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GEOTECHNICAL INVESTIGATION  
IONA SANITARY TRUNK SEWER EXTENSION  
SCENIC DRIVE TO ROYAL AVENUE  
HAMILTON, ONTARIO  
FOR  
THE REGIONAL MUNICIPALITY OF  
HAMILTON-WENTWORTH

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Job No. 91HF007

August, 1991

***PetoMacCallumLtd.***  
*C O N S U L T I N G   E N G I N E E R S*

August 6, 1991

Our Ref: 91HF007

Mr. P. Stumpo  
The Regional Municipality of  
Hamilton-Wentworth  
Department of Engineering  
71 Main Street West  
Hamilton, Ontario  
L8N 3T4

Dear Mr. Stumpo

Geotechnical Investigation  
Iona Sanitary Trunk Sewer Extension  
Scenic Drive to Royal Avenue  
Hamilton, Ontario

We are pleased to present the results of our geotechnical investigation for the proposed sanitary trunk sewer extension project briefly described above. Written authorization to proceed with this investigation was provided to Mr. L. Franco, Director of Administration, the Regional Municipality of Hamilton-Wentworth in a letter dated January 22, 1991.

The project involves the design and construction of a sanitary trunk sewer extending from the top of the Niagara Escarpment at Scenic Drive northerly to Iona Avenue and then east to Royal Avenue, in Hamilton, Ontario. An approximate 94 m deep drop shaft is to be constructed through the Niagara escarpment and connected by a tunnel under Hwy. 403 to an open cut section which begins about 125 m south of Iona Avenue.

Subsurface conditions at the drop shaft location consisted of approximately 2.8 m of silty clay overburden mantling dolostone bedrock. The bedrock stratigraphy at the borehole location was typical of the Niagara escarpment and comprised eleven geological formations consisting of interbedded carbonates, shales and sandstones. Queenston shale was encountered at 53.8 m depth below grade (elevation 136.6) at the test hole.

Subsurface conditions encountered in the boreholes within the tunnel/driving shaft location comprised surficial topsoil overlying clay/clay till mantling Queenston shale bedrock. The overburden thickness increased from south to north from 1.4 m at borehole 2 to 7.6 m at borehole 4.

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Variable clay, clay fill and topsoil layers were contacted surficially at the borehole locations within the open cut section. The fill and topsoil were generally underlain by silt and/or sand alluvium which overlies a clay/clay till unit. A discontinuous sand and/or clays and silts deposit was encountered locally below the upper units.

The level of free water could not be measured in tunnel/shaft section due to the use of drilling water during rock coring. However groundwater seepage through highly permeable zones of fractured bedrock must be anticipated. Free water was not identified in test holes 5 and 8, but it was noted at depths of 1.5 m in test holes 9 to 11 and at 4.5 m in test hole 6 during drilling. Upon completion of drilling free water was measured at depths of 1.1 to 2.4 m in boreholes 9 to 11 and at depths of 7.3 and 8.2 m in test holes 6 and 7.

It is envisioned that construction of the drop structure will involve conventional rock excavation/blasting techniques. Excavation of the driving shaft through the native clay/clay till should be straightforward. Excavation through the Queenston shale bedrock to the driving shaft will be more difficult requiring large excavation equipment and possible blasting and jack-hammering. The tunnel section connecting the two shafts should be constructed using a conventional tunnel boring machine suited for soft rock excavation and/or standard mining/blasting techniques. The Queenston shale in the tunnel and shafts is expected to converge about 40 mm following excavation, over a period of about one month.

Construction of the open cut section should be relatively straightforward and slopes may generally be excavated at inclinations of 1 horizontal to 1 vertical, locally flatter where loose/soft/wet soils are present, provided sufficient space is available.

Where space limitations and/or excavation depths do not permit construction of sufficiently shallow slopes, such as at locations adjacent to existing hydro towers, braced excavations will be required.

The standard concrete cradle or granular bedding requirement of The Regional Municipality of Hamilton-Wentworth should be satisfactory.

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The moisture contents of the native soils in the proposed open cut section are typically above optimum moisture content. Reuse of on-site soils for bulk fill purposes is considered feasible only if above normal post constructions settlements can be tolerated.

We trust this will be sufficient.

Sincerely

Peto MacCallum Ltd.



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

TJG:lh

5 cc: Client

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DRAWING 4 - Earth Pressure Diagram (Multi-Braced Cuts)

## **1.0 Introduction**

Peto MacCallum Ltd. has been retained by the Regional Municipality of Hamilton-Wentworth to carry out a geotechnical investigation for the proposed extension of the Iona sanitary trunk sewer.

The alignment of the proposed trunk sewer extends westerly along the Ontario Hydro right-of-way from the intersection of Royal Avenue and Bowman Street to the north-south Hydro corridor west of Brodick Street, and then southerly under Hwy. 403 to Scenic Drive at the top of the Niagara Escarpment, in Hamilton, Ontario. The relative location of the sewer alignment is shown on Drawing No. 1.

The section of sewer from Royal Avenue to about 150 m north of Hwy. 403 is to be constructed using conventional open cut trenching techniques. An approximate 100 m deep drop shaft and tunnel is planned to connect the existing sewer at the top of the escarpment with the proposed sewer installed using open cut techniques to within 150 m of Hwy. 403. Installation of the sewer in a tunnel bored through bedrock using conventional rock tunnelling design and equipment is planned.

The proposed sanitary trunk sewer will be 1200 mm in diameter in the open cut section and 4.4 m high by 3.1 m wide in the tunnel section. The diameters of the drop shaft and driving shaft have not been provided. The trunk sewer extends over a total length of about 1650 m.



The purpose of this study was to define the subsurface conditions and engineering properties of in situ soil/rock at the site in order to provide geotechnical engineering comments and recommendations pertinent to the design and construction of the proposed trunk sewer extension.

The scope of work and details of this investigation were defined in the proposal request prepared by the Regional Municipality of Hamilton-Wentworth, dated November 20, 1990. The study was expanded during the field work to enable more complete definition of the subsurface conditions and additional boreholes/rock probes were required in the vicinity of the driving shaft, north of the escarpment base.

Franklin Geotechnical Ltd. was retained to provide specialist advice pertinent to the rock mechanics aspects of the tunnel/shaft design.

## **2.0 Investigation Procedures**

The field work for this study, carried out during the period from January 30 to April 3, 1991, involved 11 test holes and 2 rock probes.

Boreholes 5 to 11, inclusive were advanced to depths of 8.1 to 11.1 m at selected locations along the open cut portion of the sewer that extends between Royal Avenue and the north-south Hydro corridor, west of Brodick Street on January 30 and 31, 1991. The test holes were drilled using continuous flight hollow and solid stem augers, powered by a track-mounted CME-55 drillrig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of our engineering staff.

Following the completion of these test holes, the field work was delayed pending approval from Ontario Hydro to carry out a subsurface investigation within the north-south Hydro corridor.

