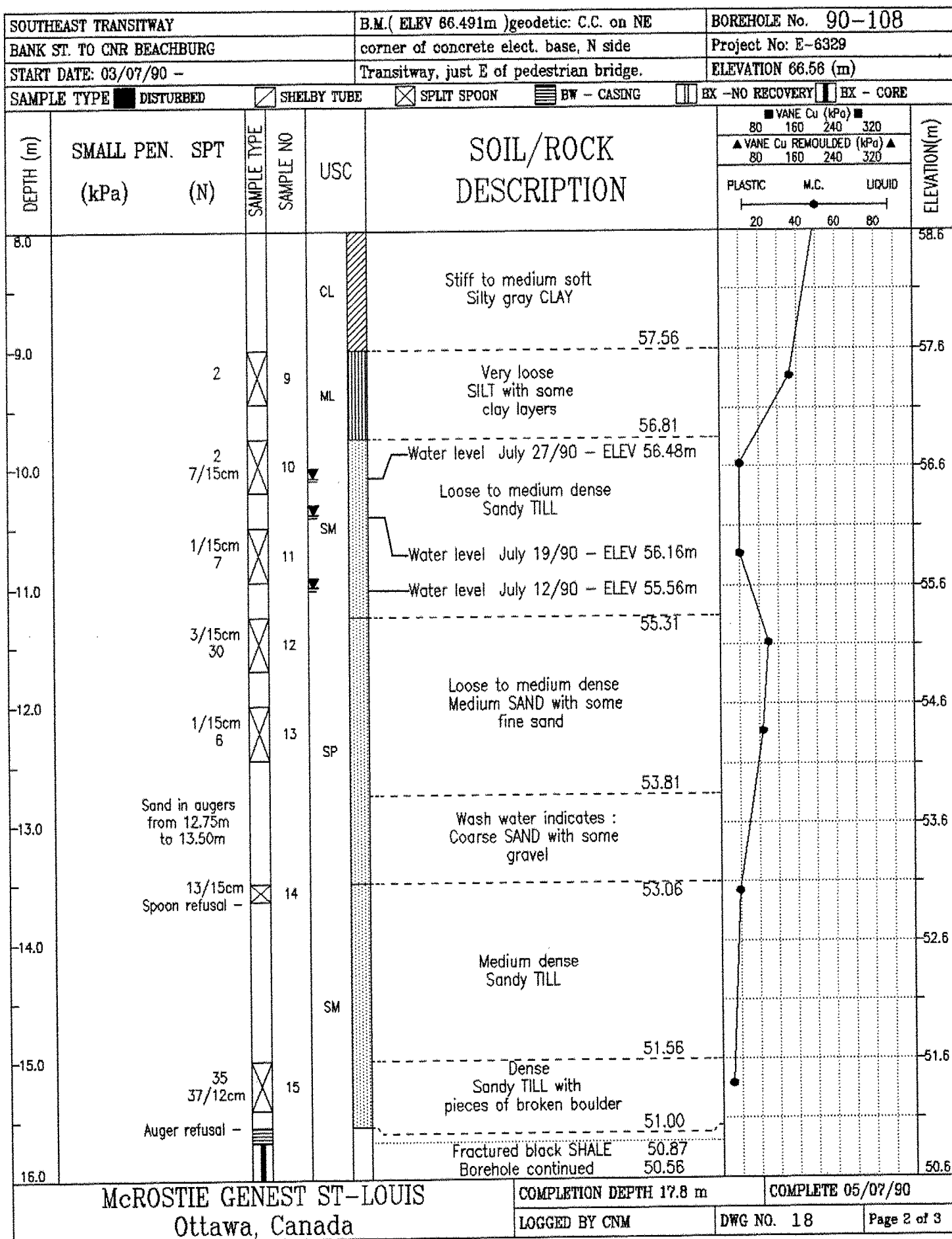
Page 3 of 4

SOUTHEAST TRANSITWAY			B.M.( ELEV 86.491m )geodetic: C.C. on NE			BOREHOLE No. 90-107		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 11/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 73.34 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input type="checkbox"/> BW - CASING			<input type="checkbox"/> BX - NO RECOVERY <input type="checkbox"/> HX - CORE		

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)				ELEVATION(m)
							▲ VANE Cu REMOULDED (kPa) ▲				
							80	160	240	320	
							PLASTIC      M.C.      LIQUID 20      40      60      80				
24.0						Fractured and sound Black SHALE ( C.R. = 100% )					49.3
25.0						Bottom of hole 48.02					48.3
26.0											47.3
27.0						C.R. = Core Recovery					46.3
28.0											45.3
29.0											44.3
30.0											43.3
31.0											42.3
32.0											41.3

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 25.3 m	COMPLETE 12/07/90
LOGGED BY CNM		DWG NO. 16	Page 4 of 4





SOUTHEAST TRANSITWAY		B.M.( ELEV 66.491m )geodetic: C.C. on NE		BOREHOLE No. 90-108				
BANK ST. TO CNR BEACHBURG		corner of concrete elect. base, N side		Project No: E-6329				
START DATE: 03/07/90 -		Transitway, just E of pedestrian bridge.		ELEVATION 66.56 (m)				
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE	<input checked="" type="checkbox"/> SPLIT SPOON	<input type="checkbox"/> BW - CASING	<input type="checkbox"/> EX -NO RECOVERY <input type="checkbox"/> HX - CORE			
DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa) 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
16.0						Fractured Black SHALE ( C.R. = 100% )		50.6
17.0						Fractured to sound Black SHALE ( C.R. = 83% )		49.6
18.0						Bottom of hole		48.6
19.0						C.R. = Core Recovery		47.6
20.0								46.6
21.0								45.6
22.0								44.6
23.0								43.6
24.0								42.6

McROSTIE GENEST ST-LOUIS  
Ottawa, Canada

COMPLETION DEPTH 17.8 m  
LOGGED BY CNM

COMPLETE 05/07/90  
DWG NO. 19

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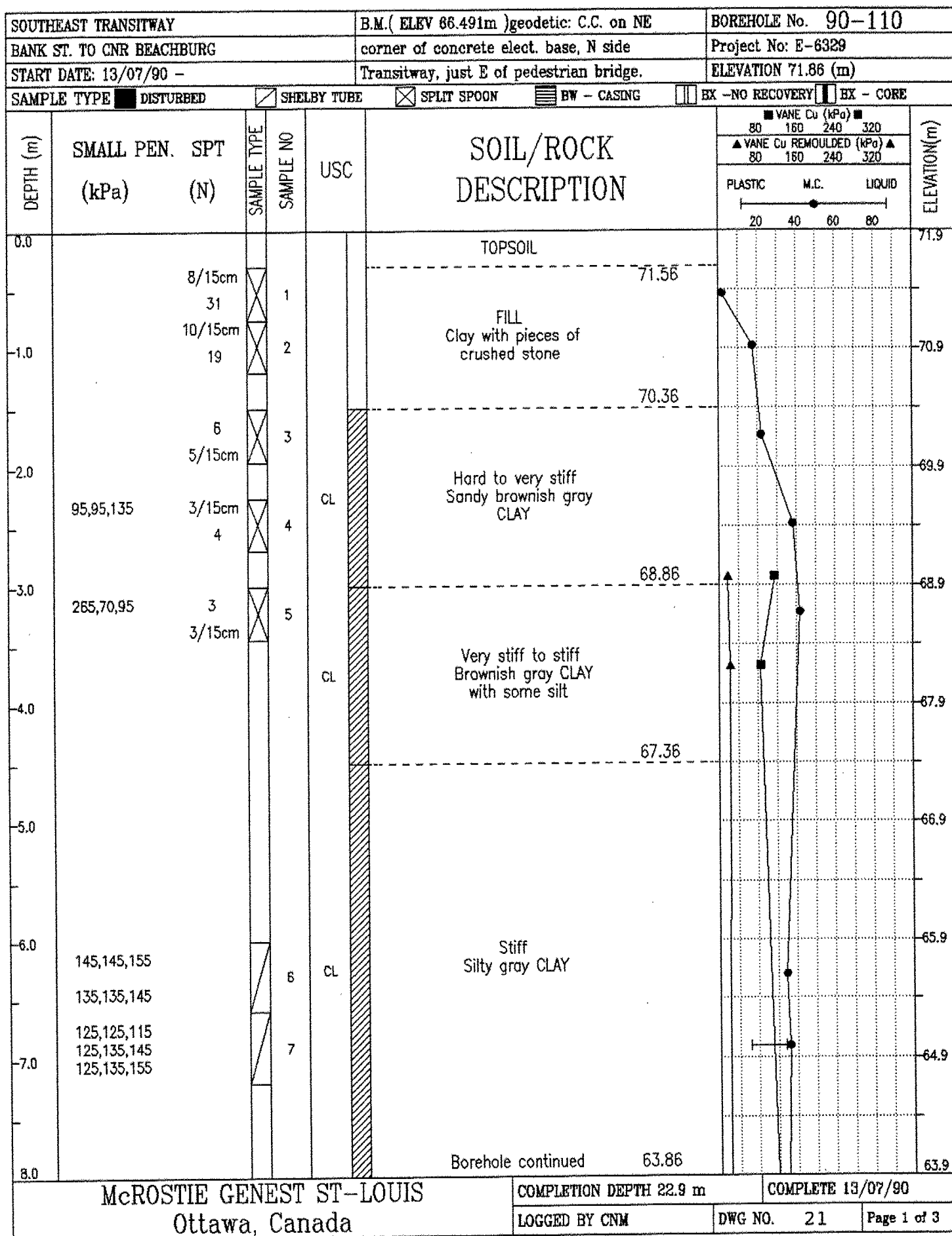
SOUTHEAST TRANSITWAY			B.M. (ELEV 66.491m) geodetic: C.C. on NE			BOREHOLE No. 90-109		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 18/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 73.39 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input checked="" type="checkbox"/> BW - CASING			<input type="checkbox"/> BX - NO RECOVERY		
						<input type="checkbox"/> BX - CORE		

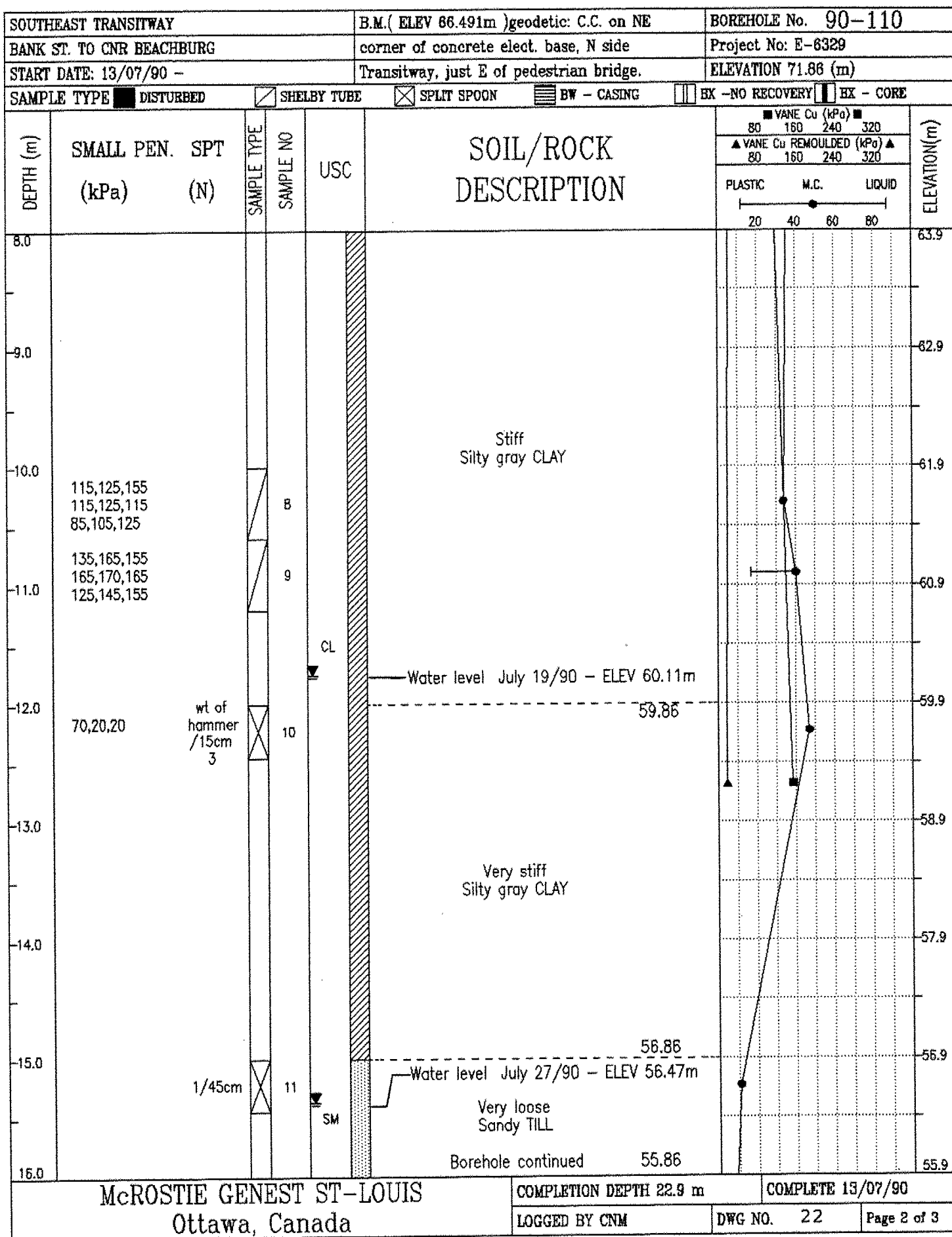
  

DEPTH (m)	SMALL PEN. SPT (kPa) (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)		VANE Cu REMOULDED (kPa)		ELEVATION (m)
						80	160	80	160	
						PLASTIC M.C. LIQUID				
0.0					FILL Sandy topsoil					73.4
-1.0										
-1.0				CL	Hard Silty brownish gray CLAY					72.4
-2.0					Bottom of hole					71.29
-3.0										70.4
-4.0										69.4
-5.0										68.4
-6.0										67.4
-7.0										66.4
-8.0										65.4

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 2.1 m	COMPLETE 18/07/90
		LOGGED BY CNM	DWG NO. 20
			Page 1 of 1







SOUTHEAST TRANSITWAY		B.M. (ELEV 86.491m) geodetic: C.C. on NE		BOREHOLE No. 90-110	
BANK ST. TO CNR BEACHBURG		corner of concrete elect. base, N side		Project No: E-6329	
START DATE: 13/07/90 -		Transitway, just E of pedestrian bridge.		ELEVATION 71.86 (m)	
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON	
		<input checked="" type="checkbox"/> BW - CASING		<input type="checkbox"/> EX - NO RECOVERY	
				<input type="checkbox"/> EX - CORE	

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)				ELEVATION (m)
							REMOULDED (kPa)				
							80	160	240	320	
							PLASTIC      M.C.      LIQUID 20      40      60      80				
16.0						Very loose Sandy TILL					55.9
17.0											54.9
18.0					SM	Medium dense to dense Sandy TILL with boulders					53.9
19.0											52.9
20.0						Limestone BOULDERS					51.9
						Limestone and granitic BOULDERS and fractured SHALE					
21.0						Fractured Black SHALE (C.R. = 100%)					50.9
22.0						Fractured and sound Black SHALE (C.R. = 97%)					49.9
23.0						Bottom of hole					48.9
24.0											47.9

McROSTIE GENEST ST-LOUIS

Ottawa, Canada

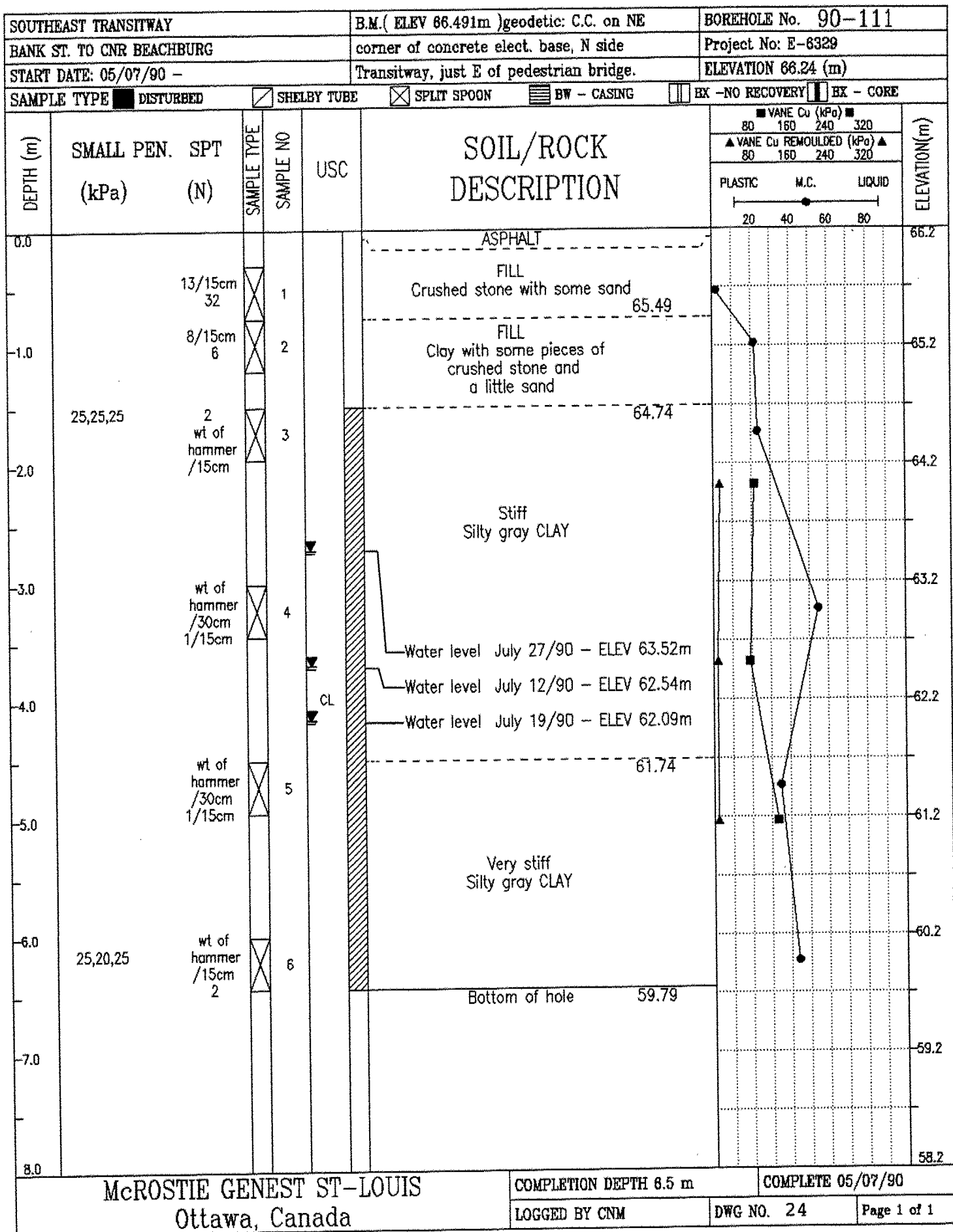
COMPLETION DEPTH 22.9 m

LOGGED BY CNM

COMPLETE 13/07/90

DWG NO. 23

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McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-112

Date :

JULY 3, 1990

SOUTH EAST TRANSITWAY

ELEV. 72.31	DEPTH in metres	DESCRIPTION	REMARKS
		FILL	
		clay with some sand	
		gravel, topsoil,	
70.31	-- 2 --	broken rock, wood and	
		slabs of concrete	
		(0.9x0.45x0.20)	
68.31	-- 4 --		
66.91	5.40		
66.51	5.80	brownish gray CLAY	
	-- 6 --	Bottom of pit	
	NOTE:	unable to dig any deeper	
		with this equipment	
		FORD 555C EXTEND -A- HOE	
	-- 8 --		
			Plate No. 25

SOUTHEAST TRANSITWAY			B.M. (ELEV 86.491m) geodetic: C.C. on NE			BOREHOLE No. 90-112A		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 28/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 72.31 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input checked="" type="checkbox"/> PROBING			<input type="checkbox"/> NO RECOVERY		
						<input type="checkbox"/> CORE		

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)			ELEVATION (m)
							80	160	240	
							▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320			
							PLASTIC M.C. LIQUID 20 40 60 80			
0.0										72.3
1.0										71.3
2.0										70.3
3.0										69.3
4.0										68.3
5.0										67.3
6.0										66.3
7.0										65.3
8.0										64.3

McROSTIE GENEST ST-LOUIS			COMPLETION DEPTH 7.1 m			COMPLETE 28/07/90		
Ottawa, Canada			LOGGED BY CNM			DWG NO. 26		
						Page 1 of 1		

McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-113

Date :

JULY 3, 1990

SOUTH EAST TRANSITWAY

ELEV. 70.43	DEPTH in metres	DESCRIPTION	REMARKS
		FILL	
		clay with some topsoil	
68.43	-- 2 --	sand, brick, wood,	
		rock slabs (limestone)	
		pieces of concrete	
		and steel	
66.43	-- 4 --		
65.83	4.60	Bottom of pit	no water seepage
	NOTE	unable to dig any deeper with this equipment	
	-- 6 --		
	-- 8 --		
			Plate No.27

SOUTHEAST TRANSITWAY			B.M. (ELEV 86.491m) geodetic: C.C. on NE			BOREHOLE No. 90-113A		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 28/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 70.43 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input type="checkbox"/> PROBING			<input type="checkbox"/> NO RECOVERY		
						<input checked="" type="checkbox"/> CORE		

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)		VANE Cu REMOULDED (kPa)		ELEVATION (m)
							80	160	80	160	
0.0											70.4
1.0											69.4
2.0											68.4
3.0											67.4
4.0											66.4
5.0											65.4
6.0											64.4
7.0											63.4
8.0											62.4

BOREHOLE ADVANCED BY						
AUGERING TO 4.50m						
-See Test Pit 90-113						
FILL						
Clay with some topsoil						
65.93						
FILL						
Clay with some sand and gravel						
64.58						
FILL - Clay with a little topsoil						
Borehole continued						
62.78						
62.43						

McROSTIE GENEST ST-LOUIS			COMPLETION DEPTH 9.3 m			COMPLETE 28/07/90		
Ottawa, Canada			LOGGED BY CNM			DWG NO. 28		
						Page 1 of 2		

SOUTHEAST TRANSITWAY				B.M. (ELEV 66.491m) geodetic: C.C. on NE		BOREHOLE No. 90-113A	
BANK ST. TO CNR BEACHBURG				corner of concrete elect. base, N side		Project No: E-6329	
START DATE: 28/07/90 -				Transitway, just E of pedestrian bridge.		ELEVATION 70.43 (m)	
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON		<input type="checkbox"/> PROBING	
		<input type="checkbox"/> NO RECOVERY		<input type="checkbox"/> CORE			

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)		VANE Cu REMOULDED (kPa)		ELEVATION (m)	
							80	160	240	320		80
8.0		7	<input checked="" type="checkbox"/>	6	CL	FILL Clay with a little topsoil					62.4	
		6		7		Silty gray CLAY						
		7		8								
-9.0		8		5/15cm								
						Bottom of hole					61.13	
-10.0											60.4	
-11.0											59.4	
-12.0											58.4	
-13.0											57.4	
-14.0											56.4	
-15.0											55.4	
16.0											54.4	

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 9.3 m		COMPLETE 28/07/90	
		LOGGED BY CNM	DWG NO. 29	Page 2 of 2	

McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-114

Date :

JULY 3, 1990

SOUTH EAST TRANSITWAY

ELEV.  
72.78

DEPTH  
in metres

DESCRIPTION

REMARKS

FILL

clay with some topsoil,  
brick, gravel, sand,  
wood, a little asphalt,  
concrete and steel

70.78

-- 2 --

68.78

-- 4 --

66.98

5.80  
-- 6 --

Bottom of pit

no water  
seepage

NOTE:

unable to dig any deeper  
with equipment

-- 8 --

Plate No. 30



SOUTHEAST TRANSITWAY				B.M.( ELEV 66.491m )geodetic: C.C. on NE		BOREHOLE No. 90-114A	
BANK ST. TO CNR BEACHBURG				corner of concrete elect. base, N side		Project No: E-6329	
START DATE: 27/07/90 -				Transitway, just E of pedestrian bridge.		ELEVATION 72.78 (m)	
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON		<input type="checkbox"/> PROBING	
				<input type="checkbox"/> NO RECOVERY		<input type="checkbox"/> CORE	

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)				ELEVATION(m)
							REMOULDED (kPa)				
							80	160	240	320	
							PLASTIC	M.C.	LIQUID		
							20	40	60	80	
0.0											72.8
-1.0											71.8
-2.0											70.8
-3.0											69.8
-4.0											68.8
-5.0											67.8
-6.0											66.8
-7.0											65.8
-8.0											64.8

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 7.5 m	COMPLETE 27/07/90
		LOGGED BY CNM	DWG NO. 31
		Page 1 of 1	

McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-115

Date :