After this permission was obtained, the remaining test holes were completed during the period from March 18 to April 3, 1991. The test holes were advanced using a track-mounted Acker Soilmax (boreholes 1 and 3) and a CME-55 (boreholes 2, 4 and rock probes 1 and 2) drillrig.

Boreholes 1 to 4 were augered through the overburden to the bedrock surface and then continuously cored using NQ rotary diamond drilling equipment. Test holes 1 to 3 were advanced to between 2.7 and 4.7 m below the proposed invert elevation of the tunnel (15.4 to 99.2 m below existing grade).

Test hole 4 drilled at the location of the proposed driving shaft was terminated in shale bedrock at 10.3 depth, approximately 400 mm below the sanitary sewer invert elevation.

Sound bedrock was encountered 8.8 m below the ground surface (elevation 95.6) at borehole 4. Since the proposed level of the tunnel obvert at this location was about 98.4 m and the design requires that sound bedrock be present at least 2 m above the tunnel obvert, two rock probes, RP 1 and RP 2 were drilled at the request of R.V. Anderson Associates Limited to delineate the bedrock surface south of borehole 4 in order to determine a more suitable location for the shaft. The rock probes were advanced without sampling at locations selected by R.V. Anderson Associates Limited and Peto MacCallum Ltd. to depths between 6.3 and 7.2 m, and were terminated upon auger refusal on inferred bedrock. Borehole 3 was put down near this alternate shaft location.

The relative locations of all test holes and rock probes are indicated on the borehole location plan, Drawing 1, appended.

Representative samples of the overburden were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration, pocket penetrometer and field vane tests were conducted during drilling to assess the strength characteristics of the substrata. Continuous rock cores were obtained from the boreholes advanced below the bedrock surface. The groundwater conditions in the boreholes were closely monitored during the course of the field work.

It should be noted that about 40 m of drill rod could not be retrieved from test hole 1. This should be considered during design and installation of the drop shaft.

It should also be noted that during the advance of borehole 4, auger grinding occurred at about 6.9 m depth, 0.7 m above the bedrock surface. When the augers were pulled to permit sampling at a depth of 7.6 m the sound of flowing water could be heard coming from the base of the borehole indicating an underground service may be present at this depth/location. The drillrig was moved 3 m west and the test hole was completed.

A storm sewer was not shown on the underground service drawings provided to our office by the Region of Hamilton-Wentworth. We suggest a thorough search of available information be completed to determine the nature and extent of this possible underground service.

The programmed locations of the test holes were shown on the site plan and drawings provided by the Regional Municipality of Hamilton-Wentworth. The actual field locations of the boreholes were established by Peto MacCallum Ltd. as dictated by existing underground services and terrain restrictions. Test hole 2 was originally scheduled to be drilled in the grass median of Highway 403. However, MTO was hesitant to permit drilling on the highway median and consequently the borehole was moved to a location at the base of the north slope of the highway embankment to facilitate drilling. Ground surface elevations at the test holes were determined by a survey crew from the Regional Municipality of Hamilton-Wentworth.

### **3.0 Laboratory Testing**

All of the recovered soil samples were returned to our laboratory for detailed visual examination, classification, and routine moisture content determinations. The bulk unit weight of representative samples of the overburden soils was also determined. The test results are plotted on the appropriate Log of Borehole Sheet; the bulk unit weights are summarized on Table I.

Selected rock core samples from each geologic formation penetrated in borehole 1, and at least one core from the tunnel area in boreholes 2 and 3 were submitted to Franklin Geotechnical Ltd. for specialized testing of selected engineered properties including moisture content, dry unit weight, sonic velocity, uniaxial strength and secant/tangent moduli.

Select rock core samples were carefully preserved and waxed immediately following recovery from the borehole to prevent drying. Additional samples were requested by Franklin Geotechnical Ltd.; the subsequent samples had been preserved unwaxed in the core boxes.

The test procedures employed along with the test results are presented in Appendix A and summarized on the test hole logs.

#### **4.0 Site Description and Geology**

The route of the proposed sanitary trunk sewer crosses the Niagara Escarpment which forms a major topographic feature in this area. Consequently, significant variations in the general site description occur over relatively short distances.

The section above the escarpment at Scenic Drive is principally within an Ontario Hydro corridor. The ground surface is slightly undulating and grass covered.

The ground surface drops abruptly from about elevation 190.0 on top of the escarpment to 120.0 on the north edge of the Hwy. 403 embankment over a horizontal distance of about 125 m.

The ground surface extending north from the Hwy. 403 embankment to Iona Avenue and then easterly along the Hydro lands to Royal Avenue and Bowman Street is grass covered and gently slopes north and east with local undulations. The section immediately north of the Hwy. 403 embankment is more steeply inclined.

The escarpment is formed mainly of lower and middle Silurian carbonate sedimentary deposits. The upper portion is formed by a complex succession of dolomites interbedded with limestone and shales belonging to the Lockport Formation and Clinton Groups. Although the boundary varies through the region, the Caratact Group, consisting of interbedded sandstones, shales and limestone normally exists below elevation 165. The upper limit of the Queenston Formation is near elevation 135. The regional dip of these strata varies between one and two degrees below horizontal to the west or southwest.

The bedrock geology of the study area is detailed on Ontario Division of Mines Maps 2336 and 2343 titled "Palaeozoic Geology, Hamilton" and "Palaeozoic Geology, Grimsby", respectively.

According to the Ontario Department of Mines Map 2033, "Pleistocene Geology, Hamilton Area", surficial deposits below the escarpment are described as Halton clay or silt till; and/or offshore fine-grained sand and silty sand deposits of former glacial Lake Iroquois.

## **5.0 Subsurface Conditions**

### **5.1 GENERAL**

Reference is made to the appended Log of Borehole sheets for details of the soil and rock classifications, inferred stratigraphy, standard penetration test "N" values, pocket penetrometer and field vane shear strength test results, groundwater observations and the results of laboratory testing referenced in Sections 3.0.

A schematic overview of the bedrock type and overburden thickness along the tunnel alignment is presented on Drawing 2.

The stratigraphy along the sewer alignment is consistent with the geology of the area inferred from the geological maps.

## 5.2 TOP OF ESCARPMENT

Borehole 1 was advanced at the top of the escarpment.

An approximately 150 mm thick topsoil layer was encountered surficially in test hole 1. The topsoil was comprised of dark brown silty clay and judged to be low organic. Very stiff silty clay was present beneath the topsoil to 2.8 m below the ground surface; bedrock was encountered underlying the silty clay at this depth.

The bedrock lithology at the borehole location was typical of the Niagara Escarpment in the West Hamilton area and comprised interbedded Palaeozoic carbonate, shale and sandstone sedimentary strata. Eleven geological formations were identified in the rock core recovered from test hole 1, as detailed on the borehole log and summarized in the following table. The geologic formations are numbered sequentially from the bottom of the hole upward for ease of reference.

SUMMARY OF BEDROCK LITHOLOGY AND  
STRATA THICKNESS

11. LOCKPORT FORMATION

- a) Goat Island Member (10.3 m). Dolostone, aphanitic, with chert nodules, weathered on partings, close to moderately spaced discontinuities (50 to 90 mm), fair quality.
- b) Gasport Member (1.4 m). Dolostone, medium crystalline, close to wide spaced discontinuities, (50 to 3,000 mm), excellent quality.

10. DECEW FORMATION (2.5 m). Dolostone, good quality.

9. ROCHESTER FORMATION (1.0 m). Interbedded shale and limestone, excellent quality.

8. IRONDEQUOIT FORMATION  
(1.5 m). Limestone, medium crystalline, excellent quality.

7. REYNALES FORMATION  
(2.6 m). Dolostone, with thin shale layers, good to excellent quality.

6. THOROLD FORMATION  
(2.6 m). Sandstone with layers of shale, good quality.

5. GRIMSBY FORMATION  
(6.1 m). Interbedded sandstone and shale, massive to thinly bedded, good quality.

4. CABOT HEAD FORMATION  
(18.5 m). Shale, low to medium strength, poor to fair quality.

3. MANITOULIN FORMATION  
(1.2 m). Dolostone interbedded with shale, good quality.

2. WHIRLPOOL FORMATION  
(3.4 m). Sandstone, massive to finely bedded, fair quality.

1. QUEENSTON FORMATION  
(greater than 45.4 m). Red shale with green mottling, calcite infilling, typically fair to good quality, excellent between 80.9 and 93.0 m depth.



Some thin isolated clay seams and chert/calcite lined vugs were noted in the upper carbonate deposits and are likely a result of infilling of voids and/or fractures.

Numerous zones of relatively weak shale which alter rapidly to a clay state on exposure, were encountered within both the Cabot Head and Queenston formations.

The Rochester formation was highly fractured.

In general, the in situ moisture contents of the rock core samples from borehole 1 ranged from 1.3 to 5.5 percent. Moisture contents of the rock core stored in the core boxes ranged from 0.2 to 0.7. Dry unit weights for these specimens were between 2.0 and 3.0 g/cm<sup>3</sup> and uniaxial strength varied from 7.0 MPa for the Cabot Head shale to 114.0 MPa for the DeCew dolostone.

Refer to the test hole logs and Appendix A for more detailed description of the rock properties.

### 5.3 BASE OF ESCARPMENT - NORTH OF HIGHWAY 403

Test holes 2 to 4, and rock probes 1 and 2 were advanced between of the north limit of the Highway 403 embankment and the proposed driving shaft location.

In general, the subsurface conditions encountered at the test hole locations comprised surficial topsoil overlying clay/clay till mantling Queenston shale bedrock. The overburden thickness increased from south to north from 1.4 m at borehole 2 to 7.6 m at borehole 4.

The thickness of the surficial topsoil was between 150 to 200 mm. This material comprised dark brown silty clay, judged to have a low organic content.

The topsoil was underlain by firm to very stiff, silty clay/clay till with trace to some sand. The clay/clay till was judged to be low to medium plastic, at or wetter than the plastic limit.

The bedrock surface dips to the north and weak shale bedrock was contacted below the clay/clay till at depths between 1.4 to 7.6 m below the existing ground surface. Auger refusal on sound shale was encountered at 1.7 to 8.8 m depth (El. 118.3 to 95.6) in the test holes. The following table summarizes the ground surface and bedrock surface elevations as well as auger refusal elevations at the test holes and rock probes.

<u>Test Hole/ Rock Probe No.</u>	<u>Ground Surface Elevation</u>	<u>Bedrock Surface Elevation</u>	<u>Elevation of Auger Refusal/ Sound Bedrock</u>
BH 2	120.00	118.6	115.3 *
RP 2	109.63	104.3	102.4
BH 3	106.02	102.7	102.7 *
RP 1	106.29	100.8	100.0
BH 4	104.39	96.7	95.6

\* Elevations of sound shale inferred from core recovery/RQD

Rotary core drilling was carried out to advance the three boreholes below the zone of auger refusal to depths of 28.0, 15.4 m and 10.3 m in boreholes 2, 3 and 4, respectively. The rock quality was generally fair to good improving to good/excellent with depth. A localized zone of very poor quality Queenston shale was encountered

in borehole 3 within the proposed tunnel section between 9.3 and 10.8 m below grade (elevation 96.7 to 95.2).

The in situ moisture contents of Queenston shale core specimens from boreholes 2 and 3 ranged from 1.7 to 4.7 percent; the moisture content of core samples stored in the core box ranged from 0.47 to 2.55. The dry unit weight varied between 2.21 and 2.56 g/cm<sup>3</sup> and uniaxial compressive strength test results ranged from 8.6 to 29.2 MPa.

#### 5.4 OPEN CUT SECTION, WEST OF BRODICK STREET TO ROYAL AVENUE

Boreholes 5 to 11 were located along the open cut section of the sanitary truck sewer, on Iona Avenue and the Hydro right-of-way from Brodick Street to Royal Avenue. These test holes were advanced to the depths scheduled by the Regional Municipality of Hamilton-Wentworth, that ranged between 8.0 and 11.1 m below existing grade.

Variable clay, clay/fill and topsoil layers were contacted surficially at the borehole locations. The fill and topsoil were generally underlain by silt and/or sand alluvium which overlies a clay/clay till unit. A discontinuous layered sands and/or clays and silts deposit was encountered locally below the upper units.

Surficial clay fill was present at test holes 6, 7 and 9 to 11. This fill comprised a firm to stiff, silty clay with trace to some sand, and was judged to be low to medium plastic. In situ moisture contents varied from 21 to 30 percent, and the thickness of the fill ranged from 1.0 to 2.1 m.

Topsoil was contacted surficially in borehole 8. This unit was about 100 mm thick and comprised dark brown clayey silt, judged to have a low organic content. An approximate 300 mm thick topsoil layer was encountered locally below the surficial clay fill in borehole 6. An obvious topsoil layer was not identified in test hole 5.

Stiff silty clay with some sand was present in test hole 5 from the ground surface to 1.4 m depth.

The surficial materials were underlain by variable layered sands/silts/clays, sand and/or silt alluvium and native clay.

Silt and/or sand alluvium was present below the topsoil in borehole 6 and beneath the clay fill in boreholes 9 and 11. This material comprised very loose, variable sand/silt and thicknesses ranged from 300 to 800 mm. The in situ moisture content of one sample was 18 percent.

Layered sands/silts/clays were encountered below the alluvium in test holes 6 and 9, and below the surficial clay in borehole 5. These units comprised interlayered loose brown silty fine sand, loose to compact brown to grey silt and soft to stiff brown to grey silty clays. Thicknesses ranged from 1.0 to 2.8 m and moisture contents were typically between 22 and 26%.