JULY 3, 1990

SOUTH EAST TRANSITWAY

ELEV. 73.03	DEPTH in metres	DESCRIPTION	REMARKS
		FILL	
71.03	-- 2 --	clay with some topsoil,  brick, gravel, sand,  and wood with a little  asphalt and concrete	
69.03	-- 4 --		
67.23	5.80 -- 6 --	Bottom of pit	no water seepage
	NOTE:	unable to dig any deeper	
	-- 8 --		
			Plate No. 32

SOUTHEAST TRANSITWAY			B.M.( ELEV 86.491m )geodetic: C.C. on NE			BOREHOLE No. 90-115A				
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329				
START DATE: 27/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 73.03 (m)				
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON				
			<input checked="" type="checkbox"/> PROBING			<input type="checkbox"/> NO RECOVERY				
						<input type="checkbox"/> CORE				
DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION			<div> <div> <div>■ VANE Cu (kPa) ■</div> <div>80 160 240 320</div> </div> <div> <div>▲ VANE Cu REMOULDED (kPa) ▲</div> <div>80 160 240 320</div> </div> </div> <div> <div>PLASTIC</div> <div>M.C.</div> <div>LIQUID</div> </div> <div> <div>20</div> <div>40</div> <div>60</div> <div>80</div> </div>	ELEVATION(m)
0.0						BOREHOLE ADVANCED BY AUGERING TO 5.25m -See Test Pit 90-115			73.0	
-1.0									72.0	
-2.0									71.0	
-3.0									70.0	
-4.0									69.0	
-5.0									68.0	
-6.0			14/15cm	1		67.78			67.0	
			12	2		FILL Clay with some sand and a little gravel and roots				
			2/15cm	3		66.43				
			8	4						
			3/15cm	5						
			4							
-7.0			2		CL	Silty gray CLAY			66.0	
			2						65.0	
-8.0						Bottom of hole 65.53			65.0	

McROSTIE GENEST ST-LOUIS  
Ottawa, Canada

COMPLETION DEPTH 7.5 m

COMPLETE 27/07/90

LOGGED BY CNM

DWG NO. 33

Page 1 of 1

McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-116

Date :

JULY 3, 1990

SOUTH EAST TRANSITWAY

ELEV.  
73.55

DEPTH  
in metres

DESCRIPTION

REMARKS

FILL

clay with some

71.55

-- 2 --

brick, topsoil, gravel,

and a little wood

69.55

-- 4 --

67.75

5.80

-- 6 --

Bottom of pit

slight water  
seepage at  
ELEV. 68.95

NOTE:

unable to dig any deeper

-- 8 --

Plate No. 34

SOUTHEAST TRANSITWAY			B.M.( ELEV 86.491m )geodetic: C.C. on NE			BOREHOLE No. 90-116A		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 27/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 73.55 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input type="checkbox"/> PROBING			<input type="checkbox"/> NO RECOVERY		
						<input checked="" type="checkbox"/> CORE		

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)			ELEVATION(m)
							80	160	240	
							▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320			
							PLASTIC M.C. LIQUID 20 40 60 80			
0.0										73.6
-1.0										72.6
-2.0										71.6
-3.0										70.6
-4.0										69.6
-5.0										68.6
-6.0										67.6
-7.0										66.6
-8.0										65.6

BOREHOLE ADVANCED BY AUGERING TO 5.25m -See Test Pit 90-116					
wt of hammer /15cm	2	1			68.30
FILL					
Gray clay with some roots					
2	2				67.85
wt of hammer /45cm	3		CL		
Silty gray CLAY					
Bottom of hole 66.95					

McROSTIE GENEST ST-LOUIS		COMPLETION DEPTH 6.6 m		COMPLETE 27/07/90	
Ottawa, Canada		LOGGED BY CNM		DWG NO. 35	
				Page 1 of 1	

SOUTHEAST TRANSITWAY			B.M. (ELEV 66.491m) geodetic: C.C. on NE			BOREHOLE No. 90-117		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 10/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 65.82 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input type="checkbox"/> BW - CASING			<input type="checkbox"/> EX - NO RECOVERY		
			<input type="checkbox"/> EX - CORE					

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)		ELEVATION (m)
							80	160	
0.0						ASPHALT			65.6
						FILL - Sand and pieces of crushed stone			
						FILL			65.32
						Medium sand with some gravel and fine sand			
-1.0						Medium soft to stiff Sandy brownish gray CLAY			64.87
									64.6
									64.12
-2.0						Medium soft Silty gray CLAY			63.67
						Bottom of hole			63.6
-3.0									62.6
-4.0									61.6
-5.0									60.6
-6.0									59.6
-7.0									58.6
-8.0									57.6

McROSTIE GENEST ST-LOUIS		COMPLETION DEPTH 2.0 m		COMPLETE 10/07/90	
Ottawa, Canada		LOGGED BY CNM		DWG NO. 36	
				Page 1 of 1	

SOUTHEAST TRANSITWAY			B.M. (ELEV 66.491m) geodetic: C.C. on NE			BOREHOLE No. 90-118		
BANK ST. TO CNR BEACHBURG			corner of concrete elect. base, N side			Project No: E-6329		
START DATE: 10/07/90 -			Transitway, just E of pedestrian bridge.			ELEVATION 69.76 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input type="checkbox"/> BW - CASING			<input type="checkbox"/> BX - NO RECOVERY <input type="checkbox"/> BX - CORE		

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)		ELEVATION (m)
							80 160 240 320	80 160 240 320	
0.0						ASPHALT			69.8
						CRUSHED STONE			
		23		1		FILL - Medium sand with some			69.36
		5/15cm		2		gravel and fine sand			
		3/15cm		3		Sandy brownish gray CLAY			69.06
-1.0	65,50,50	8			CL	Medium soft to stiff			69.01
						Silty gray CLAY			
						Bottom of hole			68.56
-2.0									67.8
-3.0									66.8
-4.0									65.8
-5.0									64.8
-6.0									63.8
-7.0									62.8
-8.0									61.8

McROSTIE GENEST ST-LOUIS

Ottawa, Canada

COMPLETION DEPTH 1.2 m

LOGGED BY CNM

COMPLETE 10/07/90

DWG NO. 37

Page 1 of 1

McROSTIE GENEST ST-LOUIS  
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Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-119

Date :

AUG. 2, 1990

SOUTH EAST TRANSITWAY

ELEV.	DEPTH	DESCRIPTION	REMARKS
73.87	in metres		
		FILL	
72.87	-- 1 --	clay with some topsoil, sand, gravel, brick and pieces of concrete and asphalt	
71.87	-- 2 --		
70.87	-- 3 --	fissured silty brownish gray CLAY	
70.27	3.6	Bottom of pit	no water seepage
	-- 4 --		
			Plate No. 38



McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-120

Date :

AUG. 2, 1990

SOUTH EAST TRANSITWAY

ELEV.	DEPTH	DESCRIPTION	REMARKS
73.63	in metres		
		FILL	
72.63	-- 1 --	clay with some sand, a little gravel, brick and pieces of concrete	
71.63	-- 2 --		
71.13	2.5	fissured silty brownish gray CLAY	
70.63	-- 3 --	Bottom of pit	no water seepage
	-- 4 --		
			Plate No.39

McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-121

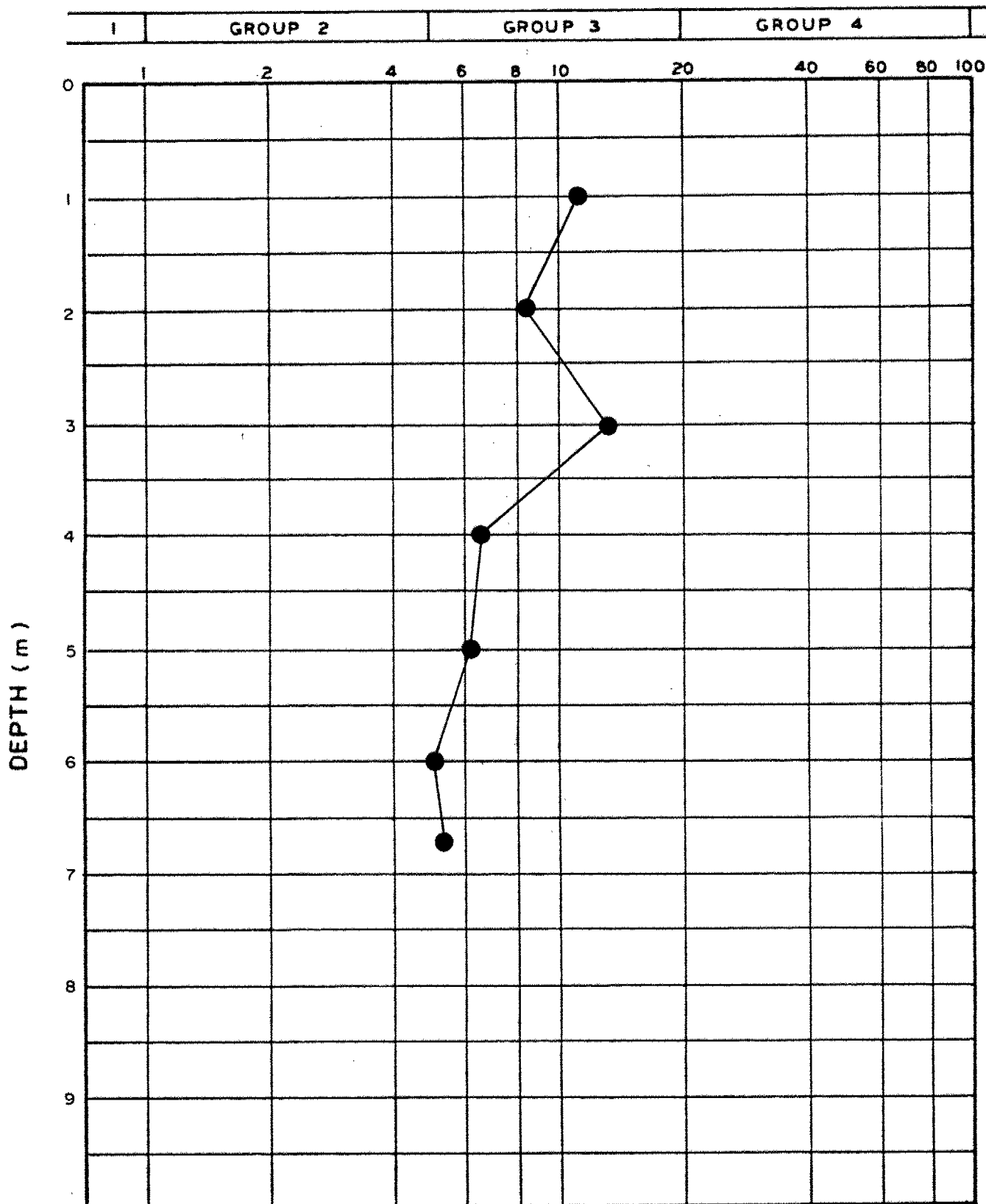
Date :

AUG. 2, 1990

SOUTH EAST TRANSITWAY

ELEV.	DEPTH	DESCRIPTION	REMARKS
73.47	in metres		
		FILL	
72.47	-- 1 --	sand, clay, gravel,	
		brick, pieces of concrete,	
		asphalt and a little wood	
71.47	-- 2 --		
70.47	-- 3 --		
70.27	3.2	fissured silty brownish gray CLAY	
69.77	3.7	Bottom of pit	no water seepage
	-- 4 --		
			Plate No.40

# CORROSION VELOCITY IN MM PER YEAR $\times 10^{-3}$



GEONOR CORROSION SOUNDER  
AS IN NGI PUBLICATION No. 42  
T. ROSENQUIST (1961)

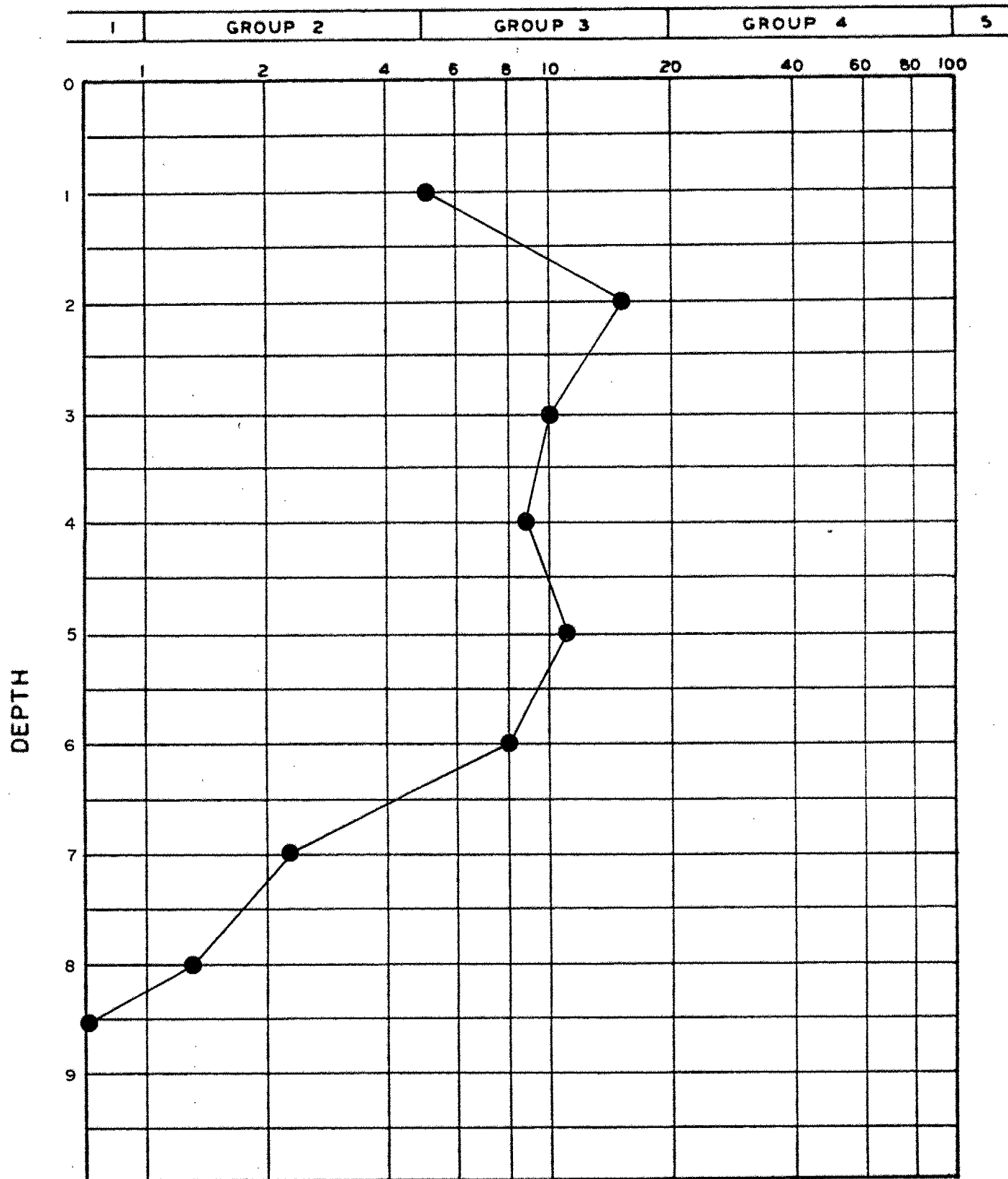
BOREHOLE 90 - 108

McROSTIE GENEST ST-LOUIS  
& ASSOCIATES LTD. & ASSOCIÉS LTÉE  
CONSULTING ENGINEERS INGENIEUR CONSEILS  
OTTAWA, CANADA

S.E. TRANSITWAY - BANK ST. TO C.N.R.  
CORROSION SOUNDER PROFILE

DATE AUGUST 1, 1990 | PLATE No. 41

# CORROSION VELOCITY IN MM PER YEAR $\times 10^{-3}$



GEONOR CORROSION SOUNDER  
AS IN NGI PUBLICATION No. 42  
T. ROSENQUIST (1961)

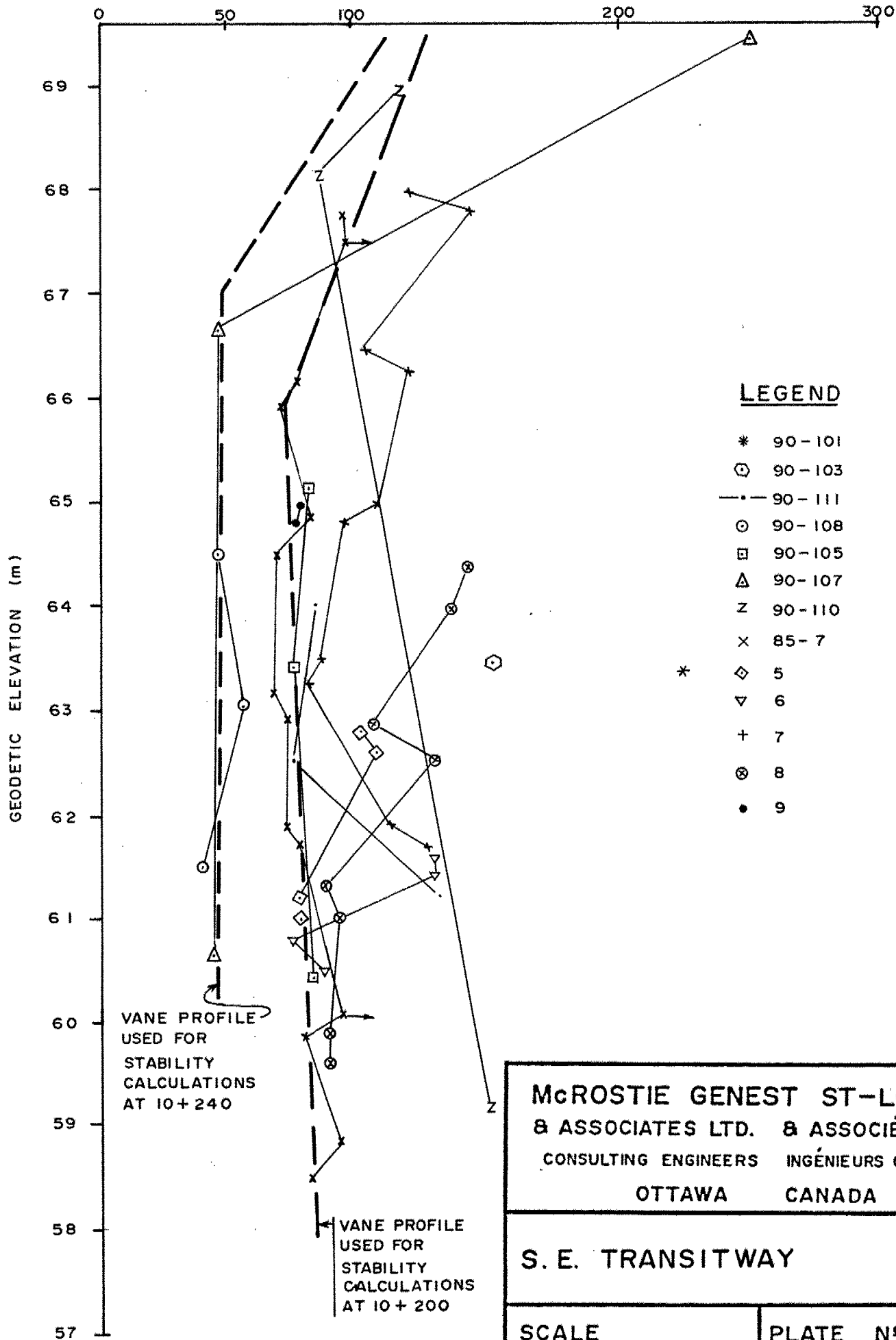
BOREHOLE 90 - 105

McROSTIE GENEST ST-LOUIS  
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CONSULTING ENGINEERS INGENIEUR CONSEILS  
OTTAWA, CANADA

S.E. TRANSITWAY - BANK ST. TO C.N.R.  
CORROSION SOUNDER PROFILE

DATE AUGUST 1, 1990 | PLATE No. 42

# VANE SHEAR STRENGTH (kPa)



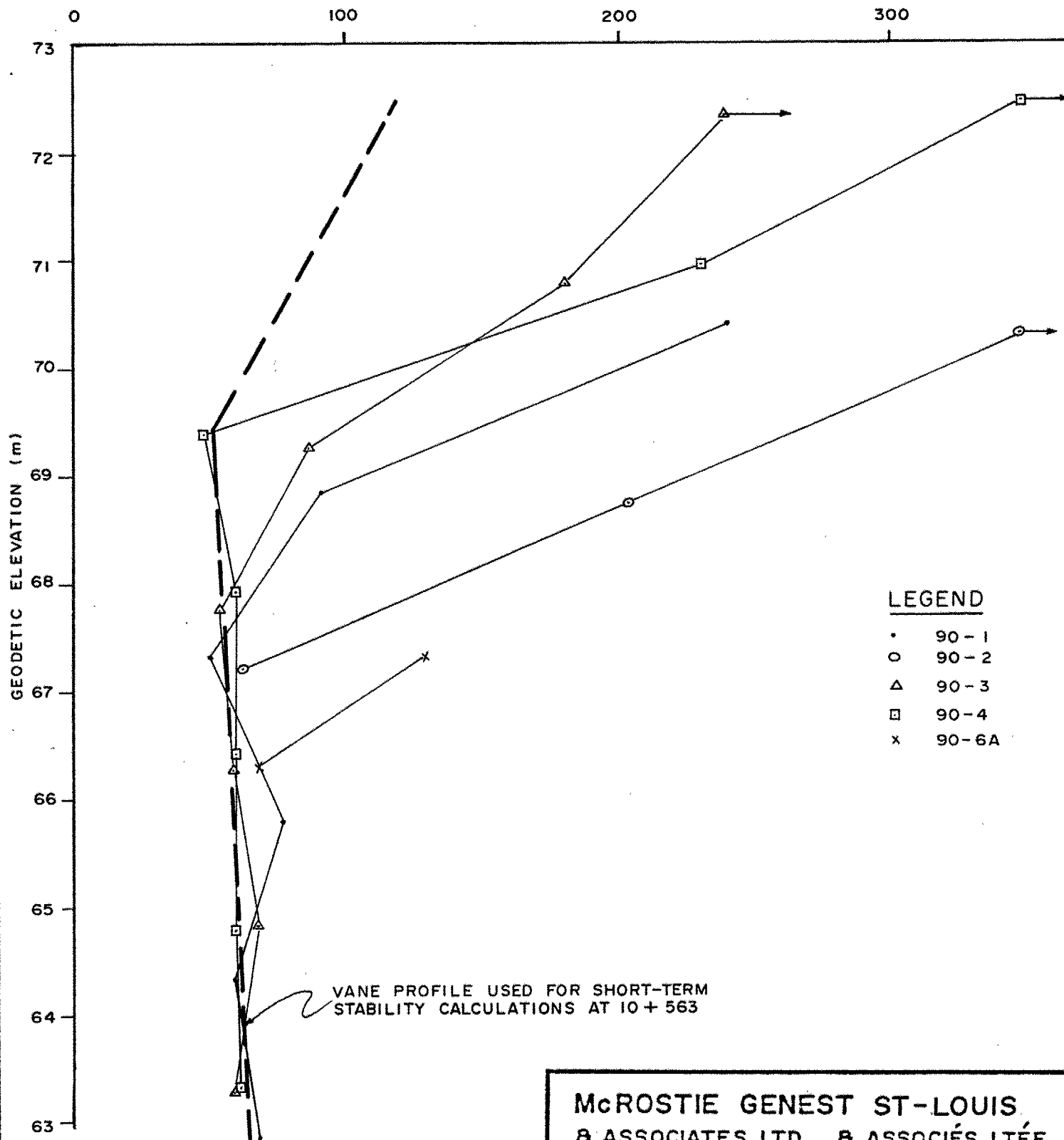
**McROSTIE GENEST ST-LOUIS**  
**& ASSOCIATES LTD. & ASSOCIÉS LTÉE**  
 CONSULTING ENGINEERS INGÉNIEURS CONSEILS  
 OTTAWA CANADA

**S. E. TRANSITWAY**

**SCALE**

**PLATE N° 43**

# VANE SHEAR STRENGTH (kPa)



## LEGEND

- 90-1
- 90-2
- △ 90-3
- 90-4
- x 90-6A

McROSTIE GENEST ST-LOUIS  
 & ASSOCIATES LTD. & ASSOCIÉS LTÉE  
 CONSULTING ENGINEERS INGÉNIEURS CONSEILS  
 OTTAWA CANADA

S. E. TRANSITWAY

SCALE

PLATE N° 44

Mc ROSTIE GENEST ST-LOUIS

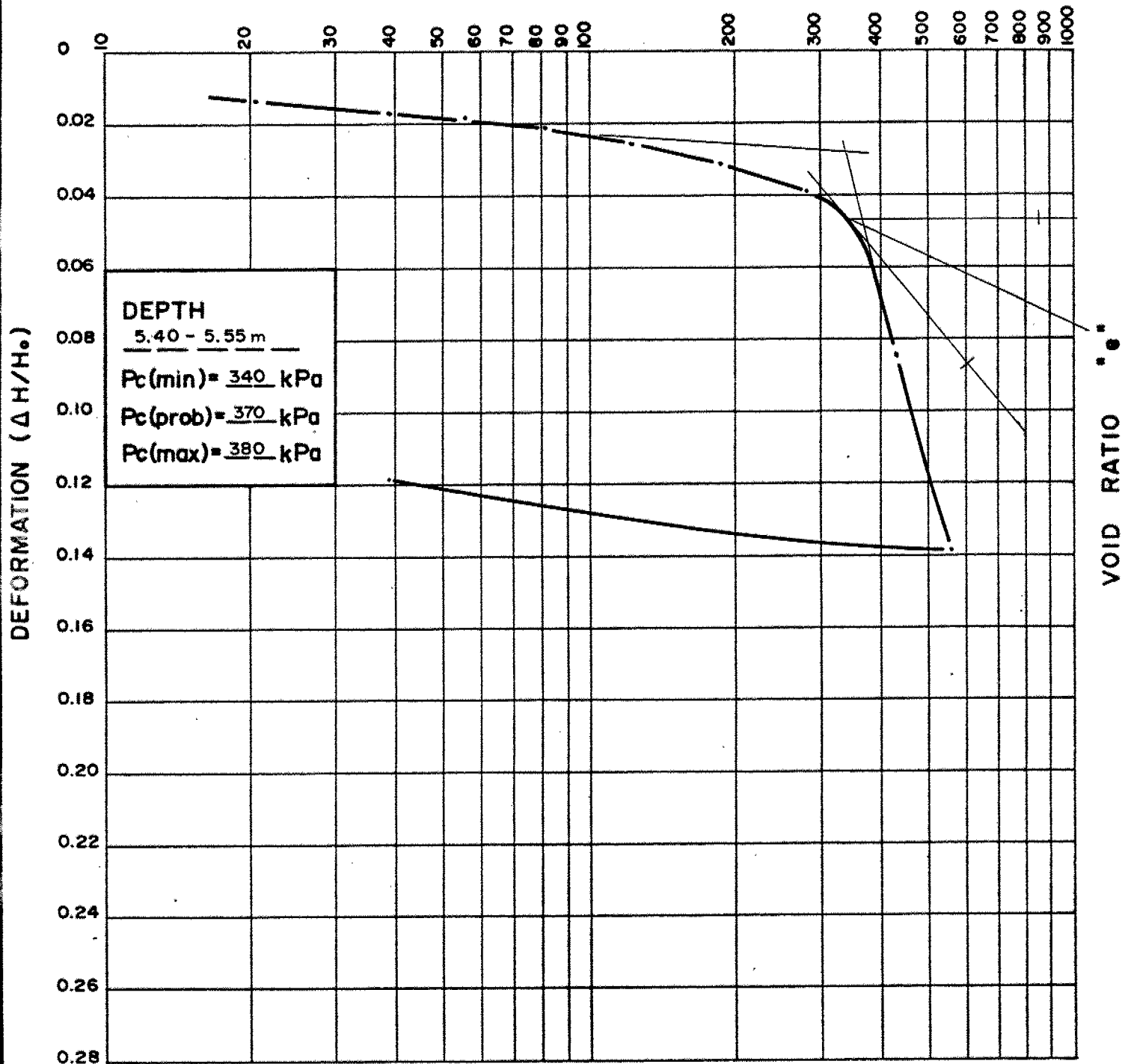
& ASSOCIATES LTD. & ASSOCIÉS LTÉE

CONSULTING ENGINEERS INGÉNIEURS CONSEILS

OTTAWA CANADA

## CONSOLIDATION TEST

EFFECTIVE STRESS (kPa)



APPARATUS N° 1 DESCRIPTION: \_\_\_\_\_

PROJECT: S.E. TRANSITWAY

PROJECT N° E-6329 SAMPLE: 90-108-6 PLATE No. 45

Mc ROSTIE GENEST ST-LOUIS

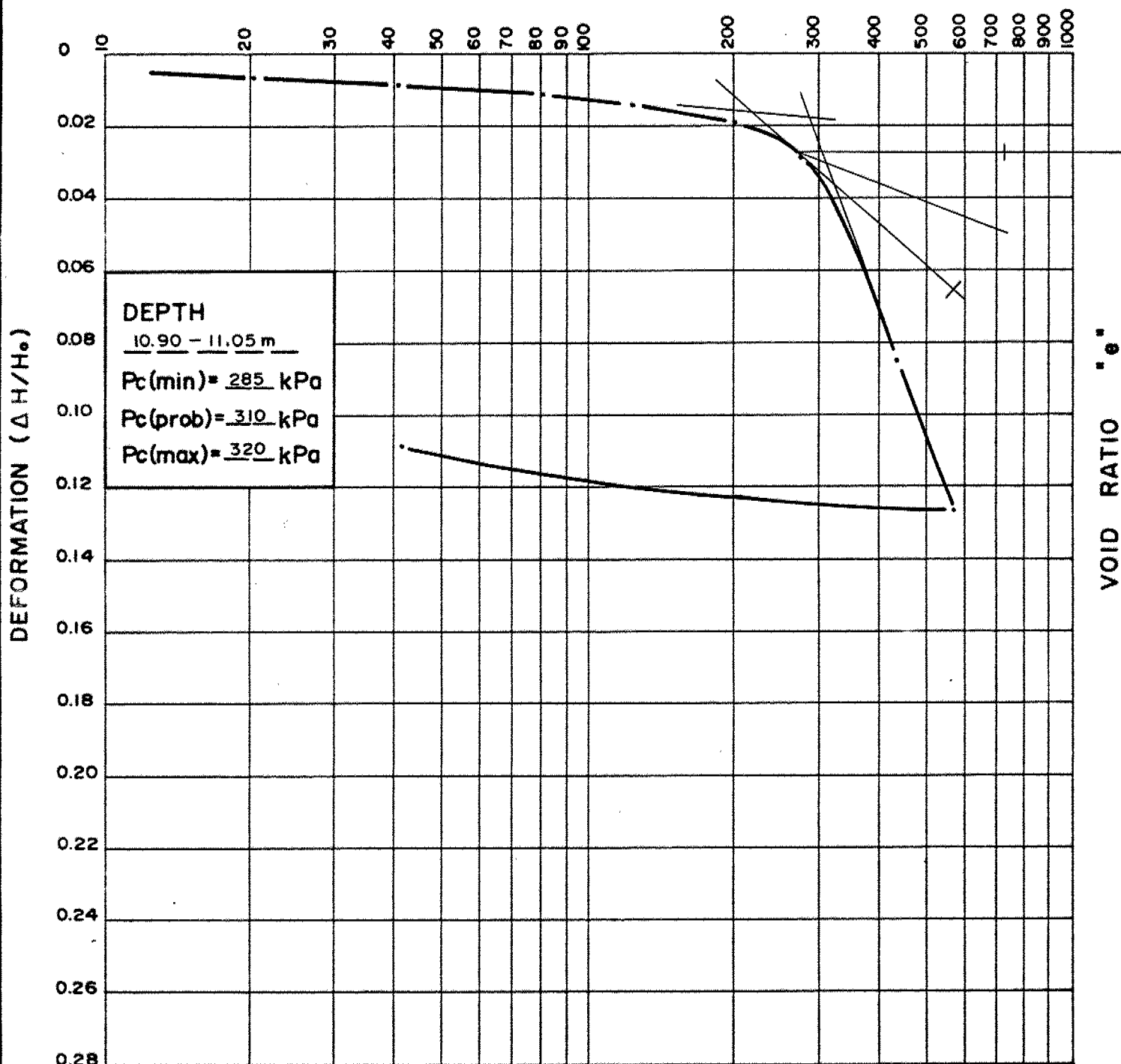
& ASSOCIATES LTD. & ASSOCIÉS LTÉE

CONSULTING ENGINEERS INGÉNIEURS CONSEILS

OTTAWA CANADA

## CONSOLIDATION TEST

EFFECTIVE STRESS (kPa)



APPARATUS N° 2 DESCRIPTION:

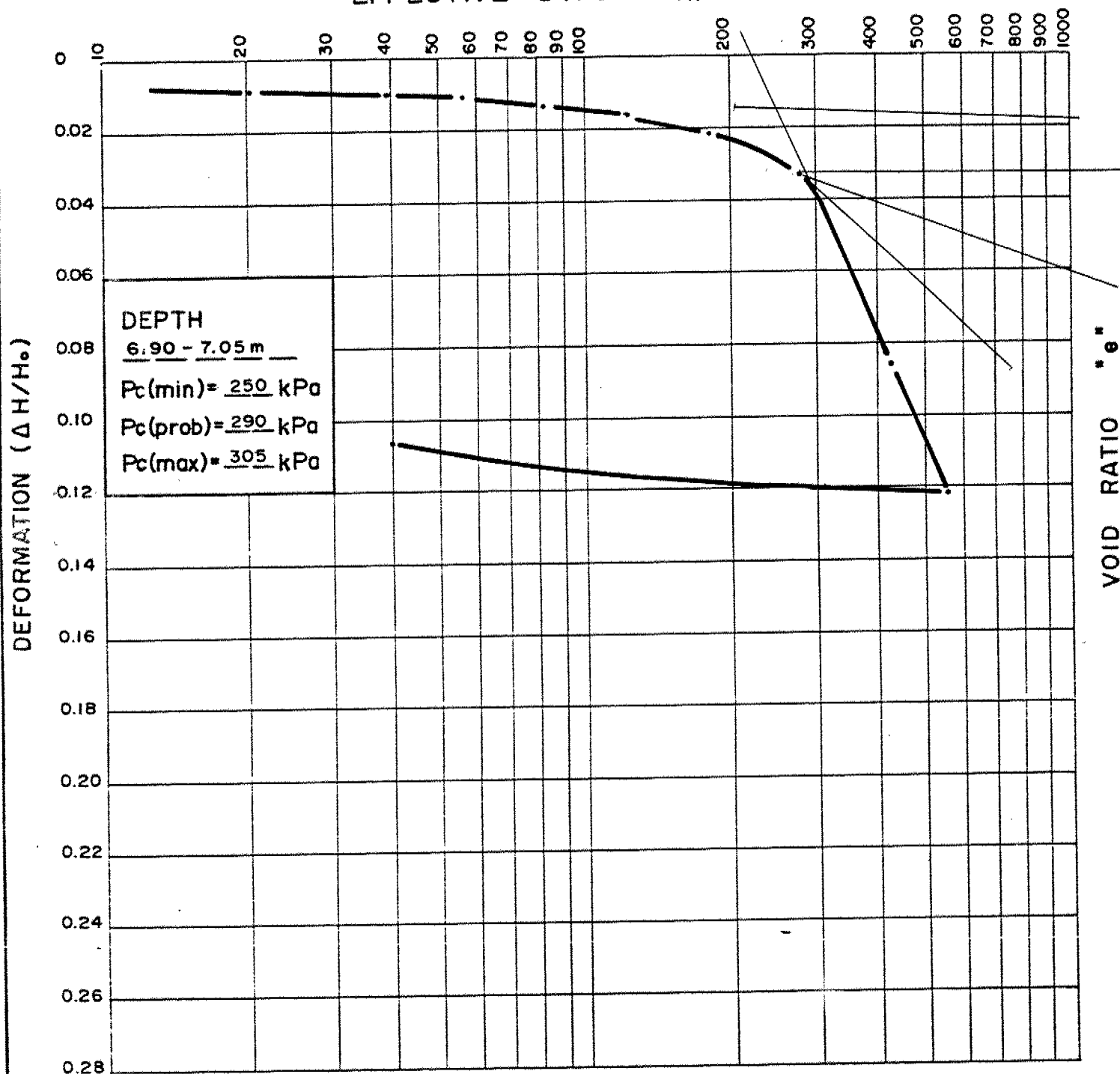
PROJECT: S.E. TRANSITWAY

PROJECT N° E-6329 SAMPLE: 90-110-9 PLATE No. 46



Mc ROSTIE GENEST ST-LOUIS  
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OTTAWA CANADA

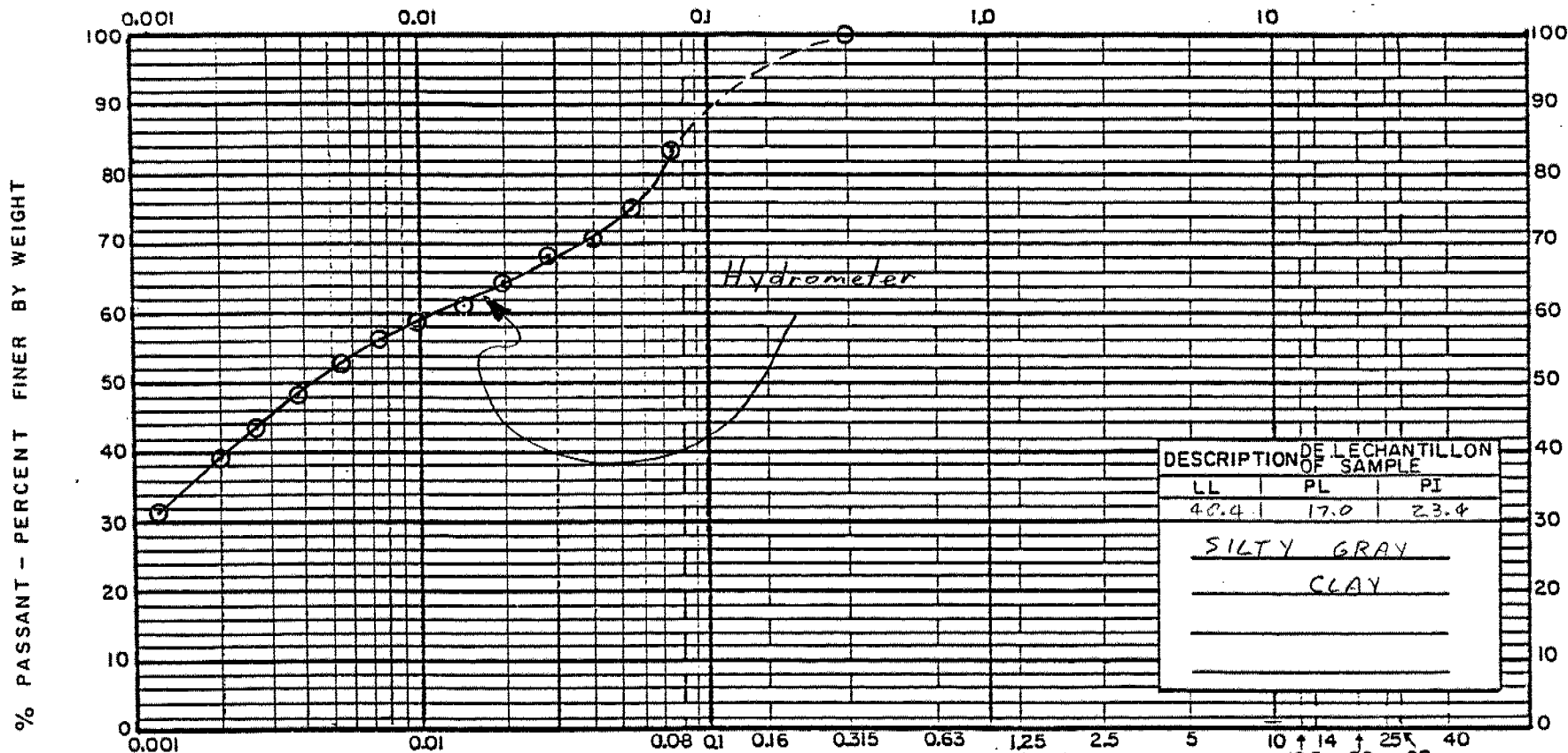
CONSOLIDATION TEST  
EFFECTIVE STRESS (kPa)



APPARATUS N° 1 DESCRIPTION: \_\_\_\_\_  
PROJECT: S.E. TRANSITWAY  
PROJECT N° E 6329 SAMPLE: 90-110-7 PLATE No. 47

SYSTÈME UNIFIÉ DE CLASSIFICATION  
ANALYSE GRANULOMÉTRIQUE DES SOLS  
No. DU TAMIS C.G.S.B. STANDARD

UNIFIED SOIL CLASSIFICATION  
MECHANICAL ANALYSIS OF SOILS  
C.G.S.B. STANDARD SIEVE SIZE



DESCRIPTION DE L'ÉCHANTILLON OF SAMPLE		
LL	PL	PI
40.4	17.0	23.4
SILTY GRAY		
CLAY		

39 % ARGILE CLAY	OU OR	SILT SILT	47 %
---------------------	----------	--------------	------

CRITÈRE - CRITERIA		
TYPE DE SOL SOIL TYPE	Cu	Cc
GW	> 4	1-3
SW	> 6	1-3

SABLE - SAND			GRAVIER - GRAVEL	
FIN - FINE	MOYEN-MEDIUM	GROS-COARSE	FIN - FINE	GROS-COARSE
14 %				

PLAQUE No. - PLATE No. 48

PROJET  
PROJECT S.E. Transitway

DESSINE  
PLOTTED C.N.M. DATE 05-08-90  
VERIFIÉ  
CHECKED P.R.T. DATE 07-08-90

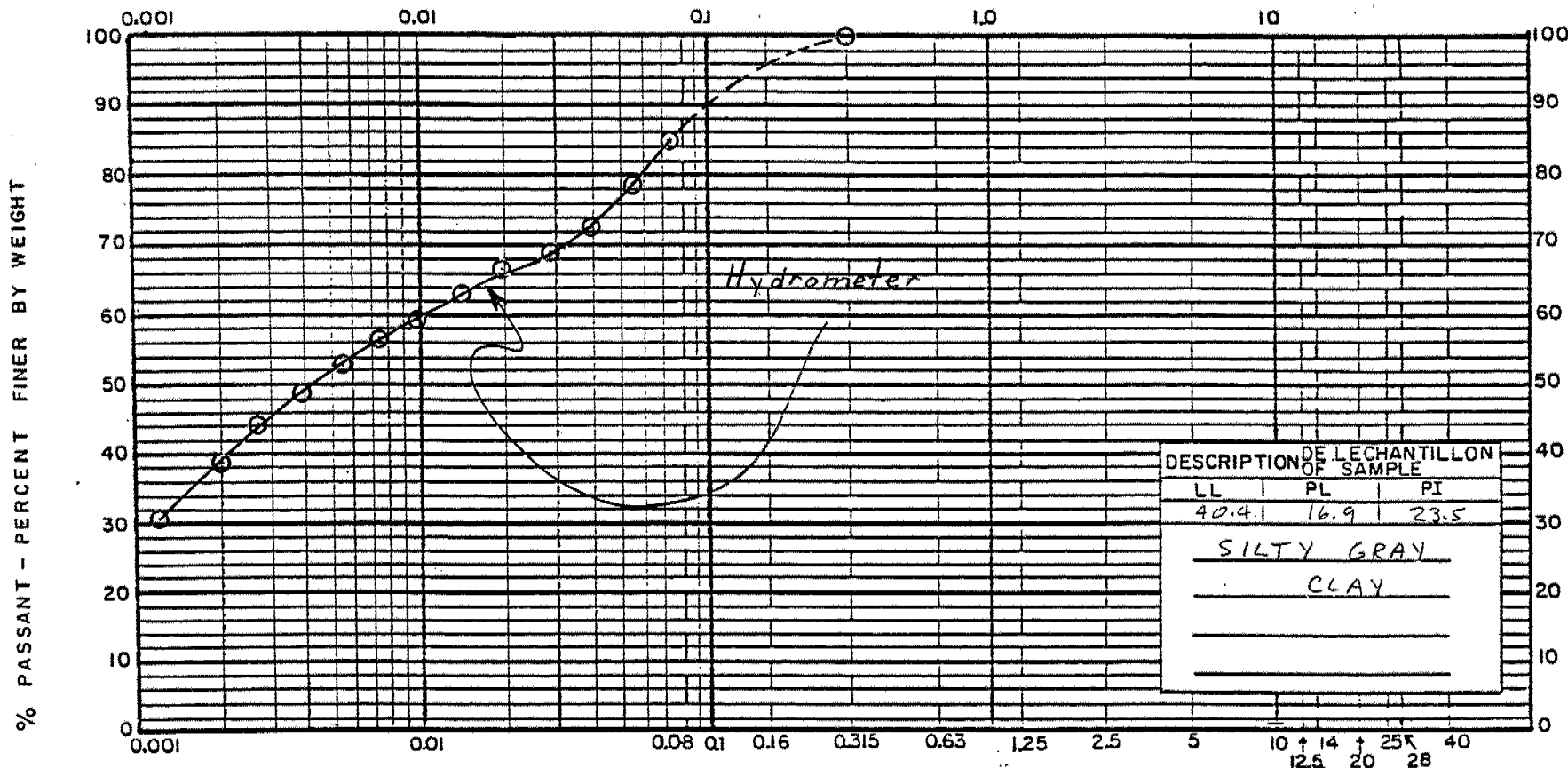
ÉCHANTILLON No.  
SAMPLE No. 90-108-6

REMARQUES  
REMARKS depth 5.25-5.40m

MCROSTIE GENEST ST-LOUIS  
& ASSOCIATES LTD. - & ASSOCIÉS LTÉE  
CONSULTING ENGINEERS - INGÉNIEURS CONSEILS  
OTTAWA, CANADA

SYSTÈME UNIFIÉ DE CLASSIFICATION  
ANALYSE GRANULOMÉTRIQUE DES SOLS  
No. DU TAMIS C.G.S.B. STANDARD

UNIFIED SOIL CLASSIFICATION  
MECHANICAL ANALYSIS OF SOILS  
C.G.S.B. STANDARD SIEVE SIZE



39 % ARGILE CLAY	OU OR	SILT SILT	48 %	SABLE - SAND	GRAVIER - GRAVEL
				FIN - FINE	FIN - FINE
				MOYEN - MEDIUM	GROS - COARSE
				GROS - COARSE	GROS - COARSE
				13 %	

CRITERE - CRITERIA		
TYPE DE SOL SOIL TYPE	Cu	Cc
GW	> 4	1-3
SW	> 6	1-3

PLAQUE No. - PLATE No. 49

PROJET  
PROJECT S.E. Transitway

ECHANTILLON No.  
SAMPLE No. 90-110-9

DESSINE  
PLOTTED C.N.M. DATE 05-08-90  
VERIFIÉ  
CHECKED R.P.T. DATE 07-08-90

REMARQUES  
REMARKS depth: 11.05-11.20 m

MCROSTIE GENEST ST-LOUIS  
& ASSOCIATES LTD. - & ASSOCIÉS LTÉE  
CONSULTING ENGINEERS - INGÉNIEURS CONSEILS  
OTTAWA, CANADA

# WATER ANALYSIS

## ANALYSE DE L'EAU

JOB No:

TRAVAIL No: E-6329

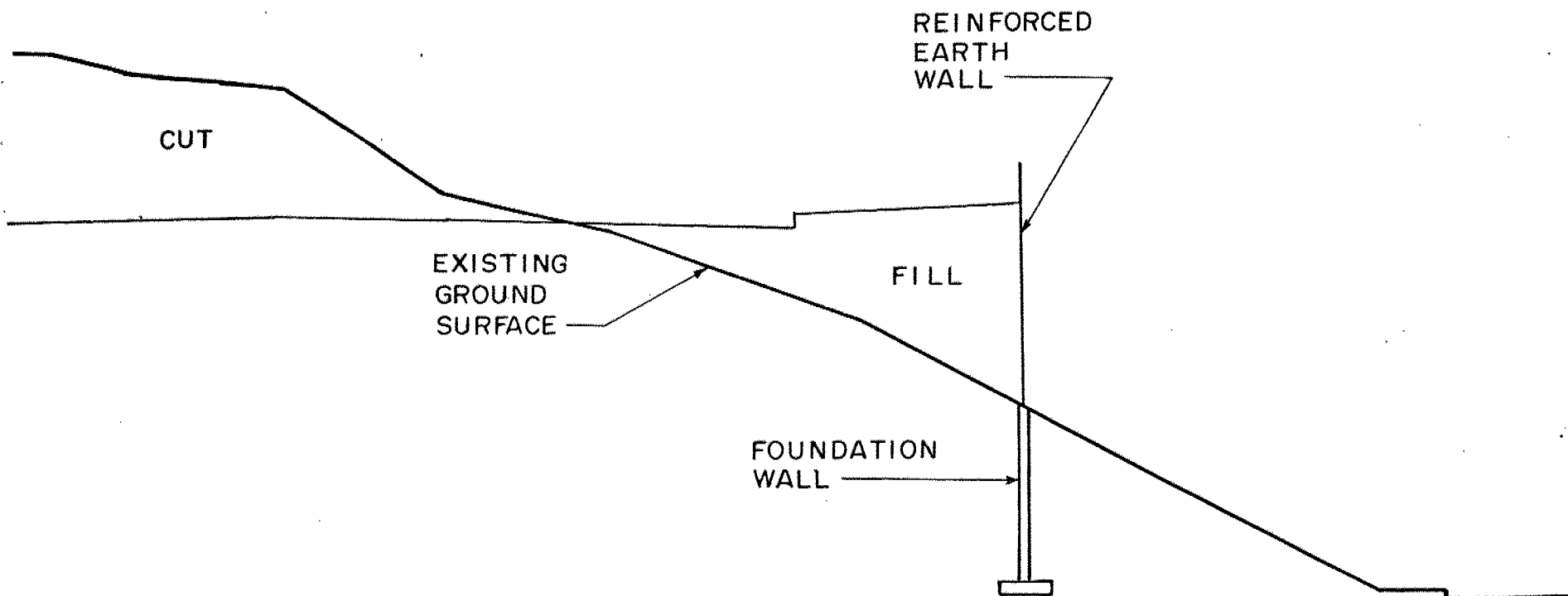
JOB NAME:

NOM DU TRAVAIL: South East Transitway

BOREHOLE No. FORAGE No.	WATER LEVEL NIVEAU D'EAU	DATE SAMPLED DATE DE L'ÉCHANTILLONNAGE	DATE TESTED DATE DE L'ANALYSE	CONDUCTIVITY MICROMHOS/CM	CONDUCTIVITÉ MICROMHOS/CM	SOLUBLE SULPHATE CONTENT (SO <sub>4</sub> ) P.P.M. TENEUR EN SULFATE SOLUBLE (SO <sub>4</sub> ) P.P.M.	SOLUBLE CHLORIDES CONTENT (CL) P.P.M. TENEUR EN CHLORURE SOLUBLE (CL) P.P.M.	PH PH
90-105	10.00	July 12 1990	July 23, 1990	260		45±5	50±-5	7.5
90-111	3.70	July 16 1990	July 23, 1990	1,200		80±8	90±9	7.7
90-108	11.00	July 16 1990	July 23, 1990	280		70±7	60±6	7.8
90-101	8.80	July 16 1990	July 23, 1990	700		90±9	60±6	7.6
90-106	7.60	July 17, 1990	July 23, 1990	440		55±5	15±1	7.9
90-107	6.65	July 17, 1990	July 23, 1990	400		45±5	22±2	7.4

PLATE No. 50

**McROSTIE GENEST ST-LOUIS**  
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 OTTAWA, CANADA



STA. 10 + 100

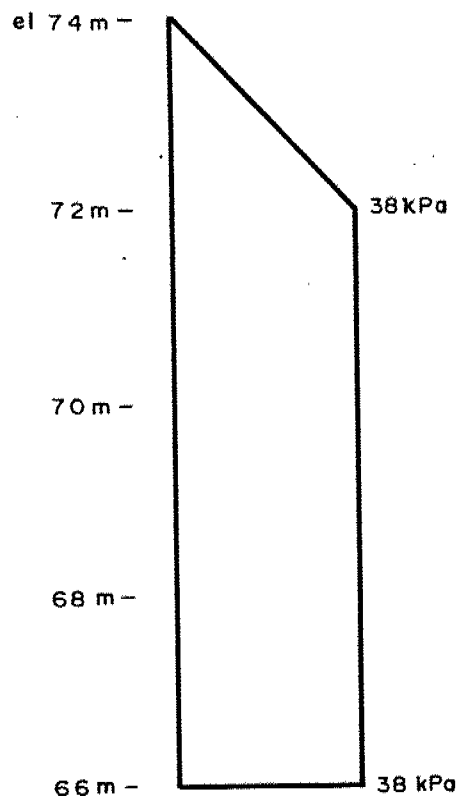
**McROSTIE GENEST ST-LOUIS**  
**& ASSOCIATES LTD. & ASSOCIÉS LTÉE**  
 CONSULTING ENGINEERS INGÉNIEURS CONSEILS  
 OTTAWA CANADA

**WALL / SLOPE SYSTEM**

**SCALE 1:100**

**PLATE N° 51**

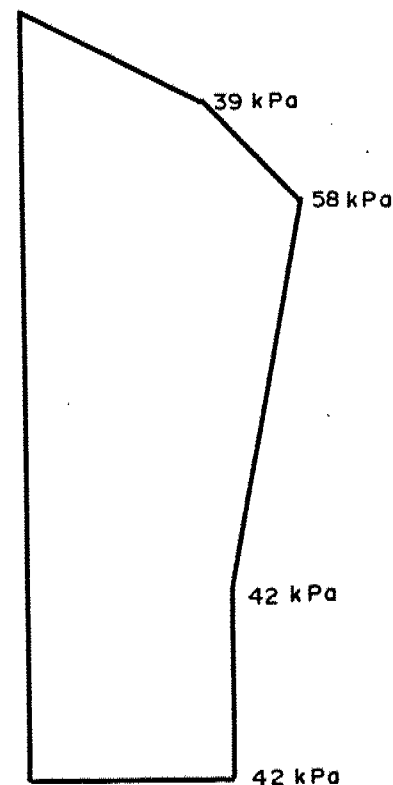
EARTH PRESSURE



E-85 RAILWAY LOADING



TOTAL



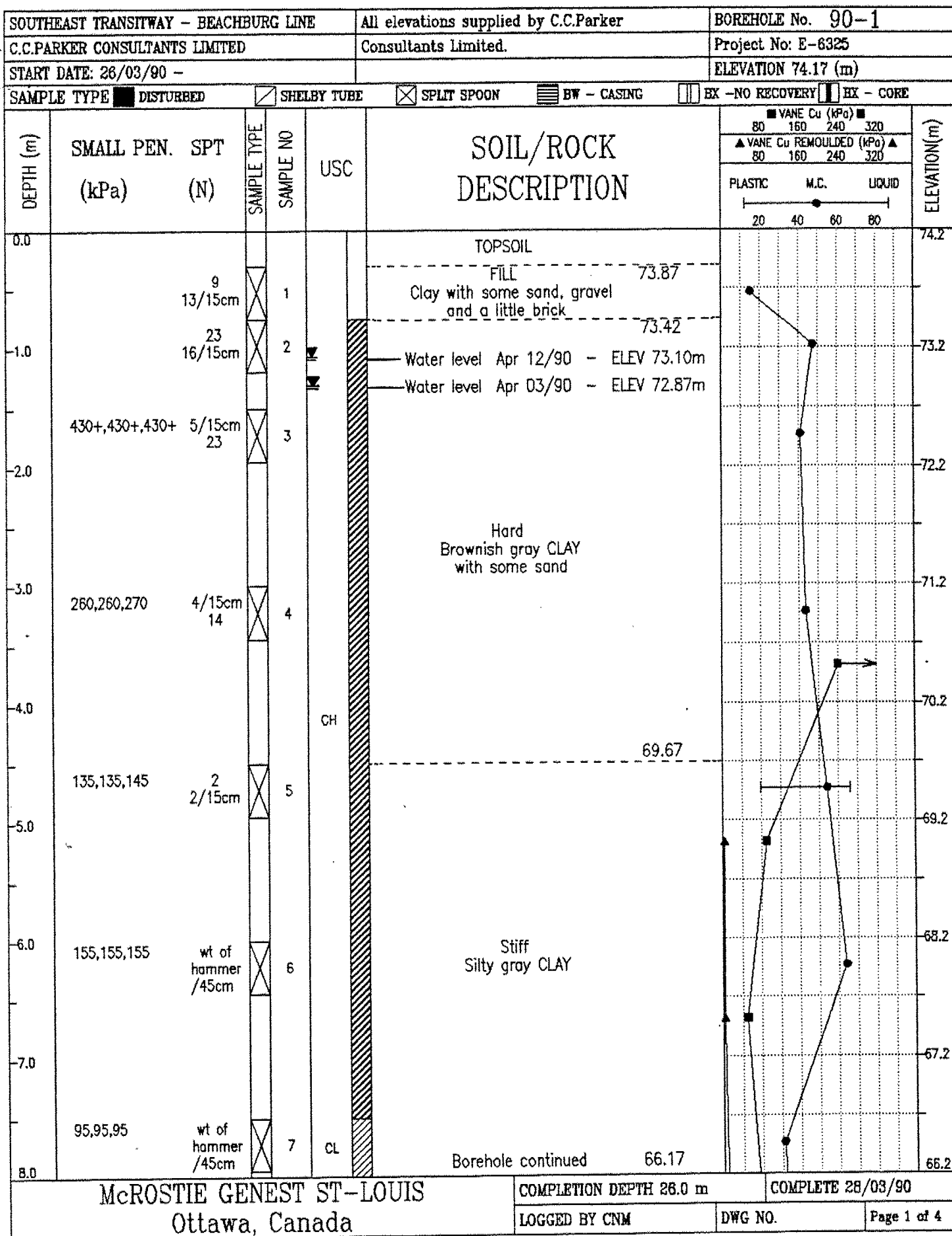
**McROSTIE GENEST ST-LOUIS**  
**& ASSOCIATES LTD. & ASSOCIÉS LTÉE**  
CONSULTING ENGINEERS INGÉNIEURS CONSEILS  
OTTAWA CANADA

TEMPORARY SHORING  
S.E. TRANSITWAY - STATION 10+250

SCALE AS SHOWN

PLATE N° 52

Test holes from previous studies.





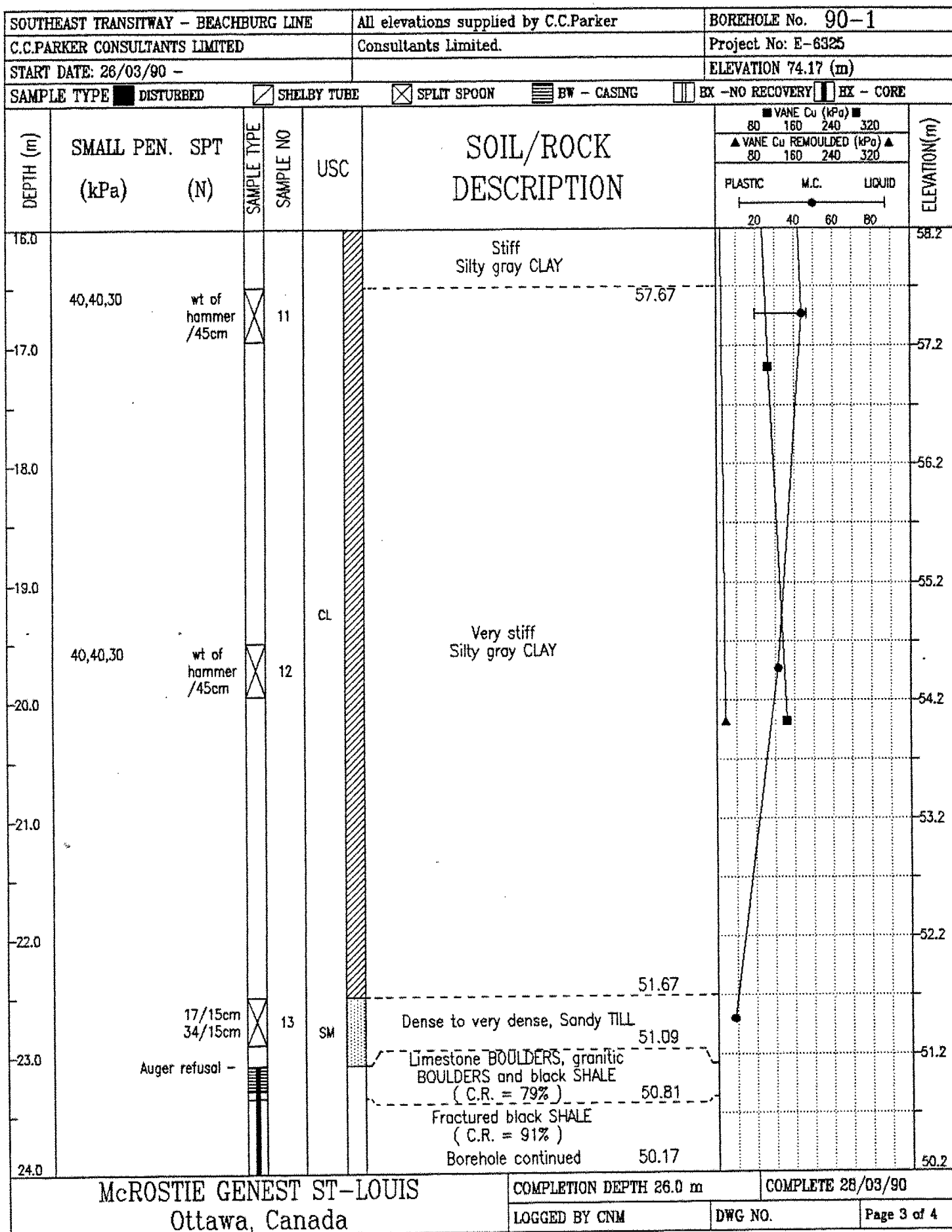
SOUTHEAST TRANSITWAY - BEACHBURG LINE				All elevations supplied by C.C.Parker		BOREHOLE No. 90-1	
C.C.PARKER CONSULTANTS LIMITED				Consultants Limited.		Project No: E-6325	
START DATE: 26/03/90 -				ELEVATION 74.17 (m)			
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON		<input type="checkbox"/> HW - CASING	
				<input type="checkbox"/> EX - NO RECOVERY		<input type="checkbox"/> EX - CORE	

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)			VANE Cu REMOULDED (kPa)			ELEVATION (m)		
							80	160	240	320	80	160		240	320
							PLASTIC			M.C.				LIQUID	
8.0													66.2		
9.0	65,60,60												65.2		
		wt of hammer /45cm		8											
10.0													64.2		
	10,10,10												63.2		
		wt of hammer /45cm		9											
11.0													62.2		
													61.2		
12.0													60.2		
													59.2		
13.0													58.2		
14.0															
	10,10,10														
		wt of hammer /45cm		10											
15.0															
16.0															

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 26.0 m		COMPLETE 28/03/90	
		LOGGED BY CNM	DWG NO.	Page 2 of 4	



SOUTHEAST TRANSITWAY - BEACHBURG LINE			All elevations supplied by C.C.Parker			BOREHOLE No. 90-1		
C.C.PARKER CONSULTANTS LIMITED			Consultants Limited.			Project No: E-6325		
START DATE: 26/03/90 -						ELEVATION 74.17 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input type="checkbox"/> BW - CASING			<input type="checkbox"/> EX - NO RECOVERY		
						<input type="checkbox"/> EX - CORE		

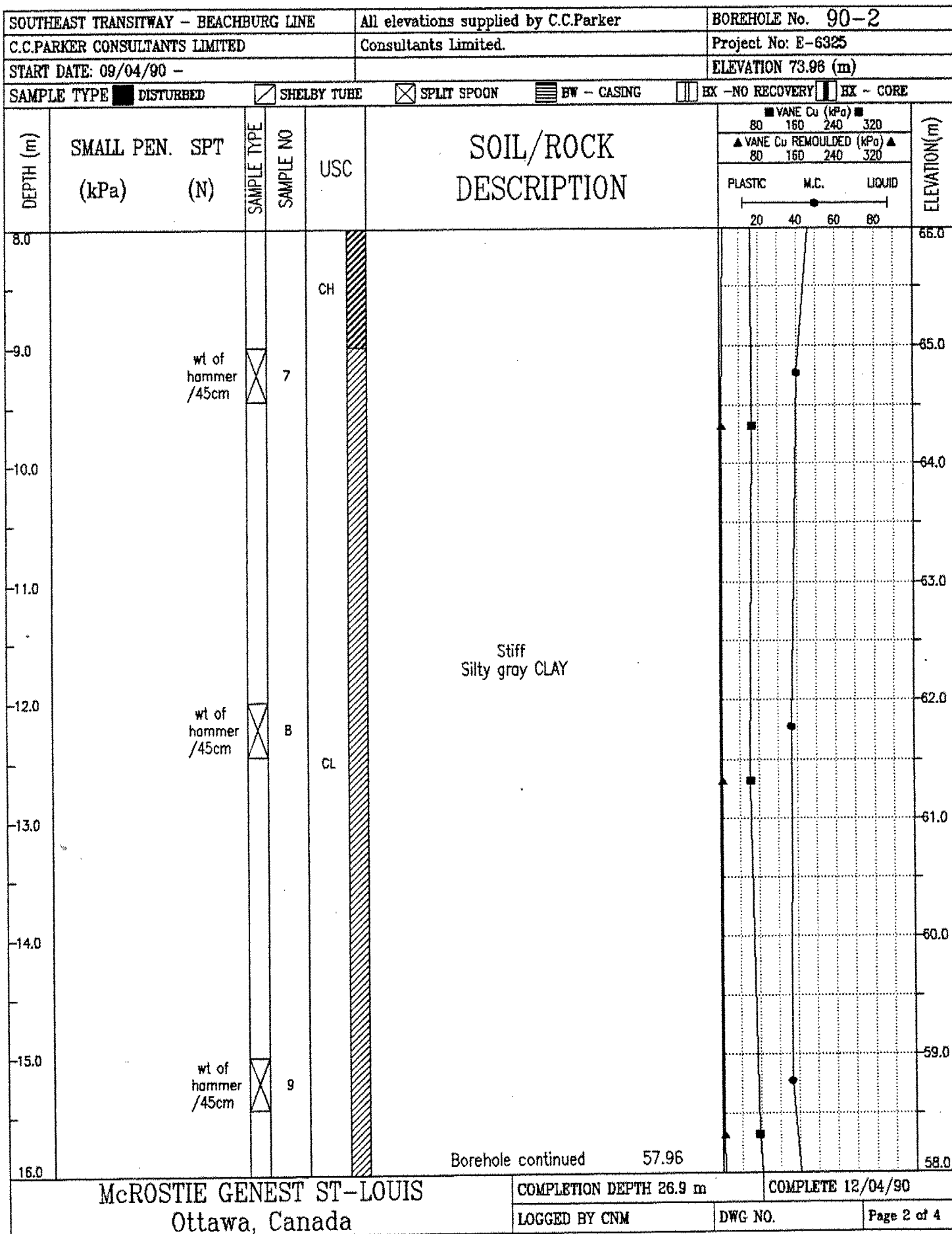
  

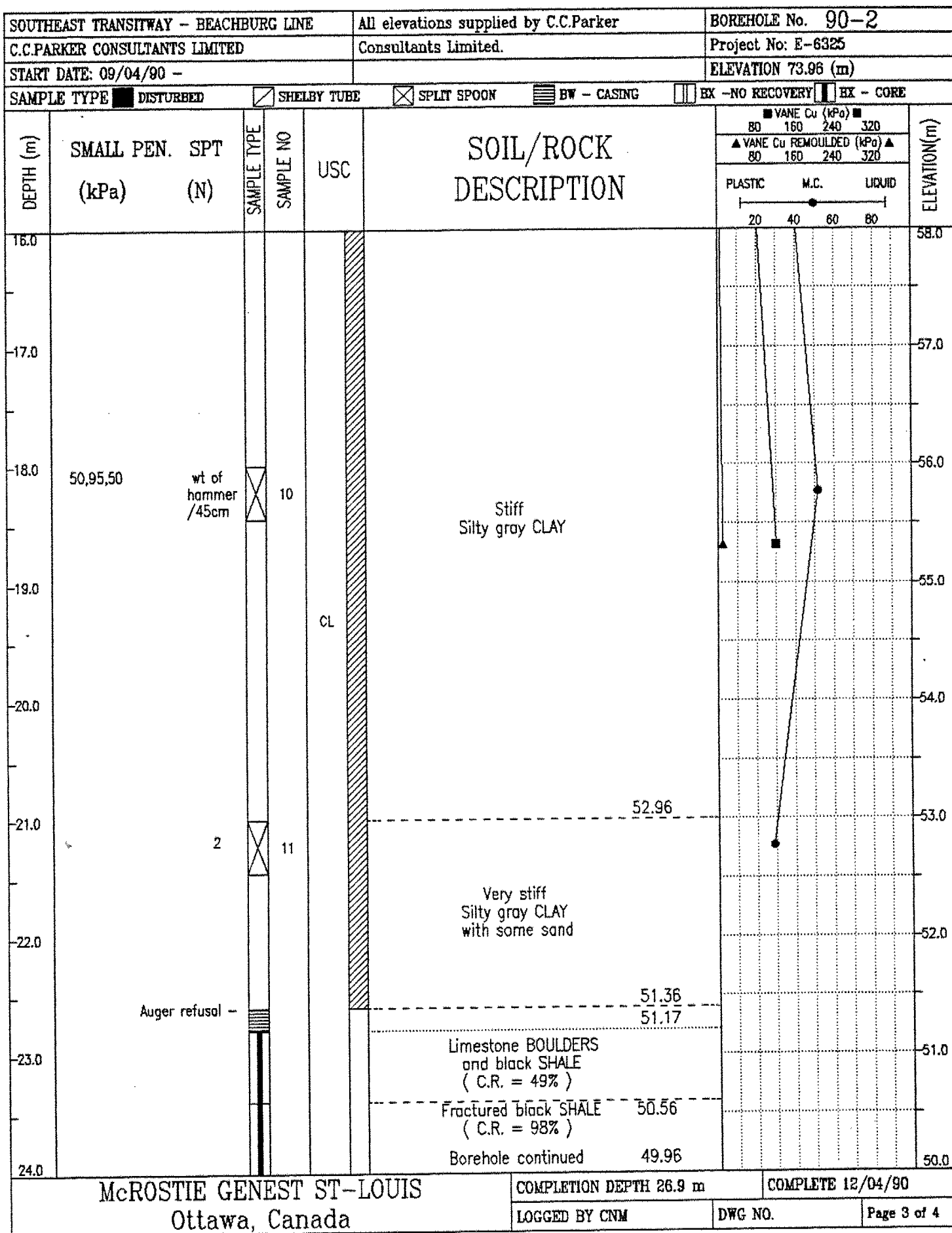
DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)			VANE Cu REMOULDED (kPa)			ELEVATION (m)		
							80	160	240	320	80	160		240	320
							PLASTIC			M.C.				LIQUID	
							20	40	60	80					
24.0						Fractured black SHALE ( C.R. = 91% )							50.2		
25.0						Fractured black SHALE ( C.R. = 98% )							49.2		
26.0						Bottom of hole							48.2		
27.0						C.R. = Core Recovery							47.2		
28.0													46.2		
29.0													45.2		
30.0													44.2		
31.0													43.2		
32.0													42.2		

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 26.0 m		COMPLETE 26/03/90	
		LOGGED BY CNM	DWG NO.	Page 4 of 4	







SOUTHEAST TRANSITWAY - BEACHBURG LINE		All elevations supplied by C.C.Parker		BOREHOLE No. 90-2	
C.C.PARKER CONSULTANTS LIMITED		Consultants Limited.		Project No: E-6325	
START DATE: 09/04/90 -		ELEVATION 73.96 (m)			
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED <input checked="" type="checkbox"/> SHELBY TUBE <input checked="" type="checkbox"/> SPLIT SPOON <input checked="" type="checkbox"/> BW - CASING <input type="checkbox"/> EX - NO RECOVERY <input type="checkbox"/> EX - CORE					

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)			ELEVATION(m)
							80	160	240	
							▲ VANE Cu REMOULDED (kPa) ▲			
							80	160	240	
							<div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>PLASTIC</span> <span>M.C.</span> <span>LIQUID</span> </div> <div style="text-align: center; margin-top: 5px;"> <span style="margin: 0 10px;">20</span> <span style="margin: 0 10px;">40</span> <span style="margin: 0 10px;">60</span> <span style="margin: 0 10px;">80</span> </div>			
24.0						Fractured black SHALE ( C.R. = 98% )				50.0
						49.91				
25.0						Fractured black SHALE ( C.R. = 76% )				49.0
						48.69				
26.0						Fractured and sound black SHALE ( C.R. = 100% )				48.0
27.0						Bottom of hole				47.0
						47.09				
28.0						C.R. = Core Recovery				46.0
29.0										45.0
30.0										44.0
31.0										43.0
32.0										42.0

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 26.9 m		COMPLETE 12/04/90	
		LOGGED BY CNM	DWG NO.	Page 4 of 4	





SOUTHEAST TRANSITWAY - BEACHBURG LINE				All elevations supplied by C.C.Parker		BOREHOLE No. 90-3	
C.C.PARKER CONSULTANTS LIMITED				Consultants Limited.		Project No: E-6325	
START DATE: 29/03/90 -						ELEVATION 74.64 (m)	
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED <input checked="" type="checkbox"/> SHELBY TUBE <input checked="" type="checkbox"/> SPLIT SPOON <input checked="" type="checkbox"/> BW - CASING <input type="checkbox"/> EX - NO RECOVERY <input type="checkbox"/> EX - CORE							

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	<div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>■ VANE Cu (kPa) ■</span> <span>80 160 240 320</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>▲ VANE Cu REMOULDED (kPa) ▲</span> <span>80 160 240 320</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>PLASTIC</span> <span>M.C.</span> <span>LIQUID</span> </div> <div style="text-align: center; font-size: 0.8em;"> 20      40      60      80 </div>	ELEVATION (m)
8.0								65.6
9.0								65.6
		wt of hammer /45cm	X	B				
10.0	10,10,10							64.6
		wt of hammer /45cm	X	9				
11.0								63.6
12.0						Stiff Silty gray CLAY		62.6
					CL			61.6
13.0								60.6
		wt of hammer /45cm	X	10				59.6
14.0								58.6
15.0								
16.0						Borehole continued	58.64	58.6

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 25.3 m		COMPLETE 02/04/90	
		LOGGED BY CNM		DWG NO.	