A clay deposit was identified below the upper soil units except test hole 9. This unit was present below the layered sands/silts/clays in boreholes 5 and 6, clay fill in boreholes 7 and 10, surficial topsoil in borehole 8, and sand alluvium in borehole 11. The clay was generally stiff to very stiff, becoming stiff to firm with depth; locally very stiff to hard becoming very stiff to stiff in borehole 10. The undrained shear strength of

the upper 1.5 m of this deposit measured using a pocket penetrometer, typically ranged from 100 to 200 kPa. In situ vane shear strength test results conducted in the lower portion of this unit ranged from 50 to 75 kPa.

The in situ moisture content for this material was typically 25%, but ranged from 18 to 35% and generally increased with depth. The thickness of the clay layer varied from 3.9 to 6.1 m in test holes 5, 7 and 8, where the unit was fully penetrated. Boreholes 6, 10 and 11 were terminated in the clay at depths between 8.1 and 11.1 m.

Clay till was contacted underlying the clay unit in test holes 5, 7 and 8 and below the layered silts and clays in test hole 9. The till comprised silty clay with trace of sand and was judged to be medium plastic. The consistency of the clay till was generally firm to stiff, locally hard becoming very stiff to firm with depth in borehole 9. Undrained shear strengths of this material determined by field vane tests varied from 5.0 to 100 kPa. In situ moisture contents for this unit were typically 22 to 26% with locally lower values in test hole 5 and higher in test hole 8.

Boreholes 5 and 7 to 9 were terminated in the clay till at depths between 9.0 to 11.1 m.

None of the test hole locations programmed by the Regional Municipality of Hamilton-Wentworth were within the existing roadway sections crossed by the proposed sewer route. Therefore the depth of asphalt, concrete and granular material at these locations could not been measured during this study. Further field work, including coring of the roadway, is required to provide this information.

Free water was not identified in test holes 5 and 8 during drilling. It was noted at depths of 1.5 m in test holes 9 to 11 and at 4.5 m in test hole 6 during drilling. Upon completion of drilling free water was measured at depths of 1.1 to 2.4 m in test holes 9 to 11 and at depths of 7.3 and 8.2 m in test holes 6 and 7. The water level in the standpipe installed in borehole 10 was measured at 1.8 m depth on February 21, 1991. It is considered that the water is perched in the upper layered clays and silts/clay fill/sand alluvium above the clay/clay till over the eastern portion of the proposed alignment (boreholes 9 to 11).

Stabilized groundwater levels have not been determined and varying levels may be encountered during construction.

## **6.0 Engineering Considerations**

### **6.1 GENERAL**

Reference is made to the Regional Municipality of Hamilton-Wentworth Department of Engineering drawings for the Iona Sanitary Trunk Sewer, sheets 1 to 6, inclusive for details of the proposed sanitary sewer, drop structure and tunnel. As currently proposed, the sanitary trunk sewer will drop some 94 m through a drop shaft to invert elevation 96.3 m and then proceed northerly in tunnel section for approximately 370 m beneath the face of the escarpment, Highway 403, and the north-south Hydro corridor. The tunnel dimensions are to be 4.4 m high and 3.1 m wide and the tunnel invert is about 500 mm below the sewer invert. It will terminate at an approximate 10.5 m deep driving shaft to be constructed about 125 m south of Iona Avenue near the location of test hole 4 (MH 10). The diameters of the two shafts were not been provided.

From the north portal of the tunnel, a 1200 mm diameter sanitary sewer will follow northerly to Iona Avenue and then along the east-west Hydro right-of-way north of Iona Avenue terminating at Royal Avenue/Bowman Street. The planned length of this section is about 1270 m and invert elevations range from 94.4 m at the junction with the tunnel section to 88.6 m at the east end of the sewer.

## 6.2 SHAFTS AND TUNNEL

Based on the stratigraphy and bedrock lithology encountered at borehole 1, the subsurface conditions to the invert of the 94 m deep drop structure will comprise 2.8 m of silty clay over the various lithological units of the Niagara Escarpment between dolomite of the Goat Island member of the Lockport Formation to shale of the Queenston Formation.

The current design calls for the invert of the tunnel to drop from elevation 95.8 at the bottom of the drop shaft to 94.0 at the driving shaft. The sewer invert elevation drops from 96.3 to 94.4 along this section. Boreholes 2, 3 and probe holes 1 and 2 indicate the tunnelling operation will be carried out entirely within Queenston Shale provided it is terminated south of borehole 4.

The bedrock surface was contacted at elevation 96.7 in test hole 4. The design obvert elevation of the tunnel at this borehole location is 98.4, therefore the tunnel would be partially constructed in soil if advanced to the design location about 125 m south of Iona Avenue. We were advised by R.V. Anderson Associates Limited that the entire tunnel should be excavated in rock, with at least 2 m of competent bedrock above the tunnel obvert to

satisfy the present design requirements. The bedrock surface at test hole 4 was considered to be too deep to accommodate the intended tunnel and driving shaft construction.

The bedrock surface and sound bedrock elevations at borehole 3 are 102.7 and 101.3, respectively. The tunnel obvert is shown to be 98.7 on the design drawings, thus the requirement that at least 2 m of competent bedrock is present above the tunnel obvert appears to be satisfied at this location.

From our conversations with R.V. Anderson, we understand the driving shaft will be located near borehole 3. At this location approximately 3.3 m of clay/clay till overburden is anticipated to be encountered above Queenston Shale bedrock.

#### 6.2.1 EXCAVATION AND GROUNDWATER CONTROL

Excavation through the native clay/clay till overburden at the driving shaft location should be straightforward. Sidewalls in the soil may be sloped at an angle of 1 horizontal to 1 vertical if space permits. If steeper side slopes are required due to the space restrictions, then adequately designed and constructed temporary shoring will be required.

Excavation through the Queenston Shale bedrock to the invert elevation of the driving shaft will be more difficult requiring large excavation equipment and possibly implementation of standard rock excavation methods such as blasting and jack-hammering. The actual equipment required and method of excavation within the bedrock will be somewhat dependent upon the geometry of the cut and relative depth of excavation into the bedrock. The rock excavation should be carried out such that fracturing of the bedrock surface on which the proposed service is to be founded is minimized.



It is anticipated that construction of the 94 m deep drop shaft will also require blasting to advance the excavation, particularly through the competent shale, dolostone, limestone and sandstone formations.

Excavation/blasting of the shaft upwards from the base may be the most expeditious method of constructing the drop shaft but other techniques such as downward excavation with blasting or with rock coring equipment are possible. The preferred construction techniques will primarily be dictated by construction constraints, worker safety and economic considerations. Input from a specialist excavation contractor is recommended during the selection process.

Blasting must be carefully controlled to minimize overbreak. A preconstruction survey of the residential structures near the two shaft locations is recommended if blasting is carried out. Blast monitoring is recommended during construction to determine whether charges should be reduced or maintained based on accepted damage criteria or if a different excavation method should be utilized. The tunnel section between the two shafts may be excavated using a conventional tunnel boring machine suited to soft rock excavation and/or conventional mining/blasting techniques.