Page 2 of 4

SOUTHEAST TRANSITWAY - BEACHBURG LINE				All elevations supplied by C.C.Parker		BOREHOLE No. 90-3	
C.C.PARKER CONSULTANTS LIMITED				Consultants Limited.		Project No: E-6325	
START DATE: 29/03/90 -						ELEVATION 74.64 (m)	
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON		<input checked="" type="checkbox"/> BW - CASING	
						<input checked="" type="checkbox"/> EX - NO RECOVERY <input checked="" type="checkbox"/> EX - CORE	

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)		VANE Cu REMOULDED (kPa)		ELEVATION (m)
							80	160	80	160	
							<div style="display: flex; justify-content: space-between;"> <span>PLASTIC</span> <span>M.C.</span> <span>LIQUID</span> </div> <div style="text-align: center;"> <div style="width: 100px; border: 1px solid black; position: relative;"> <div style="position: absolute; left: 0; right: 0; height: 2px; background: linear-gradient(to right, black 40%, white 40%, white 60%, black 60%);"></div> </div> </div>				
							20	40	60	80	
16.0	10,10	wt of hammer /45cm	<input checked="" type="checkbox"/>	11							58.6
17.0			<input checked="" type="checkbox"/>								57.6
18.0											56.6
19.0											55.6
20.0	10,40,40	wt of hammer /45cm	<input checked="" type="checkbox"/>	12	CL	Stiff Silty gray CLAY					54.6
21.0											53.6
22.0											52.6
23.0						Granitic BOULDERS and limestone BOULDERS ( C.R. = 50% )					51.6
24.0						Borehole continued					50.6
											52.14

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 25.3 m		COMPLETE 02/04/90	
		LOGGED BY CNM	DWG NO.	Page 3 of 4	

SOUTHEAST TRANSITWAY - BEACHBURG LINE				All elevations supplied by C.C.Parker		BOREHOLE No. 90-3	
C.C.PARKER CONSULTANTS LIMITED				Consultants Limited.		Project No: E-6325	
START DATE: 29/03/90 -						ELEVATION 74.64 (m)	
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON		<input type="checkbox"/> BW - CASING	
				<input type="checkbox"/> BX - NO RECOVERY		<input type="checkbox"/> BX - CORE	

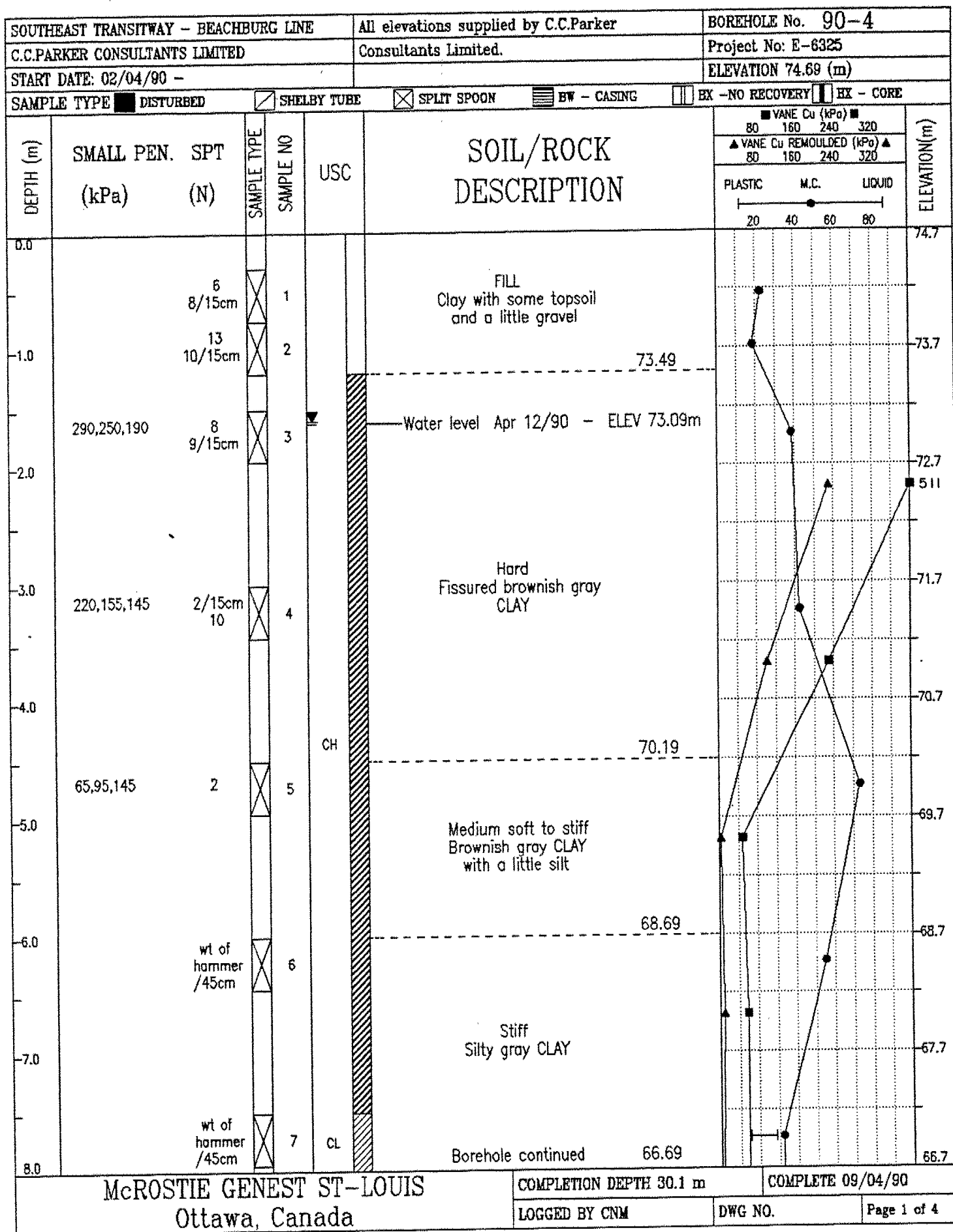
  

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	<div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>■ VANE Cu (kPa) ■</span> <span>80 160 240 320</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>▲ VANE Cu REMOULDED (kPa) ▲</span> <span>80 160 240 320</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>PLASTIC</span> <span>M.C.</span> <span>LIQUID</span> </div> <div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <span>20</span> <span>40</span> <span>60</span> <span>80</span> </div>				ELEVATION(m)							
							24.0							50.64				50.6
							25.0							Limestone BOULDERS, granitic BOULDERS and black SHALE ( C.R. = 24% )				49.6
														( C.R. = 0% ) Bottom of hole	49.44			49.34
26.0											48.6							
27.0											47.6							
28.0											46.6							
29.0											45.6							
30.0											44.6							
31.0											43.6							
32.0											42.6							

McROSTIE GENEST ST-LOUIS Ottawa, Canada	COMPLETION DEPTH 25.3 m	COMPLETE 02/04/90
	LOGGED BY CNM	DWG NO.

Page 4 of 4



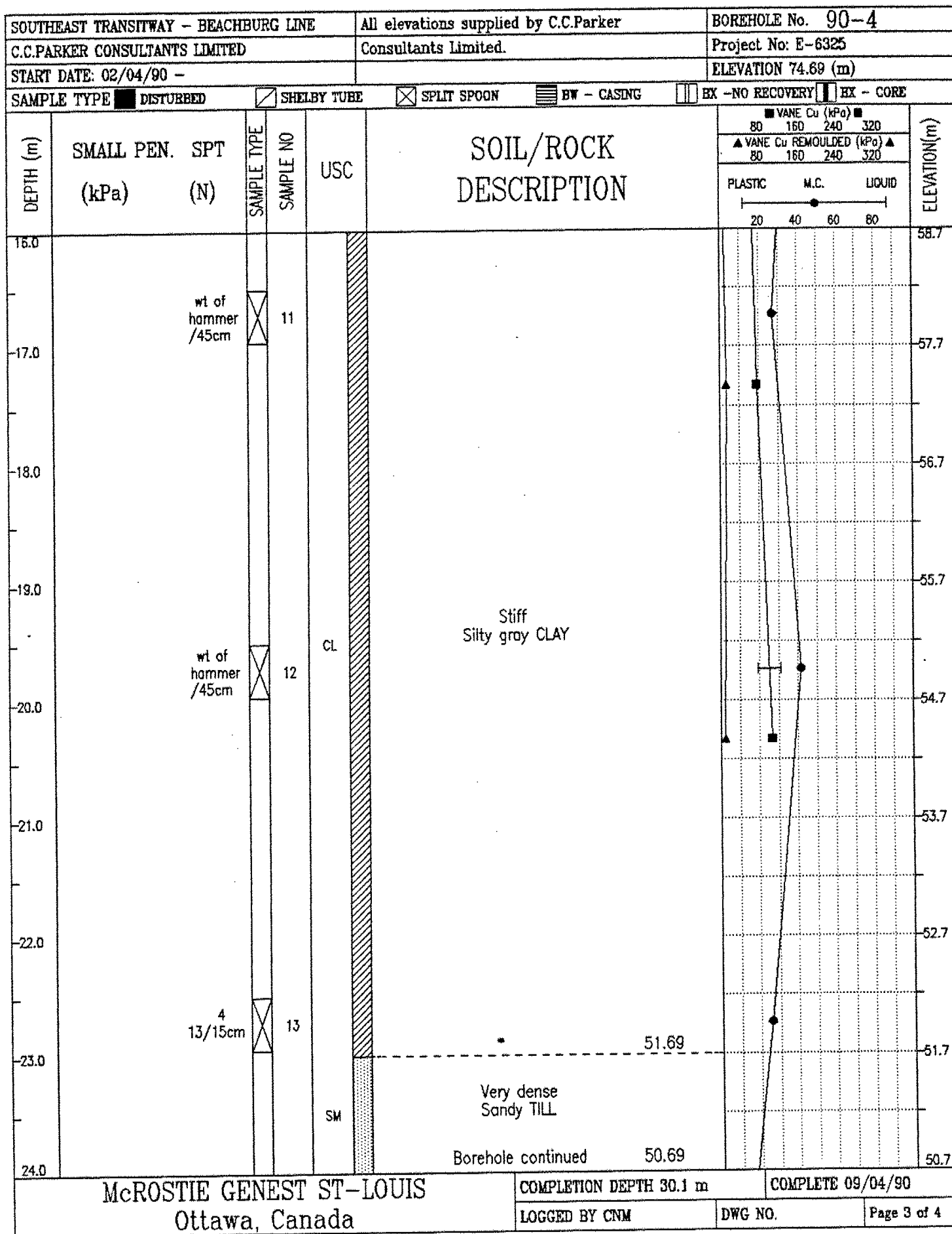
SOUTHEAST TRANSITWAY - BEACHBURG LINE				All elevations supplied by C.C.Parker		BOREHOLE No. 90-4	
C.C.PARKER CONSULTANTS LIMITED				Consultants Limited.		Project No: E-6325	
START DATE: 02/04/90 -				ELEVATION 74.69 (m)			
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED		<input checked="" type="checkbox"/> SHELBY TUBE		<input checked="" type="checkbox"/> SPLIT SPOON		<input type="checkbox"/> BW - CASING	
				<input type="checkbox"/> EX - NO RECOVERY		<input type="checkbox"/> EX - CORE	

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)			VANE Cu REMOULDED (kPa)			ELEVATION(m)		
							80	160	240	320	80	160		240	320
							PLASTIC			M.C.	LIQUID				
							20	40	60	80					
8.0													66.7		
9.0													65.7		
				8											
10.0													64.7		
				9											
11.0													63.7		
12.0													62.7		
13.0													61.7		
14.0													60.7		
				10											
15.0													59.7		
16.0													58.7		
						Borehole continued	58.69								

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 30.1 m		COMPLETE 09/04/90	
		LOGGED BY CNM	DWG NO.	Page 2 of 4	



SOUTHEAST TRANSITWAY - BEACHBURG LINE			All elevations supplied by C.C.Parker			BOREHOLE No. 90-4		
C.C.PARKER CONSULTANTS LIMITED			Consultants Limited.			Project No: E-6325		
START DATE: 02/04/90 -						ELEVATION 74.89 (m)		
SAMPLE TYPE <input checked="" type="checkbox"/> DISTURBED			<input checked="" type="checkbox"/> SHELBY TUBE			<input checked="" type="checkbox"/> SPLIT SPOON		
			<input checked="" type="checkbox"/> BW - CASING			<input type="checkbox"/> EX - NO RECOVERY		
						<input type="checkbox"/> EX - CORE		

DEPTH (m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	USC	SOIL/ROCK DESCRIPTION	VANE Cu (kPa)			ELEVATION (m)
							80	160	240	
							▲ VANE Cu REMOULDED (kPa) ▲			
							80	160	240	
							PLASTIC      M.C.      LIQUID 20      40      60      80			
24.0										50.7
25.0						Very dense Sandy TILL				49.7
	42/15cm 80		<input checked="" type="checkbox"/>	14	SM					
	Auger refusal -									
26.0										48.7
							48.24			
27.0						Sound and fractured black SHALE (C.R. = 90%)				47.7
							47.24			
28.0						Sound and fractured black SHALE (C.R. = 96%)				46.7
							45.74			
29.0						Sound and fractured black SHALE (C.R. = 99%)				45.7
										44.7
30.0						Bottom of hole	44.59			
										43.7
31.0										
						C.R. = Core Recovery				42.7
32.0										

McROSTIE GENEST ST-LOUIS Ottawa, Canada		COMPLETION DEPTH 30.1 m		COMPLETE 09/04/90	
		LOGGED BY CNM		DWG NO.	
				Page 4 of 4	

McRDSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-5

Date :

MARCH 8, 1990

SOUTHEAST TRANSITWAY  
CN BEACHEBURG

ELEV.	DEPTH in metres	DESCRIPTION	REMARKS
73.84			NO FROST
		FILL	
		clay with some brick	
73.24	0.60		
		clayey TOPSOIL	
73.04	0.80		
		fissured brownish gray CLAY	
72.84	-- 1 --		
72.64	1.2		HOLE ADVANCED BY HAND AUGER
		BOTTOM OF TEST PIT	
72.34	1.5		VANE STRENGTH U 268kPa R 32kPa
		hard	
71.84	-- 2 --		
		fissured	U 261kPa R 128kPa
		brownish gray	
71.34	2.5		U 255kPa R 128kPa
		CLAY	
70.84	-- 3 --		
70.69	3.15		U 230kPa R 121kPa
		Bottom of hole	
		NOTE; TEST PITS DUG WITH RUBBER TIRE BACKHOE CASE 580E	U Undisturbed R Remoulded
69.84	-- 4 --		
			Plate No.19



McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-6

Date :

MARCH 8, 1990

SOUTHEAST TRANSITWAY  
CN BEACHBURG

ELEV.	DEPTH in metres	DESCRIPTION	REMARKS
73.32			FROZEN TO 0.60m
		FILL	
		clay with some	
72.32	-- 1 --	gravel, brick,	
		and sand	
71.32	-- 2 --		
71.22	2.1	FILL	
		clayey topsoil	
70.92	2.4		
		FILL	
		brownish gray clay	
70.32	-- 3 --	with some brick,	
		weathered shale,	
		and wood	
69.32	-- 4 --		
		Bottom of pit	
	NOTE;	unable to dig any deeper with this equipment	Plate No20



McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
90-7

Date :

MARCH 8, 1990

SOUTHEAST TRANSITWAY  
CN BEACHBURG

ELEV.	DEPTH in metres	DESCRIPTION	REMARKS
72.53			FROZEN TO 0.60m
		FILL	
		clay, brick,	
71.53	-- 1 --	and pieces of	
		limestone slabs	
70.63	1.90		
70.53	-- 2 --	clayey TOPSOIL	
			HOLE ADVANCED BY HAND AUGER
70.33	2.2		
		BOTTOM OF PIT	
			VANE STRENGTH
70.03	2.5		
		hard	U 240kPa R 43kPa
69.53	-- 3 --		
		fissured	U 472kPa R 192kPa
69.03	3.5		
		brownish gray	U 313kPa R 128kPa
		CLAY	
68.53	-- 4 --		
			U 261kPa R 128kPa
68.38	4.15		
		Bottom of hole	Plate No.22

# Log of Borehole \_\_\_\_\_



Auger Sample ☒ Natural Moisture  
 SPT (N) Value ☐ Plastic and Liquid Limit  
 Dynamic Cone Test ☐ Undrained Triaxial at  
 Shelby Tube ☐ Overburden Pressure  
 Field Vane Test ☐ % Strain at Failure  
☐ Penetrometer



Project Bank Street Structures Dwg. No. 5

Bank Street, Ottawa, Ontario

Project No. R-00435A/GE

Hole location and datum see drawing No. 1

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH m	N Value				Natural Moisture Content and Aterberg Limits % Dry Weight			Natural Unit Weight kN/m <sup>3</sup>
					20	40	60	80	10	20	30	
		TOPSOIL: silty fine grained sand, medium organics, dark brown.	65.02 65.01	0						X		20.0
		FILL: silty fine grained sand, some clay and gravel, some shale fragments, inclusions of light brown silt, occasional organics, slightly plastic, dry, dark brown.		1						X		18.4
		with concrete fragments from 0.8m to 1.2m depths.	63.3	2						X		20.5
		with 25mm diameter gravel piece and 25mm diameter root from 1.5m to 1.7m depth.		3								
		AUGER REFUSAL		4								
		NOTES:		5								
		1. Borehole drilled with a truck mounted drill rig equipped with continuous flight hollow stem auger equipment to an auger refusal depth of 1.7m on December 28, 1989.		6								
		2. Upon completion of drilling, no cave, no water.		7								
				8								
				9								
				10								

NOTE: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS.

# Log of Borehole \_\_\_\_\_



Auger Sample

SPT (N) Value

Dynamic Cone Test

Shelby Tube

Field Vane Test



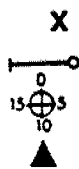
Natural Moisture

Plastic and Liquid Limit

Undrained Triaxial at Overburden Pressure

% Strain at Failure

Penetrometer



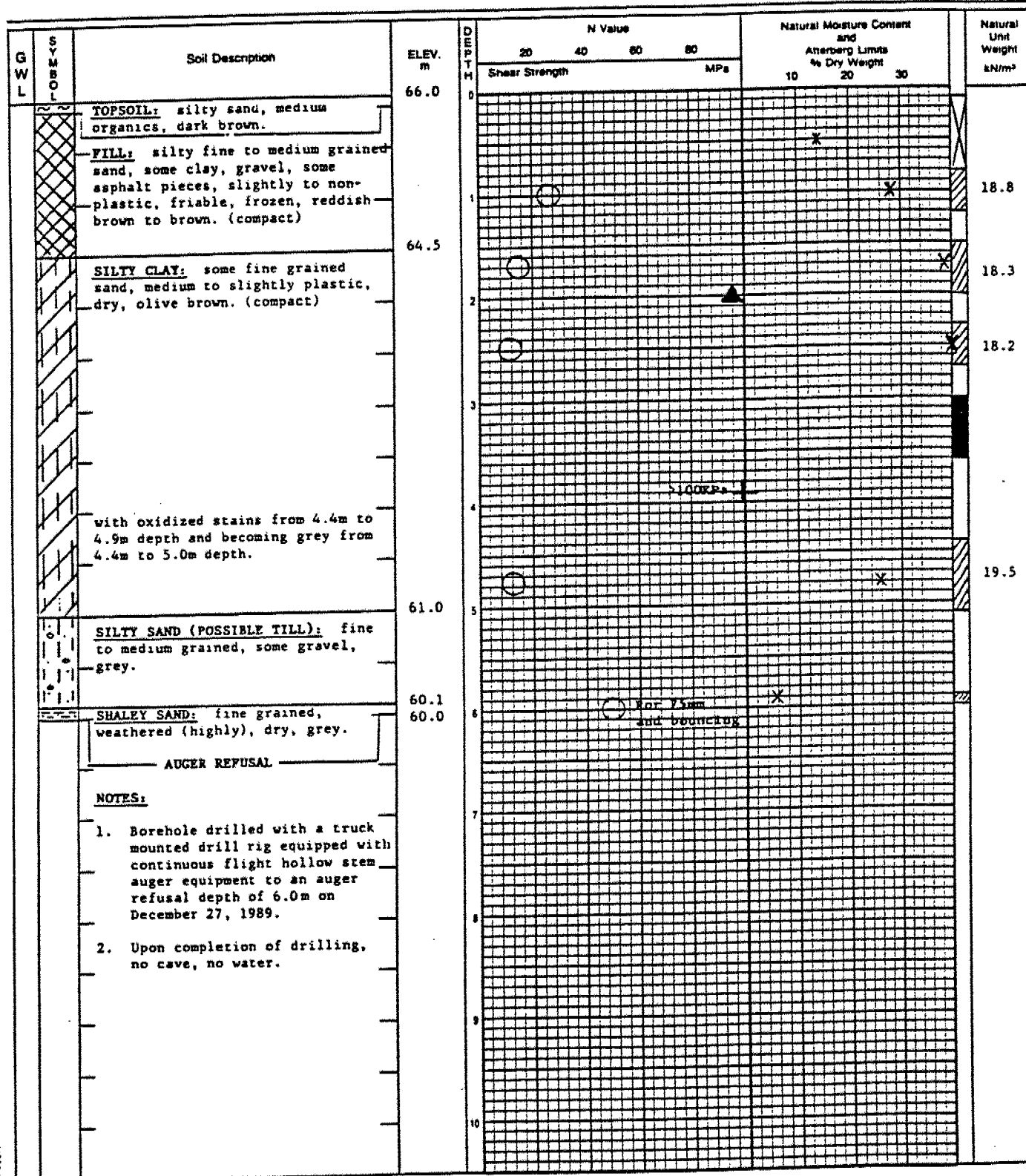
Project Bank Street Structures

Dwg. No. 6

Bank Street, Ottawa, Ontario

Project No. R-00435A/GF

Hole location and datum see drawing No. 1



G 009R

NOTE: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS.

# Log of Borehole 6 (DEC. 1989)



Auger Sample ☒ Natural Moisture  
 SPT (N) Value ☐ ☒ Plastic and Liquid Limit  
 Dynamic Cone Test ☐ Undrained Triaxial at  
 Shelby Tube ☐ Overburden Pressure  
 Field Vane Test ☐ % Strain at Failure  
☐ Penetrometer



Project Bank Street Structures Dwg. No. 6A

Bank Street, Ottawa, Ontario

Project No. R-00435A/GE

Hole location and datum see drawing No. 1

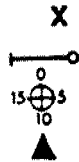
GWL	SYMBOL	Soil Description	ELEV. m	DEPTH m	N Value				Natural Moisture Content and Atterberg Limits % Dry Weight			Natural Unit Weight kN/m <sup>3</sup>
					20 40 60 80				10 20 30			
					Shear Strength MPa							
		<b>TOPSOIL:</b> silty sand, medium organics, dark brown.	65.6 65.7	0							X	
		<b>FILL:</b> silty medium grained sand, some gravel, trace of clay, brown to dark brown. (compact to very dense) with concrete pieces from 1.5m to 1.9m depth.		1						X		
				2							X	
		<b>FILL:</b> medium grained sand, some concrete pieces, roots up to 30mm diameter, grey.	63.5 63.4	3							X	
		<b>AUGER REFUSAL</b>		4								
		<b>NOTES:</b>		5								
		1. Borehole drilled with a truck mounted drill rig equipped with continuous flight hollow stem auger equipment to an auger refusal depth of 2.4m on December 28, 1989.		6								
		2. Upon completion of drilling, no cave, no water.		7								
				8								
				9								
				10								

NOTE: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS.

# Log of Borehole 7



Auger Sample ☒ Natural Moisture  
 SPT (N) Value ☐ Plastic and Liquid Limit  
 Dynamic Cone Test ☐ Undrained Triaxial at Overburden Pressure  
 Shelby Tube ☐ % Strain at Failure  
 Field Vane Test ☐ Penetrometer



Project Bank Street Structures Dwg. No. 8  
Bank Street, Ottawa, Ontario  
 Project No. R-00435A/GE  
 Hole location and datum see drawing No. 1

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH m	N Value			Natural Moisture Content and Atterberg Limits % Dry Weight			Natural Unit Weight kN/m³	
					20	40	60	80	10	20		30
					Shear Strength MPa							
		TOPSOIL: silty sand, medium organics, dark brown	64.9	0								
		FILL: silty fine to medium grained sand, some gravel, occasional black cinders, rootlets, non-plastic, frozen, brown. (compact).	64.6	1						X		19.3
				2						X		18.9
		CLAYEY SILT: thin partings of saturated clay and brown silt, fissured, grey, occasional rootlets, decayed 10mm length 3mm diameter wood piece, low to medium plastic, brown. (firm). (possible fill)	62.6	3						X		18.6
			62.3	4						X		21.0
		SANDY SILT TILL: fine grained, some gravel, boulders and shale fragments dry, dark brown. (very dense).	61.1	5						X		22.9
		SANDY SILT: trace of clay, non-plastic, brown. (very dense).	60.4	6						X		
		SHALE BEDROCK:	59.0	7								
			58.3	8								RUN 1
				9								RUN 2
				10								RUN 3
			54.4									

NOTE: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS.

BOREHOLE NO. 7

NOTES:

1. Borehole drilled with a truck mounted drillrig equipped with continuous flight hollow stem auger equipment to an auger refusal depth of 5.9m. Borehole core drilled using BW casing and a BQ core barrel. The borehole was drilled on January 25, 1990.

2. Upon completion of drilling, no cave, no water.

3. Core recovery as follows:

<u>RUN NO.</u>	<u>DEPTH (m)</u>	<u>TCR (%)</u>	<u>RQD (%)</u>
1	5.9 - 7.4	98	24
2	7.4 - 8.9	98	0
3	8.9 - 10.4	88	0

4. Total Core Recovery (T.C.R.) expressed as the ratio of the length of core recovered to the total core run expressed as a percentage.

5. Rock Quality Designation (R.Q.D.) expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

6. Upon completion of overburden drilling, no water.

7. Upon completion of drilling, bi-level piezometer installed from 0.0m to 5.9m depth and 0.0m to 10.5m depth. Piezometer sealed from 0.0m to 0.4m and 5.9m to 6.2m depths.

8. Water level readings as follows:

<u>DATE</u>	<u>WATER LEVEL (m)</u>	
	Shallow	Deep
Jan. 25, 1990	3.9	6.5
Feb. 12, 1990	Blocked	4.5
Feb. 23, 1990	Blocked	5.5
MAR. 11, 1990	2.6	6.6



# Log of Borehole 8



Auger Sample

SPT (N) Value

Dynamic Cone Test

Shelby Tube

Field Vane Test



Natural Moisture



Plastic and Liquid Limit



Undrained Triaxial at Overburden Pressure



4% Strain at Failure



Penetrometer



Project Bank Street Structures

Dwg. No. 9

Bank Street, Ottawa, Ontario

Project No. R-00435A/GE

Hole location and datum see drawing No. 1

G W L	S Y M B O L	Soil Description	ELEV. m	D E P T H m	N Value				Natural Moisture Content and Aterberg Limits % Dry Weight			Natural Unit Weight kN/m <sup>3</sup>
					20	40	60	80	10	20	30	
		ASPHALTIC CONCRETE: concrete	66.0	0								
		GRANULAR FILL: sand and gravel, brown, moist.	65.2	1								
		FILL: silty clay to clayey silt, trace of sand and gravel, oxidized stains, grey, (loose to very dense). frozen from 0.8m to 1.5m depth.		2								18.4
				3								17.9
				4								18.5
		CLAYEY SILT: thin partings of grey silt, mottled reddish brown, slightly to low plastic, moist, brown. (firm).	62.2	5								
		SILT: some clay and fine grained sand, inclusions of brown silty clay, non-plastic, dilatent, grey. (loose).	61.5	6								
		WEATHERED SHALE: grey.	60.0	7								
		AUGER REFUSAL		8								
		NOTES:		9								
		1. Borehole drilled with a truck mounted drill rig equipped with continuous flight hollow stem auger equipment to an auger refusal depth of 6.6m on. January 23, 1990.		10								
		2. Upon completion of drilling, no cave, no water.										

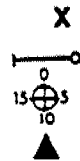
G 009R

NOTE: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS.

# Log of Borehole \_\_\_\_\_



Auger Sample ☒ Natural Moisture  
 SPT (N) Value ☐ Plastic and Liquid Limit  
 Dynamic Cone Test ☐ Undrained Triaxial at  
 Shelby Tube ☐ Overburden Pressure  
 Field Vane Test ☐ % Strain at Failure  
☐ Penetrometer



Project Bank Street Structures Dwg. No. 10

Bank Street, Ottawa, Ontario

Project No. R-00435A/CF

Hole location and datum see drawing No. 1

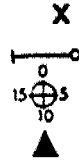
G W L	S Y M B O L	Soil Description	ELEV. m	D E P T H m	N Value				Natural Moisture Content and Atterberg Limits % Dry Weight			Natural Unit Weight kN/m <sup>3</sup>
					20	40	60	80	10	20	30	
		ASPHALTIC CONCRETE	65.4	0								
		GRANULAR FILL: sand and gravel, grey brown, moist.										
		FILL: silty clay, inclusions of reddish brown silt, black staining, dry, brown (very dense).	64.6	1								18.2
		TOPSOIL:	63.9	2								18.4
		SILTY CLAY: some fine grained sand, inclusions of dark brown silt, slightly to medium plastic, grey brown, (very stiff to soft).		3								18.0
				4								17.6
			61.6	5								
		SILTY CLAY: mottled brown, slightly dilatant medium plastic, moist, grey	61.2	6								18.5
		AUGER REFUSAL (possible on boulders)		7								
		NOTES:		8								
		1. Borehole drilled with a truck mounted drill rig equipped with continuous flight hollow stem auger equipment to an auger refusal depth of 4.2m on January 23, 1990.		9								
		2. Upon completion of drilling, no cave, no water.		10								

NOTE: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

# Log of Borehole 10



Auger Sample ☒ Natural Moisture  
 SPT (N) Value ☐ Plastic and Liquid Limit  
 Dynamic Cone Test ☐ Undrained Triaxial at Overburden Pressure  
 Shelby Tube ☐ % Strain at Failure  
 Field Vane Test ☐ Penetrometer



Project Bank Street Structures Dwg. No. 11  
Bank Street, Ottawa, Ontario  
 Project No. R-00435A/GE  
 Hole location and datum see drawing No. 1

G W L	S Y M B O L	Soil Description	ELEV. m	D E P T H m	N Value				Natural Moisture Content and Atterberg Limits % Dry Weight			Natural Unit Weight kN/m <sup>3</sup>
					20	40	60	80	10	20	30	
		ASPHALTIC CONCRETE	64.9	0								
		CONCRETE	64.6									
		GRANULAR FILL: sand and gravel, grey brown, moist.	64.1	1								19.7
		FILL: silty clay, trace of sand, brown staining, medium plastic, brown (very dense).		2								17.9
				3								
		FILL: clayey silt, some gravel to 25mm diameter, inclusions of white powder, moist, grey, to brown. (loose).	61.9	4								18.6
				5								21.8
		SAND: fine to medium grained, some gravel, brown. (very dense).	60.4	6								
				7								
		AUGER REFUSAL	59.3	8								
		NOTES:		9								
		1. Borehole drilled with a truck mounted drill rig equipped with continuous flight hollow stem auger equipment to an auger refusal depth of 5.6m on January 23, 1990.		10								
		2. Upon completion of drilling, no cave, no water.										

NOTE: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS.

NOTE: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS.

BOREHOLE NO. 11

NOTES:

1. Borehole drilled with a truck mounted drillrig equipped with continuous flight hollow stem auger equipment to an auger refusal depth of 7.0m. Borehole core drilled using BW casing and a BQ core barrel. The borehole was drilled on February 20, 1990.

2. Upon completion of overburden drilling, no water, no cave.

3. Core Recovery as follows:

<u>RUN NO.</u>	<u>DEPTH (m)</u>	<u>TCR (%)</u>	<u>RQD (%)</u>
1	7.0 - 7.6	73	25
2	7.6 - 9.1	69	0
3	9.1 - 10.2	57	0

4. Total Core Recovery (T.C.R.) expressed as the ratio of the length of core recovered to the total core run expressed as a percentage.

5. Rock Quality Designation (R.Q.D.) expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.



## BOREHOLE NO. 12

NOTES:

1. Borehole drilled with a truck mounted drillrig equipped with continuous flight hollow stem auger equipment to an auger refusal depth of 7.2m. Borehole core drilled from 7.2m to 10.4m depth with BW casing and BQ core barrel. The borehole was drilled on January 23, 1990.

2. Upon completion of drilling, no cave, no water.

3. Core recovery as follows:

<u>RUN NO.</u>	<u>DEPTH (m)</u>	<u>TCR (%)</u>	<u>RQD (%)</u>
1	7.2 - 7.6	72	48
2	7.6 - 9.1	86	0
3	9.1 - 10.4	94	0

4. Total Core Recovery (T.C.R.) expressed as the ratio of the length of core recovered to the total core run expressed as a percentage.

5. Rock Quality Designation (R.Q.D.) expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

6. Upon completion of overburden drilling, no water.

7. Upon completion of drilling, bi-level piezometer installed from 0.0m to 7.2m depth and from 0.0m to 10.4m depth. Piezometer sealed from 0.0m to 0.4m depth and 7.2m to 7.6m depth.

8. Water level reading as follows:

<u>DATE</u>	<u>WATER LEVEL (m)</u>	
	<u>Shallow</u>	<u>Deep</u>
Jan. 25, 1990	2.0	6.9
Feb. 23, 1990	2.6	7.0
Mar. 11, 1990	2.4	7.0

# RECORD OF BOREHOLE 87-4

SHEET 1 of 1



LOCATION See Figure 2

BORING DATE Jan. 15, 1988

DATUM Geodetic

SAMPLER HAMMER, 83.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 83.5kg, DROP, 760mm

SCAL METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, CM/SEC		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3M	SHEAR STRENGTH Cu, kPa	nat.V.- + Q.- ● rem.V.- ⊗ U.- □	WATER CONTENT, PERCENT Wp W L		
0	Power Auger 200mm Diam (Hollow Stem)	Ground Surface	84.82								Bentonite Seal Backfill
		TOPSOIL	0.00								
			0.12								
		Very stiff grey brown silty clay, trace gravel and roots (FILL)									
1			63.48	1	50 DO	3					
		Loose brown SANDY SILT	1.18								
			62.94								
2			1.68	2	50 DO	10					
		Very stiff grey brown SILTY CLAY (Weathered Crust)									
				3	50 DO	11					
3		61.83									
		2.99	4	50 DO	7						
	Very stiff layered grey brown SILTY CLAY and CLAYEY SILT, occasional sandy silt seam										
4		60.35	5	50 DO	4						
	Loose brown SANDY SILT, some gravel	4.27									
		59.90									
5		59.72	6	50 DO	13						
	Compact brown fine SAND, some silt	59.68									
		4.94									
	Loose brown fine to coarse SAND, trace silt		7	50 DO	5						
6											
		58.37	8	50 DO	100						
		6.25									
7	Rotary Drilling BXL Core	Fresh laminated dark grey SHALE BEDROCK, occasional thin calcareous siltstone seam. Zones of fractured core from 6.3m to 6.4m, 6.7m to 6.9m, 7.0m to 7.3m. Near vertical open and calcite filled joints throughout core retrieved (BILLINGS FORMATION)		9	BX RC	1	Core Recovery (%) 86 Solid Core Recovery (%) 30 R.Q.D (%) 9				
8					10	BX RC	1	Core Recovery (%) 100 Solid Core Recovery (%) 44 R.Q.D (%) 0			
9		End of Hole	55.68								
			8.98								
10											

0

10

15

5 PERCENT AXIAL STRAIN AT FAILURE

W.L. in  
Standpipe at  
Elev. 58.14  
Jan. 28, 1988

Bentonite  
Seal  
Backfill

W.L. in  
Standpipe at  
Elev. 58.14  
Jan. 28, 1988

DEPTH SCALE

1: 50

Golder Associates

LOGGED J.COBISA

CHECKED *de*

0  
15-25 PERCENT AXIAL STRAIN AT FAILURE  
10



RECORD OF BOREHOLE 85-6

LOCATION See Figure 2

BORING DATE APR. 17, 1985

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH Cu, kPa		NAT. V. - +	Q. - ●	WATER CONTENT, PERCENT		1x10	1x10		
66.18	GROUND SURFACE															
0.00	TOP SOIL															
0.18	COMPACT BROWN FINE TO COARSE SAND, SOME GRAVEL, TRACE ASHES ASPHALT (FILL)		1	SO	19	66										
65.11			2	"	22	65										
1.07	VERY STIFF GREY BROWN SILTY CLAY, OCCASIONAL FINE SAND (WEATHERED CRUST)		3	"	17											
			4	"	22	64										
			5	"	25	63										
			6	"	10											
62.96																
3.72	COMPACT BROWN FINE TO MEDIUM SAND, SOME SILTY CLAY LAYERS		7	"	16	62										
61.91																
4.27	COMPACT BLACK SILTY SAND, SOME GRAVEL, TRACE CLAY (GLACIAL TILL)		8	"	100											
4.48			9	"												
	FRESH TO FAINTLY WEATHERED LAMINATED TO THINLY BEDDED DARK GREY SHALE BEDROCK, OCCASIONAL SHALEY LIMESTONE LAYER, BEDDING AT LESS THAN 5° TO HORIZONTAL		10	"		61										
			11	"												
			12	"		60										
			13	"		59										
			14	"												
58.38																
7.80	END OF HOLE					58										

GROUND SURFACE

PLASTIC TUBING

NATIVE BACKFILL

BENTONITE SEAL

STANDPIPE

WATER LEVEL IN STANDPIPE AT ELEVATION 60.88 MAY 1, 1985

CORE RECOVERY (%)

R.O.D. (%)

R.O.D. AFTER 1 CYCLE OF WETTING & DRYING

4.63

4.79

5.09

7.10

7.71

HIGHLY FRACTURED, HIGHLY WEATHERED ROCK

CLAY FILLED NEAR VERTICAL JOINT

HIGHLY FRACTURED, HIGHLY WEATHERED ROCK

CLAY FILLED NEAR VERTICAL JOINT

Percent axial strain at failure

0

10

# OVERSIZE DRAWING(S)

# RECORD OF BOREHOLE I

**LOCATION** See Figure 2

**BORING DATE**    **MARCH 18, 1982**

**DATUM    GEODETIC**

**SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm**

**PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm**

SOIL PROFILE		SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m	HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV.'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		BLOWS/0.3m	SHEAR STRENGTH C <sub>u</sub> , kPa	NAT. V. - + REM. V. - ⊕ U.-O	O.-● U.-O	WATER CONTENT, PERCENT <div style="text-align:center;">1x10   1x10   1x10   1x10 Wp   W   WL</div>		
60.94	ASPHALT											
60.94	GROUND SURFACE											GROUND SURFACE
0.06	BROWN SAND (GRAVEL FILL)											
0.24	BROWN SILTY SAND, SOME GRAVEL, OCCASIONAL WOOD AND COBBLES (FILL)		50	"	100							BENTONITE SEAL
59.57			1	"	60							
1.37	GREY BROWN AND DARK BROWN SILTY CLAY, TRACE GRAVEL, WOOD AND ORGANIC MATTER (FILL)		2	"	59							PLASTIC TUBING
			3	"	58							
57.80			4	"								
3.14	VERY DENSE GREY BROWN SILTY SAND, SOME GRAVEL, TRACE CLAY, OCCASIONAL SAND LAYER (GLACIAL TILL)		5	"	57							MH NATIVE BACKFILL
			6	"	56							
55.91												
5.03	PROBABLY VERY DENSE GLACIAL TILL				55							
					54							
53.3A												STANDPIPE
7.56	END OF HOLE AUGER REFUSAL POSSIBLY BEDROCK				53							W.L. IN STANDPIPE AT ELEV. 57.95 MAR. 24, 1982
					52							

VERTICAL SCALE  
1:50

## Golder Associates

DRAWN DN  
CHECKED JS

RECORD OF AUGERHOLES 2 & 3

LOCATION See Figure 2

BORING DATE MARCH 15, 1982

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH		NAT. V. - +		WATER CONTENT, PERCENT					
								Cu, kPa	REM. V. - +	Q - ●	U - ○	Wp	W	WL			
POWER AUGER 200mm DIAM. (HOLLOW STEM)		ASPHALT					64	AH. 2									
	63.21	GROUND SURFACE															
	0.06	BROWN SAND & GRAVEL (FILL)					63										
	0.27	GREY BROWN SILTY SAND															
	62.57	TRACE GRAVEL (FILL)															
	0.64	GREY BROWN SILTY CLAY (WEATHERED CRUST)					62										
	61.53																
	1.68	DARK BROWN SILTY SAND, GRAVEL AND COBBLES, TRACE CLAY (GLACIAL TILL)					61										
	60.16																
	3.05	END OF HOLE					60										
POWER AUGER 200mm DIAM. (HOLLOW STEM)		ASPHALT					66	AH. 3									
	65.09	GROUND SURFACE															
	0.06	BROWN SAND AND GRAVEL (FILL)					65										
	64.67																
	0.42	GREY BROWN SILTY CLAY (WEATHERED CRUST)					64										
							63										
	62.19																
	2.90	GREY BROWN GLACIAL TILL					62										
	3.05	END OF HOLE					61										

0 15 10

Percent axial strain at failure

VERTICAL SCALE

1:50

Golder Associates

DRAWN DN

CHECKED SW

RECORD OF BOREHOLE 4

LOCATION See Figure 2      BORING DATE MARCH 16, 1982      DATUM GEODETIC  
SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm      PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH				WATER CONTENT, PERCENT					
								Cu, kPa	NAT. V. - +	Q. - ●	REM. V. - ⊕	U. - ○	Wp	W	WL		
200 mm DIAM. (HOLLOW STEM)	65.82	GROUND SURFACE					66									GROUND SURFACE	
	0.00	TOPSOIL															
	65.12	VERY STIFF GREY BROWN SILTY CLAY, OCCASIONAL CLAYEY SILT LAYER (WEATHERED CRUST)					65					0				ELASTIC TUBING	
	0.70			1	50 mm D.O.	17											
				2	"	11	64					0					
				3	"	8	63					0					
	62.23	COMPACT TO DENSE DARK BROWN SILTY SAND AND GRAVEL TRACE CLAY, OCCASIONAL COBBLES (GLACIAL TILL)					62									NATIVE BACKFILL	
	3.59			4	"	10											
				5	"	20											
				6	"	31	61										
	60.49	VERY DENSE GREY BROWN SILTY SAND, SOME GRAVEL, COBBLES AND BOULDERS (GLACIAL TILL)					60									STANDPIPE	
	5.33			7	"	>100	59										
				8	"	>100	58										
				9	"	>100	57										
	57.19	END OF HOLE AUGER REFUSAL, POSSIBLY BEDROCK					56									W.L. IN STANDPIPE AT ELEV. 57.61 MAR. 24, 1982	
	8.63																

0

5

10

15

Percent axial strain at failure

VERTICAL SCALE  
1:50

Golder Associates

DRAWN DN  
CHECKED [Signature]

RECORD OF BOREHOLE 5

LOCATION See Figure 2 BORING DATE MARCH 15, 1982 DATUM GEODETIC  
SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION				
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH		NAT. V. - + Q. - ●	REM. V. - ⊕ U. - ○	WATER CONTENT, PERCENT									
								Cu, kPa				Wp									
								20	40	60	80	1x10	1x10	1x10	1x10						
200 mm DIAM (HOLLOW STEM)	65.25	GROUND SURFACE					66									GROUND SURFACE					
	0.00	TOPSOIL					65										PLASTIC TUBING				
	0.18	VERY STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)					64											NATIVE BACKFILL			
			1	50 mm D.O.	4										STANDPIPE						
			2	"	5														1 MH		
																				2 MH	
		3	"	2										CAVED IN MATERIAL							
	61.59	INTERLAYERED STIFF GREY SILTY CLAY AND CLAYEY SILT																			W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198
	3.66																				
	60.43	LOOSE DARK BROWN SILTY SAND AND GRAVEL, TRACE CLAY (GLACIAL TILL)																			
4.82	4		"	4									W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198								
59.40	VERY DENSE BROWN SAND AND GRAVEL, SOME SILT, OCCASIONAL COBBLES AND BOULDERS															W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198					
5.85																	W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198				
		5	"	100											W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198						
																		W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198			
56.26	END OF HOLE - AUGER REFUSAL POSSIBLY BEDROCK																		W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198		
8.99														W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198							
																				W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198	
												W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198									
																					W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198
													W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198								
																W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198					
																	W L IN STANDPIPE A ELEV. 61.35 MAR. 24, 198				
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15 0 5 10 Percent axial strain at failure

VERTICAL SCALE  
1:50

Golder Associates

DRAWN DN  
CHECKED 22

# RECORD OF BOREHOLE 6

LOCATION See Figure 2

**BORING DATE**    **MARCH 17 & 19, 1982**

**DATUM GEODETIC**

**SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm**

**PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm**

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH Cu, kPa				WATER CONTENT, PERCENT					
								20	40	60	80	NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○	Wp	W	WL		
POWER AUGER 200mm DIAM. (HOLLOW STEM)	67.73 0.00	GROUND SURFACE					68									GROUND SURFACE Z	
		VERY STIFF GREY BROWN SILTY CLAY, OCCASIONAL SILTY SAND LAYERS BELOW ELEV. 65.29 (WEATHERED CRUST)		1	50 PER DO.	4	67										
				2	"	14	66										
				3	"	5	65										
				4	"	4	64										
				5	"	6	63										
				6	"	6	62										
				7	"	4	61										
				8	"	3	60										
	60.96 6.77	INTERLAYERED STIFF GREY SILTY CLAY AND CLAYEY SILT															
				9	" PH		59										
	58.92 8.81	DENSE DARK BROWN SILTY SAND, SOME GRAVEL AND COBBLES TRACE CLAY, OCCASIONAL BOULDERS (GLACIAL TILL)		10	"	47	58										
							57										
							56										
							55										
							54										
54.53 13.20	END OF HOLE AUGER REFUSAL POSSIBLY BEDROCK														STANDPIPE 'A'		

W.L. IN STANDPIPE 'A' ELEV. 62.21  
STANDPIPE 'B' ELEV. 65.69  
MAR. 24, 1968

VERTICAL SCALE  
1:50

## Golder Associates

DRAWN DN  
CHECKED JF

**FORM U.S.A.-10-78**

**DATUM    GEODETIC**

**PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm**

WEST MOTT (MAY 1968)

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CHECKED [Signature]



RECORD OF BOREHOLE 8

LOCATION See Figure 2

BORING DATE MARCH 17 & 18, 1982

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	N	T	BLOWS/0.3m		SHEAR STRENGTH Cu, kPa		NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○		WATER CONTENT, PERCENT						
								20	40	60	80	Wp	W	WL				
200 mm DIAM. (HOLLOW STEM)	66.79 0.00	GROUND SURFACE					67										GROUND SURFACE	
		VERY STIFF GREY BROWN SILTY CLAY, TRACE ORGANIC MATTER AND WOOD (PROBABLY FILL)	X				66										PLASTIC TUBING	
				1	50 mm DO	8												
				2	"	4	65											
	64.32 2.47	VERY STIFF TO STIFF GREY BROWN SILTY CLAY, OCCASIONAL SAND LAYERS (WEATHERED CRUST)					64	⊕			+ 144						NATIVE BACKFILL	
									⊕		+ 138							
				3	"	3	63											
							⊕			+ 109								
							⊕		+ 132									
				4		3	62										STANDPIPE 'B'	
							61	⊕			+						NATIVE BACKFILL	
								⊕			+							
				5	"	3	60											
								⊕			+							
	60.05 6.74	STIFF TO VERY STIFF GREY SILTY CLAY, OCCASIONAL SAND LAYERS					59											
								58	⊕			+ 103						STANDPIPE 'A'
57.46 9.33 57.16 9.63	LOOSE TO VERY DENSE GLACIAL TILL		7	50 mm DO	5												W.L. IN STANDPIPE 'A' ELEV. 65.01 STANDPIPE 'B' ELEV. 62.17 MAR. 24, 1982	
	END OF HOLE																	

VERTICAL SCALE  
1:50

Golder Associates

DRAWN DN  
CHECKED

# RECORD OF BOREHOLE 9

**LOCATION** See Figure 2

**BORING DATE**    **MARCH 17, 1982**

DATUM GEODETIC

**SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm**

**PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm**

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH Cu, kPa				WATER CONTENT, PERCENT					
								20	40	60	80	1x10	1x10	1x10	1x10		
POWER AUGER 200mm DIAM. (HOLLOW STEM)	70.44	GROUND SURFACE					71									GROUND SURFACE PLASTIC TUBING NATIVE BACKFILL STANDPIPE W.L. IN STANDPIPE AT ELEV. 68.92 MAR. 24, 1988	
	0.00	BROWN SILTY CLAY, SOME GRAVEL, TRACE SAND AND BRICK FRAGMENTS, OCCASIONAL COBBLES (FILL)					70										
	69.28 1.16	VERY STIFF GREY BROWN SILTY CLAY, SOME SAND LAYERS BELOW ELEV. 67.00 (WEATHERED CRUST)		1	50 mm 0.0	18	69										
				2	"	7	68										
				3	"	5	67										
				4	"	5	66										
				5	"	3	65										
	66.11 4.33	STIFF GREY SILTY CLAY		6	"	1	64										
	64.65	END OF HOLE															
	5.79																

VERTICAL SCALE  
1:50

Golder Associates

DRAWN DN  
 CHECKED DP

# RECORD OF BOREHOLE 10

**LOCATION** See Figure 2

**BORING DATE**      **MARCH 18, 1982**

**DATUM    GEODETIC**

**SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm**

**PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm**

SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m	HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION			
ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		BLOWS/0.3 m	SHEAR STRENGTH Cu, kPa	NAT.V.- + REM.V.- ⊕	Q.-● U.-○	WATER CONTENT, PERCENT					
										1x10			1x10	1x10	1x10
	ASPHALT														
61.10	GROUND SURFACE												GROUND SURFACE		
60.92	BROWN SAND (GRAVEL FILL)														
60.80	BROWN FINE TO MEDIUM SAND, TRACE SILT (FILL)														
60.52															
60.58															
	VERY STIFF GREY BROWN SILTY CLAY, OCCASIONAL CLAYEY SILT LAYER BELOW ELEV. 57.00 (WEATHERED CRUST)		1	"	50 mm D.O.	27									
			2	"	10	59									
			3	"	9	58									
			4	"	5	57									
56.86	INTERLAYERED STIFF GREY SILTY CLAY AND CLAYEY SILT		5	"	4	56									
4.24			6	"	75 mm T.O.	34									
56.10	DENSE DARK BROWN GLACIAL TILL		7	"	50 mm D.O.	55									
5.00			8	"	9	54									
55.77			9	"	17	53									
5.33						52									
	LOOSE TO COMPACT BROWN FINE TO COARSE SAND					51									
50.55						50									
10.55	END OF HOLE														