Alternate techniques for advancing both the shafts and horizontal tunnel may prove acceptable but should be reviewed by our office prior to construction.

It is recommended that monitoring of rock conditions be carried out during construction. The rock conditions in the shaft should be logged by an experienced and qualified engineering geologist.

The Queenston shale has a relatively low durability and should be protected from drying out by the application of shotcrete (fibrecrete) or similar material immediately following excavation.

Normally, the shale should stand unsupported. However discontinuous limestone/sandstone layers and/or major bedding planes may be encountered which could result in local instability. Since the pattern of such bedding can change rapidly along the line of the tunnel, any temporary support system would best be selected and modified by experienced construction personnel as the heading advances.

The shotcrete liner should adequately serve as the primary tunnel liner and permit controlled displacements and arching to develop. Additional support may be provided by installing rock bolts in conjunction with wire mesh and shotcrete or similar means where considered necessary, such as in major zones of weakness, jointing, along seams, hard bands, etc., particularly at the crown of the tunnel.

As mentioned, it is recommended that the primary shotcrete liner be applied daily to keep pace with the excavation.

The shale is expected to converge as a result of the concentration of stresses around the tunnel. Detailed comments and design recommendations in this regard are presented in the following section.

Monitoring of the convergence of the sides of the shaft and tunnel should be carried out at a number of points for as long a period of time as practical. If the in situ measurements during construction indicate greater movements than anticipated, the design of the structure should be reconsidered, cognizant of the actual movements.

The final liner should not be installed until the tunnel is completed and convergence appears to have stabilized based on the monitoring data.

It would be prudent to place a layer of compressible material between the rock surface and the permanent liner. This material may consist of non-compacted dry clay, styrofoam or sprayed foam. Alternatively, an open cavity can be utilized.

The presence of water bearing zones in the rock was not detected in the boreholes drilled along the shafts/tunnel section, primarily because of the presence of drilling fluid used during coring of the rock.

Essentially all of the water employed during coring was recycled, a significant loss of drilling water was not observed. In addition, sample recovery and the relatively high RQD values suggest that the bedrock in the shafts/tunnel section is not generally highly fractured. Therefore control of groundwater in the drop shaft and tunnel is not expected to be a major concern.

It should be noted however that seepage of water onto the face of the escarpment is often evident during wet spring conditions. Hence, some seepage is to be expected.

The potential for local relatively high concentrations of groundwater inflow into the shaft/tunnel should not be overlooked. During construction local permeable zones may be encountered in discontinuities/fracture zones such as the zone of very poor rock quality encountered in borehole 3 between 9.3 and 10.8 m below grade. Stress relief during excavation may also cause some seams to open up and permit seepage along bedding plains and joints. It

is possible that the 'flowing water' observed in test hole 4 is a void in the rock.

Low volume seepage should be adequately handled using conventional sump pumping techniques. High volume seepage may require pressure grouting within the more permeable zones as described in section 6.4.3.

#### 6.2.2 DESIGN CONSIDERATIONS

The following design considerations are provided as per the requirements presented in the Regional Municipality of Hamilton-Wentworth letter dated November 20, 1990. These responses are based on the research and test results completed by Franklin Geotechnical Ltd. The detailed findings of this work are provided in Appendix A and B.

- . The Modulus of Deformation for the rock is assumed to be 10 GPa.
- . Poisson's Ratio for the rock is assumed to be 0.3.
- . Convergence in the shotcreted tunnel, the driving shaft and the lower portion of the drop shaft which is through Queenston shale is estimated to be about 40 mm, occurring over a period of approximately 1 month, with reduced rates following that time. Much less convergence is expected for hard dolostone, limestone and sandstone beds in the portion of the drop shaft above the Queenston shale (above elevation 136.6). Convergence of the Cabot Head shale is expected to be about 30 mm, occurring over about 1 month. The rates of tunnel liner displacements should be similar to these shown on Figure 2 of Report

G678.2, presented in Appendix B. In addition to long-term convergence, local movements of about 25 mm should be anticipated for the tunnel and shaft walls due to the presence of unfavourably oriented joints or discontinuities.

- . A pre-excavation horizontal stress of between 2 MPa and 7 MPa is expected, accompanying a vertical stress of 1 MPa. The stresses should be magnified by a factor of about 2 times in the vicinity of the tunnel. Hence, the levels of stress may locally approach or exceed the uniaxial compressive strength of the shale which is estimated at between 10 to 16 MPa. This overstressing should present little or no problem provided the shale is shotcreted. A plastic yield zone may develop, although the thickness of this zone should be limited to about 1 m if the shale is confined by appropriate primary support.
- . An allowable rock bearing pressure of 5000 kPa may be used for conventional footing foundations founded on the Queenston shale at the proposed shaft invert elevations between 95.8 and 94.0 m.

### 6.3 OPEN CUT SECTION

Based on the subsurface conditions encountered in boreholes 4 to 11, inclusive, excavation to the proposed invert elevations along the open cut section of the sanitary trunk sewer should encounter variable fill/topsoil/silt/sand over clay/clay till, and locally shale near test hole 4. Invert levels typically range from 10 m at the tunnel portal at the west end of the project to 5 m at the east end.

#### 6.3.1 EXCAVATION AND GROUNDWATER CONTROL

Excavation through native soils to the assumed invert depths should be relatively straight forward using conventional equipment. Where sufficient space is available open cut procedures may be used to install the underground services. Based on a simplified slope stability analysis, sidewalls in the very stiff, locally hard, to firm clay/clay till may be excavated at a slope of 1 horizontal to 1 vertical. A detailed stability analysis is recommended for slopes in the stiff to firm clay/clay till with slope heights greater than 5 m when design/construction details are finalized. Flatter side slopes may be necessary where the very loose to loose sand/silt alluvium and layered sand/silt/soft clay, are present, or if localized seepage zones are encountered.

We expect that braced excavations will be required adjacent to existing hydro towers and at all other locations where space limitations and/or excavation depths do not permit construction of sufficiently shallow slopes. Methods for calculating earth pressures and general design recommendations for singly and multi-braced excavations are provided on Drawing No 1 and 2. Rigid supporting walls should be constructed adjacent to settlement sensitive structures. Preliminary calculations indicate that base heave in deep braced excavations along the proposed sewer alignment should not be a problem, however, these findings should be confirmed when designs are completed for the open cut section.

All work should be carried out in accordance with The Occupational Health & Safety Act, 1981 and with local regulations.

#### 6.3.2 PIPE BEDDING

The founding soil at the proposed invert elevations is anticipated to be very stiff to firm clay/clay till/shale. In general, no problems are expected with respect to bearing capacity or settlement.

An allowable soil bearing pressure of 100 kPa is available for manholes/drop structures founded on the firm to stiff silty clay/clay till.