W.L. IN STANDPIPE AT ELEV. 58.05 MAR. 24, 1982

VERTICAL SCALE  
1:50

## Golder Associates

DRAWN DN  
CHECKED CE

BORING DATE MARCH 16 & 19, 1982

DATUM GEODETIC

**LOCATION** See Figure 2

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

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VERTICAL SCALE  
1:50

## Golder Associates

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RECORD OF BOREHOLES 12 & 13

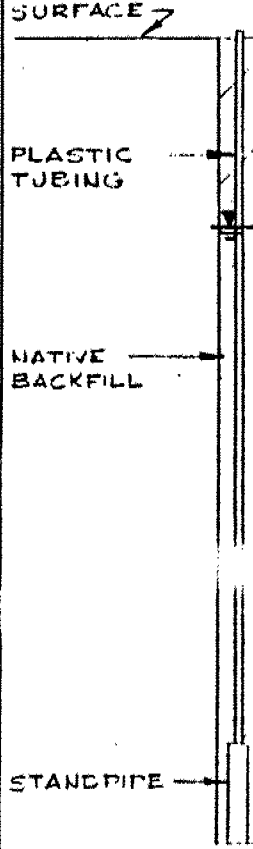
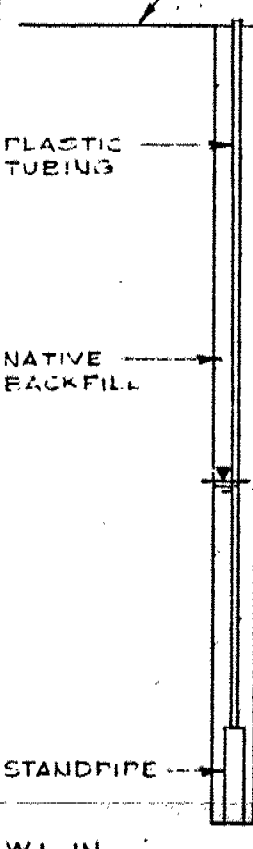
LOCATION See Figure 2

BORING DATE MARCH 16, 1982

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m			HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH Cu, kPa		NAT. V. - + REM. V. - ⊕	Q. - ● U. - ○	WATER CONTENT, PERCENT Wp      W      Wl				
68.54	GROUND SURFACE														
0.00	BROWN SILTY SAND, SOME GRAVEL, OCCASIONAL BRICK FRAGMENTS, COBBLES AND BOULDERS (FILL)														
67.63			1	50 mm D.O.	3										
0.91			2	"	2										
			3	"	1										
			4	"	3										
			5	"	2										
			6	"	2										
63.36															
5.18	END OF HOLE														
															W.L. IN STANDPIPE AT ELEV. 67.32 MAR. 24, 1982
67.45	GROUND SURFACE														
0.00	BROWN SILTY CLAY, SOME SAND AND GRAVEL (FILL)														
66.99			1	50 mm D.O.	5										
0.46			2	"	5										
			3	"	3										
			4	"	3										
			5	"	7										
			6	"	4										
62.27															
5.18	END OF HOLE														
															W.L. IN STANDPIPE AT ELEV. 64.48 MAR. 24, 1982

0 5 10 Percent axial strain at failure

VERTICAL SCALE  
1:50

Golder Associates

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CHECKED DAI

## RECORD OF AUGERHOLE 14


LOCATION See Figure 2


**BORING DATE**      **MARCH 16, 1982**

**DATUM    GEODETIC**

**SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm**

**PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm**

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m						HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETRE OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m													
								SHEAR STRENGTH Cu, kPa      NAT.V.- + Q-● REM.V.- ⊗ U-O						WATER CONTENT, PERCENT Wp      W      WL					
	64.54	GROUND SURFACE					65												
POWER AUGER 200mm DIAM. (HOLLOW STEM)	0.00	GREY BROWN SILTY CLAY, SOME SAND, TRACE GRAVEL AND BRICK FRAGMENTS (FILL)					64												
	63.02								63										
	1.52			END OF HOLE					62										


 0 — 5 — 10    Percent axial strain at failure

VERTICAL SCALE  
1:50

## Golder Associates

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CHECKED Z

EXTENSION TO  
REPORT ON  
SUBSURFACE INVESTIGATION  
FOR  
SOUTHEAST TRANSITWAY - CN BEACHBURG  
BANK STREET  
TO PROPOSED  
CNR GRADE SEPARATION  
OTTAWA, ONTARIO  
TO  
McCORMICK RANKIN  
AND THE  
REGIONAL MUNICIPALITY OF OTTAWA-CARLETON

Report No. SF-4036  
August 9, 1991

**McROSTIE GENEST ST-LOUIS**

& ASSOCIATES LTD. - CONSULTING ENGINEERS  
& ASSOCIÉS LTÉE - INGÉNIEURS CONSEILS  
OTTAWA CANADA

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	Plate No.
Borehole & Test Pit Location Plan	1
Test Pit Records	2 to 10
Effect of Watermain Trench	11
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Past Correspondence



# McROSTIE GENEST ST-LOUIS

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OTTAWA, ONTARIO K2C 0P9

## 1. TERMS OF REFERENCE

We were recently authorized by McCormick Rankin Associates Limited, on behalf of RMOC (Regional Municipality of Ottawa-Carleton) to carry out additional subsurface investigation for the Southeast Transitway between Bank Street and the proposed CNR grade separation, in Ottawa, Ontario.

Hand dug test holes were to be made between Stations 10+010 and 10+040 in the vicinity of the proposed north retaining wall.

Detailed recommendations regarding the temporary shoring and anchoring for the construction of the pedestrian underpass were also to be included in this report.

## 2. OBSERVATIONS, CONCLUSIONS & RECOMMENDATIONS

### 2.1. Subsurface Conditions

The nine (9) test pits and auger holes made as part of the present study combined with information gathered as part of previous studies would appear to indicate that the sloping land between Bank Street to about Station 10+060 is not an extension of the clay slope to the west but rather a man-made embankment for the railway. The embankment is composed of pit run gravel and sand with some boulders, according to the test pits. The material was likely placed end-dumped and is therefore expected to be variable in density.

Bedrock gets deeper in a south westerly direction in conjunction with a thinner clay layer. Due to past topographic features, the clay crust is found at increasing elevations also in a southwesterly direction. Where the clay layer is thin, the clay is all crust and hard to very stiff.

### 2.2. Soil Bearing Pressures

The foundations for the north retaining wall will need to be lowered through the fill layers and onto undisturbed natural clay soils as encountered in the test pits, auger holes and boreholes. An alternative would be to remove the fill and replace it with controlled selected backfill.

## 2.2. cont'd

A review of all information gathered to date, including the most recent test pit records, has allowed for a re-analysis of allowable bearing pressures. Our recommendations are as follows:-

	<u>BEARING PRESSURE (kPa)</u>	
	SLS II	ULS
Bank Street to 10+075	250	500
10+075 to 10+150	200	400
West of 10+150	150	300

The foundations for the most easterly portion of the north retaining wall can be lowered onto the natural clay or made to bear on compacted selected granular fill such as MTO -Granular 'B' (type II). The same bearing pressures should be used for both conditions since the granular separation would be relatively thin.

We would recommend that the transitions between the different bearing pressure values be made over a distance approximately equal to the height of the wall.

2.3. Resistance to Sliding

In accordance with the CFEM, passive resistances for the north retaining wall should be computed using a coefficient of passive pressure reduced to account for movements required to develop these.

## 2.3. cont'd

We recommend that a reduction factor of 2.5 be applied to  $K_p$  in order to reduce the movements to develop these levels of resistance compatible with those to develop adhesion under the footing. Both mechanisms would act simultaneously at the levels of resistance recommended or better.

The passive earth pressure coefficients recommended in front of the retaining walls are as follows:-

	<u>ULS</u>	<u>SLS II</u>
$K_p$	2.70	3.40

Although the allowable adhesion under footings had been recommended with respect to geodetic elevation 70 m in earlier reports, recent findings and re-analysis show that between Bank Street and Station 10+150, this general recommendation is no longer applicable because of the clay crust being at a lower geodetic elevation.

In this part of the project, we recommend the following for adhesion under footings subjected to lateral loading conditions.

<u>ULS</u>	<u>SLS II</u>
50 kPa	100 kPa

#### 2.4. Effect of Watermain Trench

The lateral stability of the retaining walls at the location of proposed utility pipes needs to be assured. The edge of trench needs to be at a minimum distance so that it falls beyond an imaginary line drawn at two (2) horizontal to one (1) vertical from the underside of footing. The trench backfill needs to be compacted however in order to provide lateral resistance.

A typical section as provided to us for Station 10+100 shows geometry in accordance with the above.

Since the construction sequence calls for the construction of the watermain prior to that of the wall, considering an excavation about six (6) metres away from the wall face and also considering that any future repairs to the watermain would likely be localized; for design purposes, some reliance on passive resistance would be reasonable.

#### 2.5. Excavation Slopes

It will be necessary to excavate into the man-made embankment just west of Bank Street. We recommend that a one on one (45 degree) excavation be called for in this pit-run gravel/sand mixture. The stability of the excavation slopes in these granular soils would not be a problem in this area, even under the influence of train loads because the geometry of construction slopes would be very close to present conditions.

## 2.5. cont'd

The pit-run gravel/sand mixture can be left in place under the future transitway route. The excavated material can also be re-used as backfill. These granular soils would have material properties similar to MTO Granular 'B' (type I).

## 2.6. Temporary Shoring Pressures

Since earth retaining systems are deformation dependant; by making the system less rigid and allowing for more movements to occur in the bracing, the earth pressures to be resisted could be reduced to a triangular distribution. An estimate of settlement at the centreline of track with the proposed distributions on Plate 52A would be about 50 mm with about 10 mm differential between ends of ties.

Of course, this depends on the quality of workmanship so packing and/or grouting behind lagging boards is very important. Also, prestressing of tie-back tendons should be limited to only the earth pressure portion of the total load in order to avoid having too rigid a system. The movement of the shoring and of the train tracks should be monitored throughout the construction period.

The pressure diagrams recommend have no allowance for frost pressure that would develop under winter conditions. The shoring can be designed using the pressure envelopes recommended without factoring but a factor of safety of at least two (2) is required to reduce the passive resistance.

### 2.7. Deadman Anchors

The total horizontal passive force resulting from the soil wedge in front of an anchor block can be approximated by the triangular passive pressure extending to the ground surface when the soil above the anchorage is less than half the height of the deadman.

Based on conventional deadman design, this horizontal passive resistance can be assumed to act relatively uniformly on the face of the anchor block. Redistribution of stresses in the soil mass due to compensating strains are such that for design purposes, equalizing total horizontal forces would be acceptable.

In order to increase anchorage capacity by a significant amount, increasing the depth of anchor is the most effective means. Therefore, a relatively rigid driven sheet-pile wall is likely the most economical anchor. We understand that the tie-backs would be at depths of 2.5m and 5.5 m at the shoring and would be near horizontal in order to avoid a vertical downward force component in the soldier piles.

At a certain depth below the lowest tie-back, the sheet-piling would bend rather than plough against a soil face to develop passive resistance and would therefore be ineffective. This maximum depth has been evaluated as being two (2) metres below the lower tie-back.

### 3. DETAILS OF THE INVESTIGATION

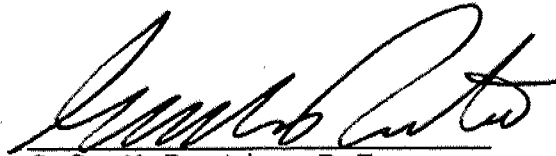
Nine (9) hand dug test pits extended by hand augering were made at the site in the locations shown on Plate No. 1 of this report. These holes were made in sloping ground inaccessible to conventional drilling equipment.

The information gathered as part of the present subsurface investigation allowed us to speculate on the most probable natural topographic features before the introduction of the railway line. This investigation therefore enabled us to modify earlier recommendations on bearing pressure between Stations 10+010 and 10+150.


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This report prepared by:

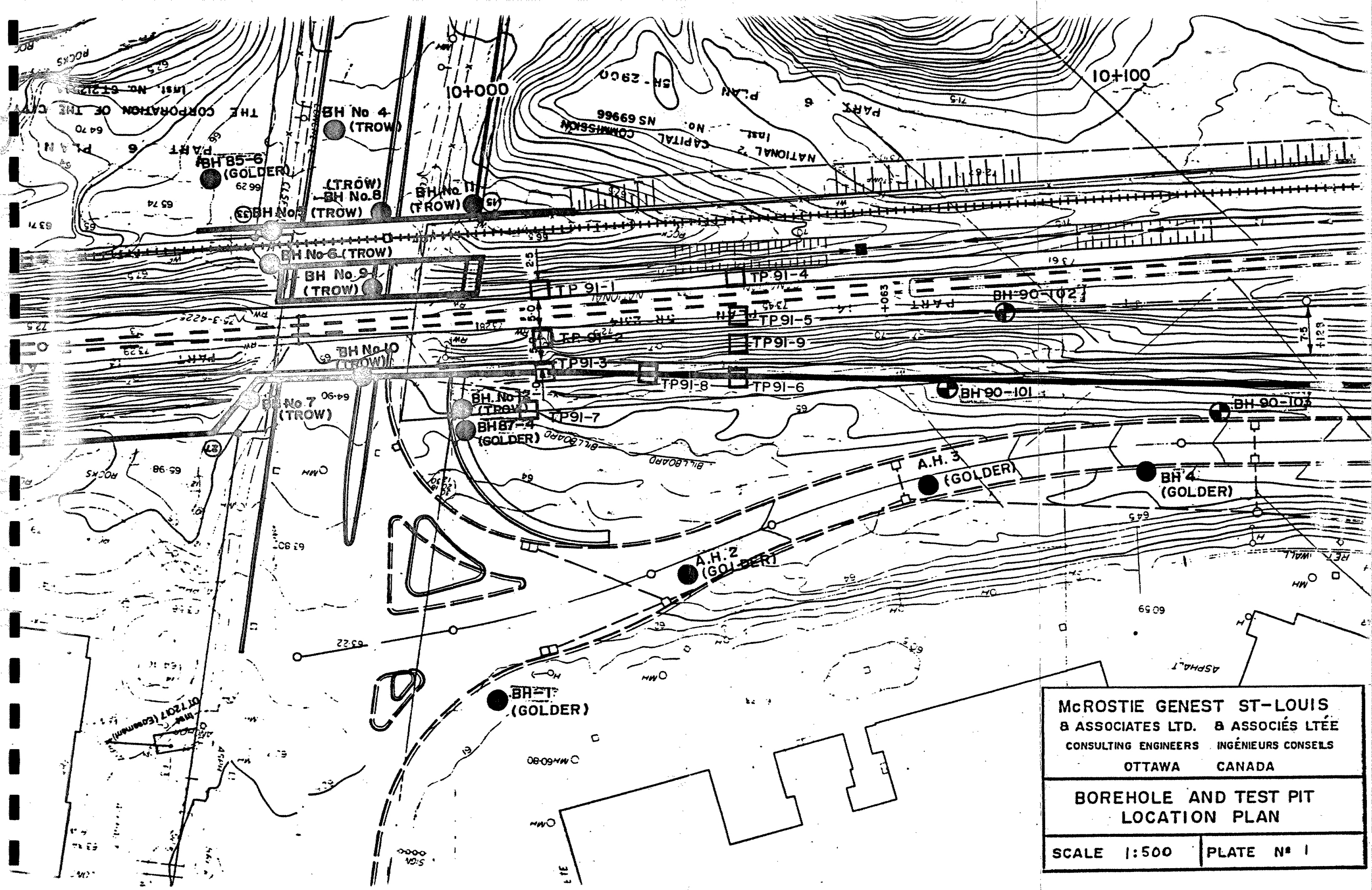


  
 G.C. McRostie, P.Eng.  
 McRostie Genest St-Louis  
 & Associates Ltd.



  
 M.W. St-Louis, P.Eng.  
 McRostie Genest St-Louis  
 & Associates Ltd.





McROSTIE GENEST ST-LOUIS & ASSOCIATES LTD. & ASSOCIÉS LTÉE CONSULTING ENGINEERS INGÉNIEURS CONSEILS OTTAWA CANADA	
BOREHOLE AND TEST PIT LOCATION PLAN	
SCALE 1:500	PLATE N° 1

MCROSTIE GENEST ST-LOUIS & Associates Ltd. Consulting Engineers OTTAWA, CANADA		TEST PIT RECORD Date :	Test Pit No. 91-1 JULY 17 1991
SOUTH EAST TRANSITWAY			
ELEV. 72.77	DEPTH in metres	DESCRIPTION	REMARKS
71.77	-- 1 --	FILL pit run gravel with some sand	hand dug pit to ELEV 71.27m
71.27	1.5	FILL pit run gravel with some sand	bottom of pit hole advanced by hand auger
70.77	-- 2 --	FILL pit run gravel with some sand	no water seepage
70.12	2.65	Bottom of auger hole	Plate No. 2

McROSTIE GENEST ST-LOUIS & Associates Ltd. Consulting Engineers OTTAWA, CANADA		TEST PIT RECORD Date :	Test Pit No. 91-2 JULY 17 1991
SOUTH EAST TRANSITWAY			
ELEV. 71.59	DEPTH in metres	DESCRIPTION	REMARKS
		FILL pit run gravel with some sand	hand dug pit to ELEV 70.39m
70.59	-- 1 --		
70.39	1.2	FILL pit run gravel with some sand	bottom of pit hole advanced by hand auger
69.59	-- 2 --		
69.29	2.3	Bottom of auger hole	no water seepage
			Plate No. 3

MCROSTIE GENEST ST-LOUIS & Associates Ltd. Consulting Engineers OTTAWA, CANADA		TEST PIT RECORD	Test Pit No. 91-3
		Date :	JULY 17 1991
SOUTH EAST TRANSITWAY			
ELEV. 68.32	DEPTH in metres	DESCRIPTION	REMARKS
67.32	-- 1 --	FILL  pit run gravel with some sand	hand dug pit to ELEV 66.22m
66.32 66.22	-- 2 -- 2.1	FILL  pit run gravel and sand	bottom of pit hole advanced by hand auger
65.47	2.85	Bottom of hole	no water seepage
			Plate No. 4

McROSTIE GENEST ST-LOUIS & Associates Ltd. Consulting Engineers OTTAWA, CANADA		TEST PIT RECORD Date :	Test Pit No. 91-4 JULY 17 1991
SOUTH EAST TRANSITWAY			
ELEV. 72.82	DEPTH in metres	DESCRIPTION	REMARKS
		FILL pit run gravel and coarse sand	power auger to ELEV 71.62m hole caved in hand dug pit to ELEV 71.62m
71.82	-- 1 --		
71.62	1.2	FILL pit run gravel and coarse sand	bottom of pit hole advanced by hand auger
70.82	-- 2 --		
70.65	2.17	Bottom of hole	auger refusal at ELEV 70.65m
			Plate No. 5

MCROSTIE GENEST ST-LOUIS & Associates Ltd. Consulting Engineers OTTAWA, CANADA		TEST PIT RECORD	Test Pit No. 91-5
		Date :	JULY 17 1991
SOUTH EAST TRANSITWAY			
ELEV. 72.77	DEPTH in metres	DESCRIPTION	REMARKS
		FILL pit run gravel and coarse sand	power auger to ELEV 71.57m hole caving in hand dug pit to ELEV 71.57m
71.77	-- 1 --		
71.57	1.2	FILL pit run gravel and coarse sand	bottom of pit hole advanced by hand auger
71.17	1.6		auger refusal at ELEV 71.17m
			Plate No. 6

McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
91-6

Date :

JULY 17 1991

SOUTH EAST TRANSITWAY

ELEV. 66.56	DEPTH in metres	DESCRIPTION	REMARKS
		FILL	hand dug pit to ELEV 65.06m
65.56	-- 1 --	pit run gravel and coarse sand with 2 300mm Ø boulders, 4- 450mm Ø boulders, and 1 boulder 600mm x 300mm x 150mm	
65.06	1.5	----- silty fine and very fine SAND	bottom of pit hole advanced by hand auger
64.56	-- 2 --	----- sandy brownish gray CLAY	
64.36	2.2	----- Bottom of hole	no water seepage
			----- Plate No. 7

McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
91-7

Date :

JULY 18 1991

SOUTH EAST TRANSITWAY

ELEV. 64.83	DEPTH in metres	DESCRIPTION	REMARKS
64.68	0.15	TOPSOIL	hand dug pit to ELEV 63.78m
63.83 63.78	-- 1 -- 1.05	FILL pit run gravel and sand	
62.83	-- 2 --	hard brownish gray CLAY	bottom of pit hole advanced by hand auger vane shear strength U - 421 kPa R - 51kPa
62.53	2.3	Bottom of hole	U - 409 kPa R - 77kPa no water seepage
			U - undisturbed R - remoulded
			Plate No. 8



McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

Test Pit No.  
91-8

Date :

JULY 19 1991

SOUTH EAST TRANSITWAY

ELEV. 67.76	DEPTH in metres	DESCRIPTION	REMARKS
67.61	0.15	FILL topsoil and clay	hand dug pit to ELEV 65.96m
66.76	-- 1 --	FILL silty sand and gravel with 150mm $\phi$ boulders	
65.96	1.8	Bottom of pit	no water seepage
			Plate No. 9

McROSTIE GENEST ST-LOUIS  
& Associates Ltd.  
Consulting Engineers  
OTTAWA, CANADA

TEST PIT RECORD

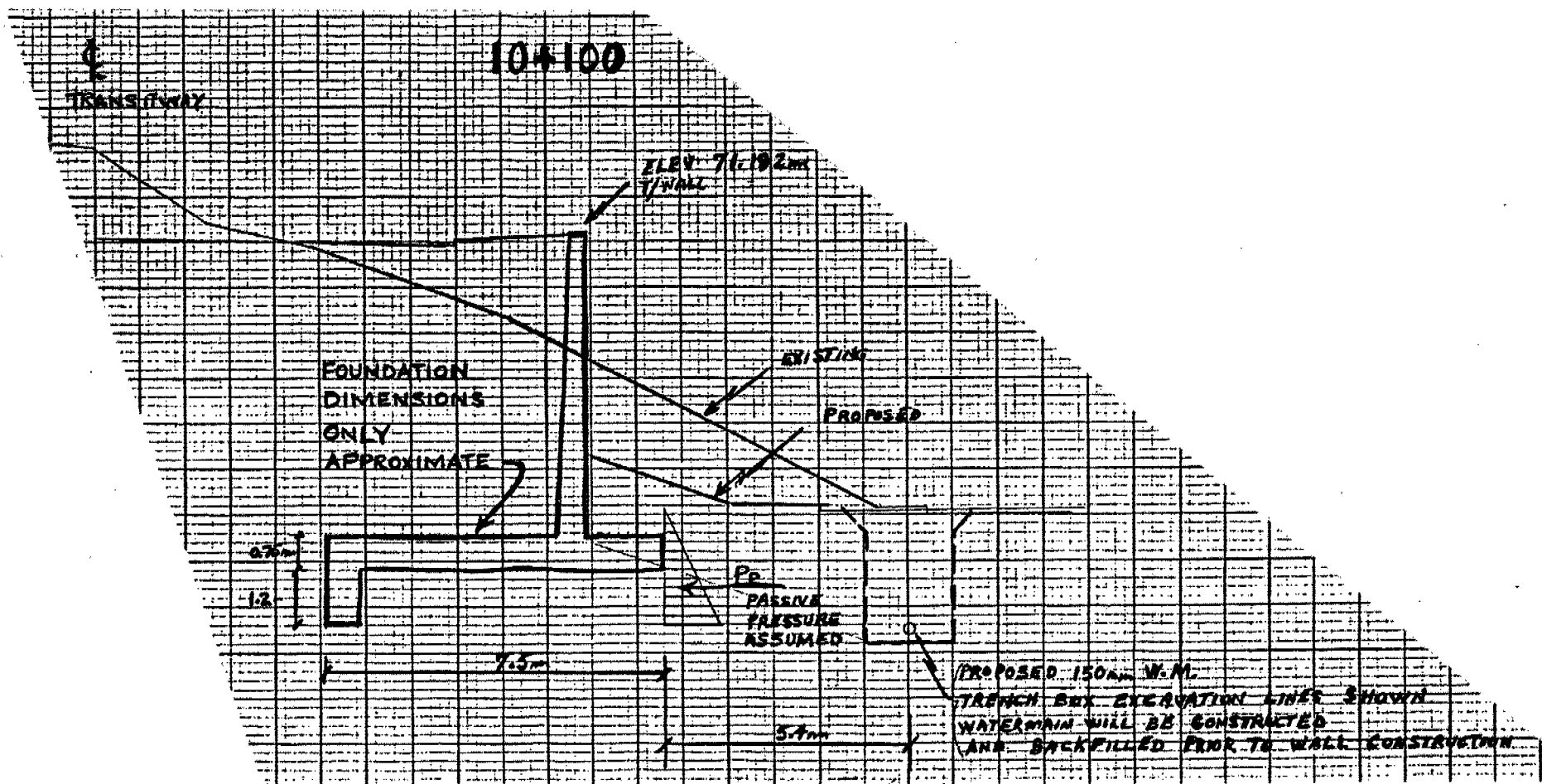
Test Pit No.  
91-9

Date :