The standard concrete cradle or granular bedding requirement of The Regional Municipality of Hamilton-Wentworth should be satisfactory. It may be necessary to increase the bedding thickness if construction softened clay/silt is present at the pipe subgrade. The need for this is best determined during construction.

#### 6.3.3 TRENCH BACKFILL

The granular bedding material should be carried up as backfill for at least 300 mm above the pipe obvert and compacted to at least 95% Standard Proctor maximum dry density.

If significant post construction settlement of the ground surface at the top of the trench cannot be tolerated, it will be necessary to ensure that the remainder of the trench backfill material comprises approved material placed in uniform 200 mm thick lifts within 2 to 4% of the optimum moisture content and compacted to at least 95% Standard Proctor maximum dry density.

The natural moisture content of the clay/clay till soils indigenous to this site typically ranges from 20 to 25%, locally 35%. We expect that a water content of about 20% is the upper limit at which this type of soil can be placed and compacted expeditiously. Above optimum moisture contents were also measured within the sands and silts. Therefore, we consider that the excavated soils will be suitable for reuse in the trench only if post construction ground surface settlements in excess of the normal 100 mm are tolerable and/or the work is carried out during the dry summer months. The construction schedule should be suited to provide for air drying to reduce the moisture content closer to the optimum value for efficient compaction. Additionally, these materials must remain free of organics and other deleterious materials.

If insufficient quantities of backfill are available on site or if the construction schedule does not provide adequate time for air drying wet materials, it will be necessary to use select imported material as trench backfill. The imported fill ideally should comprise silty clay/clayey silt fills in order to match the excavated soils as close as possible, and thus reduce the impact of construction on the local groundwater table by minimizing changes to the subsurface stratigraphy.

The pavement granulars may be used for reconstruction of the roadways provided they meet the OPSS Granular "A" gradation requirements and are appropriately stockpiled.

All asphalt, topsoil and other deleterious material should be selectively excavated and disposed of off-site.



Where the sewer trench crosses the existing roadways, attention must be given to the construction operations, particularly when native materials are used as trench backfill in order to reduce and render more uniform post construction settlements and to minimize any detrimental effects to the roadway pavement.

Should construction extend to the winter season, particular attention should be given to ensure that frozen material is not used as trench backfill.

Frequent inspection should be carried out by Peto MacCallum Ltd. geotechnical personnel to examine and approve backfill materials and to carefully inspect placement and verify the compaction by in situ density testing using nuclear gauges.

#### 6.3.4 pH AND SULPHATE CONCENTRATIONS

The results of sulphate content testing on two (2) soil samples one (1) from borehole 6 and one (1) from borehole 8, indicate concentrations of 0.029 and 0.046%, respectively. pH levels were measured to be 8.4 for both samples. These results indicate a negligible degree of attack on concrete. For recommendations regarding protective measures, reference is made to CSA Standard A.23.

#### 6.4 GROUNDWATER

##### 6.4.1 GROUNDWATER LEVELS

As previously mentioned, site specific water levels are not available for boreholes 1 to 4 due to the need to use drilling water to core the rock. Evidence of groundwater seepage on the face of the escarpment above Hwy. 403 is often noted during the

winter/spring season. In addition significant flow of groundwater through discontinuities in the rock is possible.

Our review of MOE water well records compiled from 1949 to 1979 for the area at the top of the escarpment, near borehole 1, concluded that most wells in the area are supplied by an artesian aquifer located within the upper dolomitic/limestone bedrock at the site. The water table was contacted at depths between 7.0 to 21.0 m and static levels were about 1.0 to 11.5 m below the ground surface at the well locations.

Within the open cut section observed groundwater levels ranged from 1.7 to 8.2 m depths with a distinct change evident near Leland Avenue. The water table was contacted about 3.0 to 3.5 m above the proposed invert elevation at test holes 9, 10 and 11 and 0.5 to 2.5 m below the invert level at test holes 6 and 7. Free water was not encountered in test holes 5 and 8.

#### 6.4.2 ANTICIPATED IMPACT OF CONSTRUCTION ON THE GROUNDWATER TABLE

Construction of the drop shaft could cause downward flow of water from the upper aquifer in the permeable dolomitic/limestone bedrock to lower zones. Left unchecked during the construction period or if allowed to develop following construction, this could effectively lower the local groundwater table.

Tunnelling operations through the relatively impermeable Queenston shale should have no significant impact on the local groundwater table. Anticipated minor seepage should be adequately controlled by conventional sump pumping techniques.

Construction of the portion of the driving shaft through overburden soils and the open cut portion of the sanitary trunk sewer may require temporary construction dewatering by conventional sump pumping, particularly along the east section of the underground service where invert elevations are approximately 3.0 to 3.5 m below the local groundwater level. The actual impact of construction dewatering on groundwater levels will depend on the quantity of water pumped and the hydrogeologic characteristics of the soil. Cognizant of the anticipated short construction period, it is anticipated that only a minor lowering of the groundwater table will occur.

The installation of the sanitary trunk sewer should have little impact on the stabilized (long term) groundwater levels provided native soils are reused as backfill. It is expected that impact of the installation of the trunk sewer on groundwater levels in the area will be minimized due to the presence of the existing combined box sewer which parallels the proposed sanitary sewer at an invert elevation about 1.5 m higher than the current trunk service design.

#### 6.4.3 MITIGATION OF IMPACT

The use of a grout curtain or steel liner may be required to control groundwater ingress during construction of the drop shaft. An inventory of operating wells should be completed and special care should be exercised to ensure adjacent wells in the vicinity of the shaft are not adversely affected by pumping operations.

To prevent continuous leakage from the upper zone, it is recommended that the void between the drop shaft liner and the bedrock be fully grouted from the bedrock surface (elevation 187.4) to the upper boundary of the first major shale unit (elevation 173.4).

The trench backfill for the sanitary sewer should comprise native clay/clay till or similar imported material in order to minimize the change to local subsurface conditions and thus mitigate the impact of the sewer construction on the local groundwater table. Flow along the base of the sewer trench can be reduced through the use of a concrete cradle in place of granular bedding material.

If these measures are implemented, installation of the sewer is expected to have minimal impact on the groundwater table.

We trust the information presented in this report is sufficient for your present purposes. If you have any questions, please do not hesitate to contact our office.

Yours very truly,

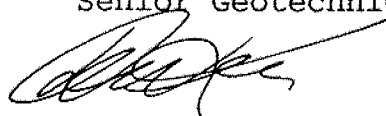
Peto MacCallum Ltd.