JULY 19 1991

SOUTH EAST TRANSITWAY

ELEV. 70.47	DEPTH in metres	DESCRIPTION	REMARKS
70.27	0.20	FILL topsoil and clay	hand dug pit to ELEV 67.37m
69.47	-- 1 --	FILL silty sand and gravel	
68.47	-- 2 --		
67.47 67.37	-- 3 -- 3.1	FILL silty sand with a little gravel	bottom of pit hole advanced by hand auger
66.87	3.6	Bottom of hole	no water seepage
			Plate No. 10



McROSTIE GENEST ST-LOUIS  
& ASSOCIATES LTD. & ASSOCIÉS LTÉE  
CONSULTING ENGINEERS INGÉNIEURS CONSEILS  
OTTAWA CANADA

EFFECT OF WATERMAIN TRENCH

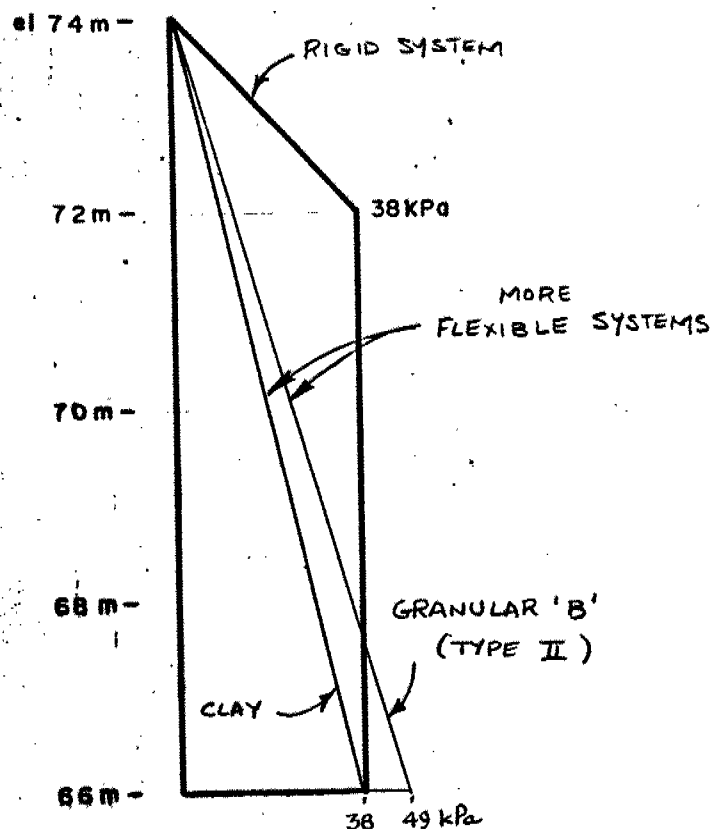
SCALE AS SHOWN

PLATE N° 11

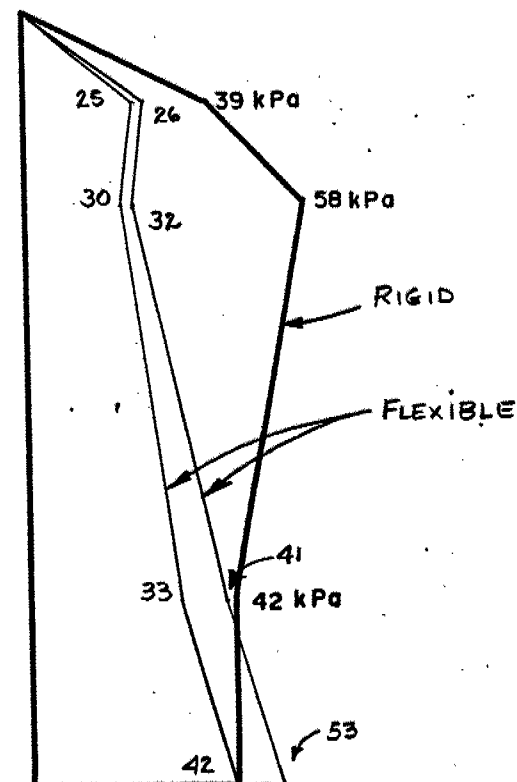
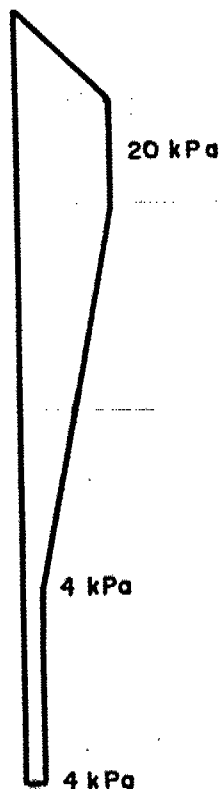
# EARTH PRESSURE

# E-85 RAILWAY LOADING

# TOTAL



TRIANGULAR  
DISTRIBUTION



**McROSTIE GENEST ST-LOUIS**  
**& ASSOCIATES LTD. & ASSOCIÉS LTÉE**  
 CONSULTING ENGINEERS INGÉNIEURS CONSEILS  
 OTTAWA CANADA

TEMPORARY SHORING  
 S.E. TRANSITWAY - STATION 10+250

SCALE AS SHOWN

PLATE N° 52 A

**PAST CORRESPONDENCE**

# McROSTIE GENEST ST-LOUIS

& ASSOCIATES LTD. — CONSULTING ENGINEERS — 1755 WOODWARD DRIVE, SUITE 201  
& ASSOCIÉS LTÉE — INGÉNIEURS CONSEILS — 1755, PROMENADE WOODWARD, BUREAU 201  
OTTAWA, ONTARIO K2C 0P9

TEL: (613) 228-7088  
FAX: (613) 228-0986

October 04, 1990  
Our reference SF-3062A

McCormick Rankin  
Associates Ltd.  
1540 Raven Avenue  
OTTAWA, Ontario  
K1Z 7Y9

Attention: Mr. A. Wing, P. Eng.

RE: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

Further to our recent meeting at your office, we would like to confirm certain recommendations pertaining to the above project.

The project will include several shelters (12 X 3m) on a concrete platform 6m wide. These platforms can be constructed as floating slabs but will need to be designed to accommodate differential settlements.

Differential settlements are somewhat proportional to the amount of backfill above the compressible clay layer. Our best estimate of the time required for these consolidation settlements to occur would be 25% after 3 months, 50% after 1 year with near completion after 5 years. Estimates of the magnitude of settlement where the backfill will be 4.5m to 5m above the present ground surface are given in section 3.11 of our report SF-3062.

Several parameters affect the amount of settlement that will occur under fill weight, including the thickness and density of fill, the thickness of the compressible clay as well as variations in soil strength and compressibility characteristics. Although, theoretically incorrect, a linear relationship from zero settlement at zero fill to the maximum values given above would be reasonable in judging the probable amounts of future settlements and their effect on surface drainage.

As discussed, some consideration could be given to localized excavations in clay, in order to get a more uniform thickness of granular fill and thus more uniform settlement.

There are shelters and platforms in a clay cut section that would result in exposing these underlying soils to a first cycle of freezing and thawing. Significant differential heaving would occur and special treatment would be necessary. We would recommend that subexcavation and replacement of the clay with non frost acting free drainage<sup>ing</sup> granular soils be made at these locations. A thickness of 1800mm is recommended.

In other locations, where the grade will not be changed significantly or will remain unchanged, some protection against frost heave will also be necessary even though the clays have been subjected to freeze-thaw cycles. These clays are also frost acting although they tend to heave somewhat less than deeper higher water content clays never exposed to these conditions.

Generally, frost protection can be accommodated by replacement or by means of an equivalent insulation scheme. The frost line can be assumed to be at 1800mm from grade throughout the project.

Of course, the selected granular fill placed below shelters and platforms should be drained to a suitable outlet.

Shelters 6 and 7 and others with geometries typical of these, are in a condition where no change of grade will occur at the front of the platform but a wedge of soil with a maximum height between 1.0 and 1.5m will be placed behind the platform. These soil wedges are not expected to induce more than 5 to 10mm of settlement behind the platforms.

Based on soil parameters obtained in the boreholes made at the site, including the statistical analysis of the variations in strength between boreholes, we recommend the following for adhesion under footings subjected to lateral loading conditions.

<u>ULS</u>	<u>SLS II</u>	
50kPa	100kPa	above el. 70m.
25kPa	50kPa	<del>above</del> el. 70m. below

The above are generalized parameters. Should the stability become critical using these values, further refinements would become necessary including review of closest borehole information.

We were requested to review the feasibility of anchoring the soldier pile and lagging wall system by means of a deadman block approach.

We feel that this temporary retaining system would be possible at the site. The resisting block could be attached at a depth of about 2500mm but should be set back at least 15m behind the face of wall. The H-piles should also be driven at least 5m below the bottom of excavation and would not be required to reach the till layer as no vertical downward force component results from this type of bracing system.



The deadman anchor block and the portion of pile below the base of the excavation will be under conditions of passive restraint. We recommend that the following passive earth pressure coefficients be used.

	<u>ULS</u>	<u>SLSII</u>
Kp	2.70	3.40

In accordance with the CFEM, the piles below the excavation can provide passive resistance over a distance equivalent to three (3) times the pile width.

Again in accordance with the CFEM, passive pressures should be computed using a coefficient of passive pressure reduced to account for movements required to develop these. We would recommend that a reduction factor of at least 2.0 and preferably 2.5 be applied to the Kp factors in order to reduce excessive movements in SLSII. Passive earth pressures considered as a resistance should be factored in accordance with section 6 of the OHBDC under ULS conditions.

Should any questions arise from the above, or should you require further information, we would be happy to discuss the details with you.

Yours very truly,



*Michael Allan*

M.W. St-Louis, P.Eng.  
McRostie Genest St-Louis  
& Associates Ltd.

MWS/evv

# McROSTIE GENEST ST-LOUIS

& ASSOCIATES LTD. — CONSULTING ENGINEERS — 1755 WOODWARD DRIVE, SUITE 201 TEL: (613) 228-7088  
& ASSOCIÉS LTÉE — INGÉNIEURS CONSEILS — 1755, PROMENADE WOODWARD, BUREAU 201 FAX: (613) 228-0986  
OTTAWA, ONTARIO K2C 0P9

November 19, 1990

Our Reference: SF-3062B

McCormick Rankin Associates Ltd.  
1540 Raven Avenue  
OTTAWA, Ontario  
K1Z 7Y9

Attention: Mr. A. Wing, P.Eng.

Re: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

Further to our recent meeting at your office, we would like to confirm certain recommendations pertaining to the above project, in view of certain site-specific conditions and restrictions.

Between Stations 10 + 300 and 10 + 334, where the height of fill to be retained by the north retaining wall reduces from about 2.1m to 0.2m, the overall stability of the wall/slope system can be assured by providing adequate frost cover only. The underside of footing at the extreme end of the toe for the retaining wall should be established from a line drawn perpendicular to the slope in front of the wall. In other areas, the general criterion given in our report SF-3062 should be followed to assure the overall stability of the wall/slope system.

The lateral stability of the retaining walls at the location of proposed utility pipes needs to be assured. The edge of trench needs to be at a minimum

distance such that it falls beyond an imaginary line drawn at two (2) horizontal to one (1) vertical units from the underside of footing as shown on the accompanying plate.

Between Stations 10 + 100 and 10 + 220, the use of a "Jersey" barrier wall is being considered at the toe of the slope south of the transitway route. The small wall could possibly be designed as a large curb without frost protection. In this case, the designers must be prepared to accept frost related movements. Otherwise, a suitably designed insulation scheme equivalent to a soil cover of 1800mm would be required.

Relatively steep backslopes are being considered behind these retaining walls (ie. 1.4:1 and 1.7:1) on the south side of the transitway route and special recommendations are needed to maintain the margins of safety of sloping ground within acceptable limits.

We understand that the entire wall will be backfilled with Granular 'B' (type II) material connected to an underdrainage system. Between Stations 10 + 100 and 10 + 220, partial removal and replacement of the clay soil will be required.

As a clay slope gets steeper, such as the inclinations being considered above, surface sloughing due to freezing/thawing and wetting/drying become more important. Furthermore, the effect of train loads on the steeper slopes has a greater impact on reducing the factors of safety.

Our analysis shows that by replacing the clay to at least 0.5m behind the retaining wall and into the slope cut at 1:1, the overall stability of the wall/slope/backfill system will have adequate margins of safety. Of course, this backfill wedge will need to be connected to an underdrainage layer, in order to reduce groundwater pressures within the slope.

Our analysis included a parametric study of both drained and undrained conditions with train loads present on a track with centreline at 1600mm from crest of slope. Some assumptions were necessary in order to analyse a 3-D load from the train wheel assembly using conventional 2-D computer techniques.

The steep backslope behind the retaining walls will result in a significant increase in earth pressure. The equivalent coefficients of active earth pressure recommended for this project are based on both theoretical and semiempirical methods for estimating the pressure of drained backfill on low retaining walls.

#### ACTIVE PRESSURE COEFFICIENT ( $K_a$ )

##### Granular 'B' (type II)

Backslope	SLS II	ULS
1.4:1	0.67	0.81
1.7:1	0.45	0.76

The earth pressure distribution can be assumed to be triangular with the direction of resultant parallel to the inclination of the backslope.

The coefficient of friction under the cast-in-place retaining wall resting on a layer of Granular 'B' (type II) backfill can be taken as follows:-

## COEFFICIENT OF FRICTION

Granular 'B' (type II)

SLS II	0.70
ULS	0.56

The use of an artificial erosion protection layer should be considered in the steep backslope areas, specially if plans call for a surficial layer of topsoil.

Special measures may be required to resist the large earth pressures. If the sliding resistance is found to be inadequate, the lateral forces acting on the wall could be reduced by the use of light weight fill. This light weight fill is apparently available from Canadian sources in the Hamilton area. We understand that the material has a unit weight of about  $11\text{kN/m}^3$  and is now being estimated at \$40.00/tonne on an Ottawa area MTO project.

Should any questions arise from the above, or should you require further information, we would be happy to discuss the details with you.

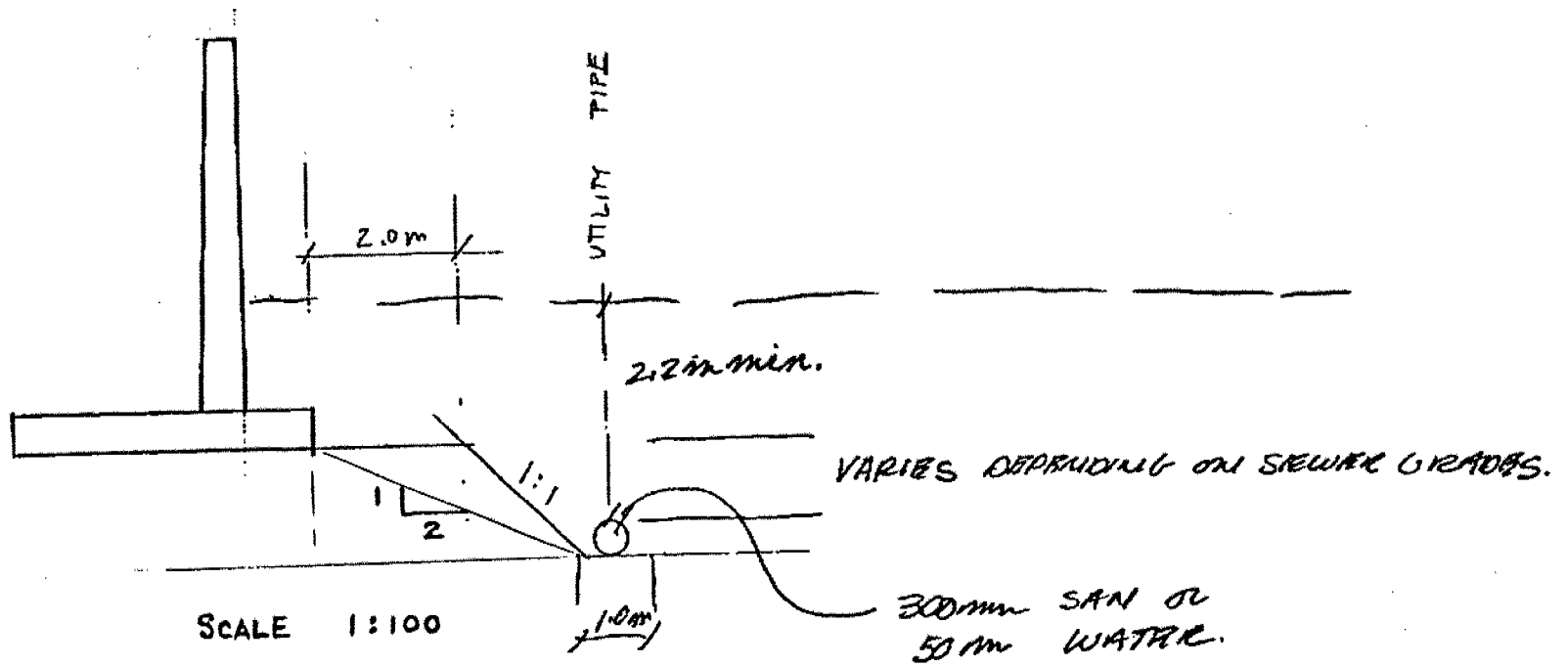


MWS/gl

Yours very truly,

A handwritten signature in cursive script, appearing to read "Michel Allard".

M.W. St-Louis, P.Eng.  
McRostie Genest St-Louis  
& Associates Ltd.



McROSTIE GENEST ST-LOUIS  
& ASSOCIATES LTD.  
CONSULTING ENGINEERS

Ste 201-1755 Woodward Dr., Ottawa, Ont. K2C 0P9

# McROSTIE GENEST ST-LOUIS

& ASSOCIATES LTD. — CONSULTING ENGINEERS — 1755 WOODWARD DRIVE, SUITE 201 TEL: (613) 228-7088  
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OTTAWA, ONTARIO K2C 0P9

December 31, 1990

Our Reference: SF-3062C

McCormick Rankin Associates Ltd.  
1540 Raven Avenue  
OTTAWA, Ontario  
K1Z 7Y9

Attention: Mr. A. Wing, P.Eng.

RE: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

Further to your recent request, we have carried out stability analysis of the wall/slope/backfill system between Stations 10+011 and 10+040 at the above site.

Boreholes made by ourselves and others at the site, indicate that the section presently under study is affected by significant quantities of unselected fill.

The lowering of the foundations for the retaining wall onto natural undisturbed soils would assure proper bearing and because of the depths involved would also assure adequate margins of safety for overall stability.

An alternative would be to subexcavate and replace the unselected fill on-site with compacted selected granular fill such as MTO - Granular 'B' (type II). The footings could then be made to bear on the selected fill layer at a depth where suitable frost protection is provided.

In order to provide lateral resistance, the backfill adjacent to and below the toe of the retaining wall should be extended at least 0.5 m from the edge of the footing and then sloped 1 on 1 (45 degrees) or flatter. On the side of the heel, the unselected fill can be removed to the natural soil and the backfilling made against an undisturbed face. The same bearing pressure can be used for both the natural soils and man-made backfill.

Should any questions arise from the above, or should you require further information, we would be happy to discuss the details with you.

Yours very truly,



A handwritten signature in cursive script, appearing to read "Michel Allou".

M.W. St-Louis, P.Eng.  
McRostie Genest St-Louis  
& Associates Ltd.

MWS/nd



# McROSTIE GENEST ST-LOUIS

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OTTAWA, ONTARIO K2C 0P9

January 31, 1991

Our Reference: SF-3062D

McCormick Rankin Associates Ltd.  
1540 Raven Avenue  
OTTAWA, Ontario  
K1Z 7Y9

Attention: Mr. M. Goetz, P.Eng.

RE: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

We have carried out stability analyses of the concrete toe wall/slope/backfill systems proposed at two (2) locations namely:- 10 + 110 to 10 + 160 and 10 + 260 to 10 + 340 (approximately). These concrete toe walls would separate the parking lot for the shopping centre from the transitway property.

Our analyses show that by replacing the clay and/or fill to at least 0.5 m behind the concrete toe walls and into the slope cut at 1:1, with Granular 'B' (type II) backfill connected to an underdrainage system, the overall stability of the wall/slope/backfill system will have adequate margins of safety.

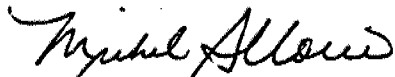
The applicable coefficients of active earth pressure for inclined backslopes are given in our report SF-3062. These are based on the assumption that adequate drainage would be provided behind retaining walls such that there would be no hydrostatic build-up.

A system of weep holes through the wall would not prevent groundwater accumulation behind the buried portion of the wall. Also, there would be a considerable reduction in friction below the wall affected by a partly buoyant condition.

The option of resorting to weep holes rather than an underdrainage system would require further analysis, if it became the only feasible solution due to site restrictions. Of course, underdrainage is preferred because of the positive impact on the stability of the wall/slope/backfill system.

Should any questions arise from the above, or should you require further information, we would be happy to discuss the details with you.

Yours very truly,



M.W. St-Louis, P.Eng.  
McRostie Genest St-Louis  
& Associates Ltd.

MWS/nd

# McROSTIE GENEST ST-LOUIS

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OTTAWA, ONTARIO K2C 0P9

February 26, 1991  
Our Ref: SF-3062D

McCormick Rankin  
& Associates  
1540 Raven Avenue  
Ottawa, Ontario  
K1Z 7Y9

Attention: Mr. Luke Foley, P.Eng

Re: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

Further to your recent request and our later telephone discussion, we would like to confirm certain comments and suggestions for the design of the north retaining wall near Station 10+155 at the above site.

We reviewed the undrained shear strength information in the closest boreholes and find the adhesion values recommended in our October 4<sup>th</sup>, 1990, letter to be representative of the area under study.

<u>ULS</u>	<u>SLS2</u>	
25kPa	50kPa	(below el. 70m)

We understand that a heel key is now being proposed under the footing in order to provide sufficient resistance against sliding. Apart from practical considerations, there is no need for a minimum key depth.

We recommend that the following passive earth pressure coefficients be used.

	<u>ULS</u>	<u>SLS2</u>
Kp	2.70	3.40

In accordance with the CFEM, passive pressures should be computed using a coefficient of passive pressure reduced to account for movements required to develop these. We would recommend that a reduction factor of 2.5 be applied to the Kp factors in order to reduce the movements to levels compatible with those to develop the adhesion given above.

The key should be poured tight against an undisturbed clay face without forms, again in an attempt to minimize movements. Passive earth pressures considered as a resistance should be factored in accordance with section 6 of the OHBDC under ULS conditions.

Should any questions arise from the above, we would be happy to discuss the details with you.

Yours Truly



M.W. St-Louis P.Eng  
McRostie Genest St-Louis  
& Associates Ltd.

MWS/jcj

# McROSTIE GENEST ST-LOUIS

& ASSOCIATES LTD. — CONSULTING ENGINEERS — 1755 WOODWARD DRIVE, SUITE 201 TEL: (613) 228-7088  
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OTTAWA, ONTARIO K2C 0P9

SF-3062 D

June 27, 1991

McCormick & Rankin  
Associates Ltd.  
1145 Hunt Club Road  
OTTAWA, Ontario  
K1V 0Y3

Attention: Mr. A. Wing, P.Eng.

RE: S.E. Transitway  
Bank Street to Grade Separation

Dear Sir,

Further to our recent telephone discussions and review of sketches related to temporary shoring for the above project, we would like to record the recommendations that we have discussed.

The earth pressure diagram on Plate No. 52 of our report SF-3062 is purposely conservative because of the restriction of limiting the settlements of the existing railroad to negligible amounts.

In order to increase the deadman anchor capacity by a significant amount, we suggest that they be deepened by about one (1) metre. At this depth they should be set back at least fifteen (15) metres behind the face of the wall.

The following soil parameters can be considered applicable for the clays encountered at the site.

- soil density 16.5 kN/m<sup>3</sup>
- passive earth pressure coefficient 3.40

There will be a few anchors in compacted Granular 'B' backfill and these can be expected to provide at least as much resistance as those placed in clay soils. We still recommend that a reduction factor of at least 2.0 be applied to passive earth pressures.

A sloped cut (1:1) will need to be made from the top of the anchor block for stability reasons. A similar cut will be needed on the exposed side of the anchor block.

Although there are some edge effects that would increase the available passive resistance, it is normal and conventional practice to design the anchors on the basis of the block area only.

Should any questions arise from the above, we would be happy to discuss the details with you.

Yours very truly,



M.W. St-Louis, P.Eng.  
McRostie Genest St-Louis  
& Associates Ltd.

MWS/nd