  
Timothy J. Garde, P.Eng.  
Senior Geotechnical Engineer

TJG:lh



  
Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

Our Ref: 91HF007

**TABLE I**

**Iona Sanitary Trunk Sewer Extension**

**The Regional Municipality of  
Hamilton-Wentworth**

**Bulk Unit Weights of Soil Samples**

<u>Borehole No.</u>	<u>Depth (m)</u>	<u>Unit Weight (Mg/m<sup>3</sup>)</u>	<u>Soil</u>
5	4.6-5.1	2.08	Clay
5	7.6-8.1	2.13	Clay Till
6	6.1-6.6	2.23	Clay
6	7.6-8.1	2.06	Clay
7	4.6-5.1	2.09	Clay
7	7.6-8.1	2.11	Clay Till
8	6.1-6.6	2.20	Clay Till
9	6.1-6.6	2.09	Clay Till
10	4.6-5.1	2.17	Clay
10	6.1-6.6	2.21	Clay
11	4.6-5.1	2.08	Clay

## LIST OF ABBREVIATIONS

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N', - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 0.3m INTO THE SUBSOIL. DRIVEN BY MEANS OF A 63.5kg HAMMER FALLING FREELY A DISTANCE OF 0.76m.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS. 0.3m INTO THE SUBSOIL. THE DRIVING ENERGY BEING 475 J PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3 m</u>	<u>c kPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3 m</u>	
VERY SOFT	0 - 2	0 - 12	VERY LOOSE	0 - 4	
SOFT	2 - 4	12 - 25	LOOSE	4 - 10	
FIRM	4 - 8	25 - 50	COMPACT	10 - 30	
STIFF	8 - 15	50 - 100	DENSE	30 - 50	
VERY STIFF	15 - 30	100 - 200	VERY DENSE	> 50	
HARD	> 30	> 200			
W.T.P.L.	WETTER THAN PLASTIC LIMIT		D.T.P.L.	DRIER THAN PLASTIC LIMIT	
	A.P.L.		ABOUT PLASTIC LIMIT		

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W	THINWALL OPEN
W.S	WASHED SAMPLE	T.P	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S	OESTERBERG SAMPLE
A.S	AUGER SAMPLE	F.S	FOIL SAMPLE
C.S	CHUNK SAMPLE	R.C	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		
	P.H	SAMPLE ADVANCED HYDRAULICALLY	
	P.M	SAMPLE ADVANCED MANUALLY	

### SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL		

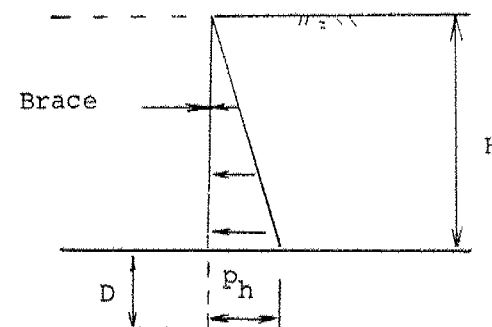
▲,△ - Undisturbed and remoulded undrained shear strength determined from insitu vane test

■ - Undrained shear strength determined from pocket penetrometer test

## NOTES:

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system is, in addition to the soil type and temporary/permanent surcharge loads, dependent upon the permissible lateral/vertical movements adjacent to the excavation, the groundwater conditions, drainage provisions, the type of bracing system adopted, weather conditions, quality of workmanship and length of time excavation will be supported. Hence, recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system is established.
2. Earth pressure diagram applicable to multi-braced cuts in stiff clays. Maximum depth of excavation 12 m (40 ft.).
3. Design lateral pressure may be reduced if some surface movement acceptable and design life of bracing system less than 1 month.
4. Stability of base of excavation must be confirmed when bracing system, excavation geometry and surcharge loads established.
5. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system not included in earth pressure diagram.
6. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
7. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
8. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
9. Earth pressure diagram applicable for time frame of relatively short construction periods. If excavation to be open for long periods, monitoring of deformation is essential; earth pressure diagram to be reviewed; remedial works may be required.
10. If settlement sensitive structures located near excavation, special measures to be undertaken to control settlements. Condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
11. Structural components of bracing system to be confirmed adequate for each level of excavation.
12. Bracing system to be regularly examined for signs of distress.
13. All work to be carried out in accordance with conventional construction practice, good quality workmanship and satisfy requirements of local building codes.
14. This sheet to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

## EARTH PRESSURE DIAGRAM



## DESIGN PARAMETERS

$p_h$  = lateral earth pressure  
=  $K\gamma H$

$K$  = lateral earth pressure coefficient

$\gamma$  = unit weight of soil

$H$  = depth of excavation

$D$  = depth of embedment of soldier piles  
(if used)

## RECOMMENDED DESIGN PARAMETERS

$\gamma$  = 20.5 kN/m<sup>3</sup>

$k$  = 0.6 for rigid wall

= 0.4 for flexible wall

LATERAL EARTH PRESSURE DISTRIBUTION  
Singly Braced Cuts in  
Stiff Clays



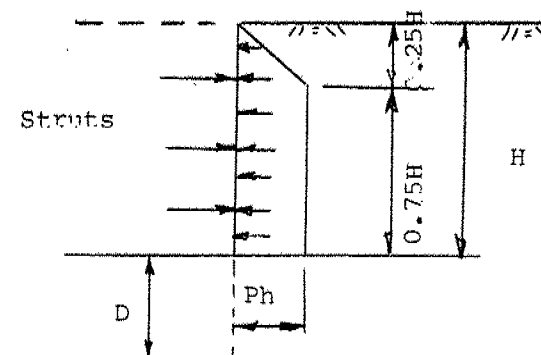
PETO MACCALLUM LTD.  
CONSULTING ENGINEERS

DATE	SCALE	JOB NO.	DRAWING NO.
Aug. '91	NTS	91HF007	3

# NOTES:

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system is, in addition to the soil type and temporary/permanent surcharge loads, dependent upon the permissible lateral/vertical movements adjacent to the excavation, the groundwater conditions, drainage provisions, the type of bracing system adopted, weather conditions, quality of workmanship and length of time excavation will be supported. Hence, recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system is established.
2. Earth pressure diagram applicable to multi-braced cuts in soft to firm saturated normally consolidated clays which extend well below base of excavation. Maximum depth of cut 12 m (40 ft.).
3. Stability of base of excavation must be confirmed when bracing system, excavation geometry and surcharge loads established.
4. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system not included in earth pressure diagram.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. Earth pressure diagram applicable for time frame of relatively short construction periods. If excavation to be open for long periods, monitoring of deformation is essential; earth pressure diagram to be reviewed; remedial works may be required.
9. If settlement sensitive structures located near excavation, special measures to be undertaken to control settlements. Condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
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12. All work to be carried out in accordance with conventional construction practice, good quality workmanship and satisfy requirements of local building codes.
13. This sheet to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

## EARTH PRESSURE DIAGRAM



## DESIGN PARAMETERS

$p_h$  = design lateral earth pressure  
 $p_h = \gamma H - 1.6 c_s \geq 0.4 \gamma H$   
 $c_s$  = average undrained shear strength of clay long face of excavation  
 $\gamma$  = unit weight of soil  
 $H$  = depth of excavation  
 $D$  = depth of embedment of soldier piles (if used).

## RECOMMENDED DESIGN PARAMETERS

$\gamma = 20.5 \text{ kN/m}^3$   
 $c_s = 75 \text{ kPa}$

LATERAL EARTH PRESSURE DISTRIBUTION  
 Multi-Braced Cuts in soft to firm normally consolidated saturated clays extending below base of excavation



**PETO MACCALLUM LTD.**  
 CONSULTING ENGINEERS

DATE	SCALE	JOB NO.	DRAWING NO.
Aug. '91	N.T.S.	91HF007	4



**APPENDIX A**

**Rock Core Test Results**

**Provided by**

**Franklin Geotechnical Ltd.**



# **IONA SANITARY TRUNK SEWER HAMILTON, ONTARIO**

## **Results of Testing on Rock Cores**

Prepared for:

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45 Burford Road  
Hamilton, Ontario  
L8E 3C6

Prepared by:

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---

Peto MacCallum Ltd.,  
45 Burford Road  
Hamilton, Ontario  
L8E 3C6

27th May, 1991

Attention: Mr. Ty Garde, P.Eng  
Project Engineer

Dear Sirs,

**IONA SANITARY TRUNK SEWER - ROCK TESTING**

**REPORT G678.1**

We have pleasure in submitting the results of rock testing as authorized in your letter of February 21st.

The numerical modelling is still in progress. After some initial difficulties in calibrating the constitutive law for the shale, the results are now giving a realistic simulation of long term tunnel behaviour. A report will follow later this week.

I suggest that when convenient a meeting be arranged between R.V. Anderson Associates, you and us, probably at the University of Waterloo, to discuss the interpretation of results and to decide on the scope of any further calculations.

Sincerely,

**FRANKLIN GEOTECHNICAL LTD.**

John A. Franklin, Ph.D., P.Eng., CPGS,  
President

JAF/amp

## **TABLE OF CONTENTS**

### **1. INTRODUCTION**

### **2. TEST METHODOLOGIES**

- 2.1 Specimen Preparation
- 2.2 Uniaxial Compression Tests
- 2.3 Moisture Contents and Units Weights

### **3. RESULTS OF TESTING**

Table 1: Sample Information-Iona Sewer Project Rock Core

Table 2: Test Results

Appendix 1: Stress-Strain Graphs

## 1. INTRODUCTION

This report presents the results of tests on shale and limestone core from the Iona Sanitary Trunk Sewer Project in Hamilton, Ontario. The tests were conducted in the Rock Mechanics Laboratories at the University of Waterloo, on instructions from Franklin Geotechnical Ltd.

Table 1 identifies the core samples supplied by Peto MacCallum Ltd., Consulting Engineers. Fifteen specimens were tested, prepared from 22 core sticks of Niagara Escarpment Series rock formations.

## 2. TEST METHODOLOGIES

### 2.1 Specimen Preparation

The cores were delivered by Peto MacCallum to the University of Waterloo. They arrived in good condition; no damage had been caused by sampling or transport. The core sticks were 47 mm diameter and from 120 mm to 230 mm long.

ASTM Standards and ISRM Suggested Methods were used in determining Young's modulus, uniaxial compressive strength, moisture content, and wet/dry densities.

### 2.2 Uniaxial Compression Tests

Fifteen specimens were prepared for uniaxial compressive strength tests in accordance with the techniques outlined in ASTM D 4543-85; Standard Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances. Cores were cut and surface ground, and each specimen was measured to ensure a length-to-diameter ratio (L/D) of 2.0 to 2.5 and all elements straight to within 0.50 mm.

Uniaxial compressive strength was determined after one or more unload/reload cycles. During the first unload cycle, the load frame was manually adjusted to prevent total unloading of the specimen, with a minimum of 0.3 kN of load being retained during this and all subsequent unloading cycles. Axial displacement was applied at a constant rate of 1% strain per five minutes.

Graphs were automatically plotted of force versus displacement during the test. The secant and tangent Young's moduli were calculated at stress levels of half the uniaxial compressive strength, using the ISRM Suggested Method for Determining Deformability of Rock Materials in Uniaxial Compression.

### 2.3 Moisture Contents and Unit Weights

The moisture contents of the rock cores were determined using the techniques outlined in ASTM D 2216 - 80; Standard Method for Laboratory Determination of Water (Moisture) Content in Soil, Rock, and Soil-Aggregate Mixtures.

### 3. RESULTS

The results of index tests are included in Table 1 and those for strength and modulus are presented in Table 2. Stress-strain plots are given in Appendix 1.

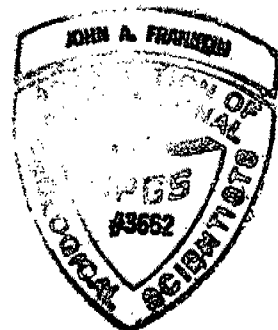
For the dolostone rocks, the average strength value of 79.2 MPa classifies this as a "very strong" rock according to ISRM and Canadian Foundation Engineering Manual criteria. The average tangent Young's Modulus of 32.8 GPa is in the middle of the range 17 to 100 GPa for crystalline limestones reported in Franklin & Dusseault (1989).

For the five Queenston Shale specimens tested (the rock formation through which the proposed tunnel is to pass), the average strength value of 15.8 MPa classifies this as a "weak" rock. The average tangent Young's Modulus of 11.9 GPa compares with a range of 2 to 30 GPa for low-durability shales reported in Franklin and Dusseault (1981). It is somewhat higher than the typical value of 1.3 GPa given for the Queenston shale in the report "Evaluation of Shales for Construction Projects", Franklin (1983), MTO Report RR229. This is not unexpected, because data on the Queenston Shale have been few and not very reliable. The values reported in the literature probably represent a weathered and softened shale, whereas the shale test results reported here apply to an unweathered shale recovered from a relatively deep drillhole.

Respectfully Submitted  
Franklin Geotechnical Ltd.



John A. Franklin, Ph.D., P.Eng., CPGS  
President



SAMPLE NUMBER	FORMATION	ROCK TYPE*	BH	DEPTH (ft, in)**	NOTES
1	Lockport	Dolostone	1	26'6"-27'1"	Waxed
2	DeCew	Dolostone	1	53'0"-53'7"	Waxed
3	Rochester	Shale	1	55'8"-56'5"	Unwaxed
4	Rochester	"	1	57'6"-58'0"	Unwaxed
5	Rochester	"	1	58'0"-58'7"	Unwaxed
6	Irondequoit	Limestone	1	63'4"-63'9"	Waxed
7	Reynales	Dolostone	1	64'8"-65'3"	Unwaxed
8	Thorold	Sandstone	1	74'10"-75'3"	Unwaxed
9	Grimsby	Shale	1	89'6"-90'0"	Waxed
10	Cabot Head	Shale	1	113'3"-113'9"	Waxed
11	Cabot Head	"	1	133'8"-134'2"	Waxed
12	Cabot Head	"	1	137'8"-138'2"	Waxed
13	Manitoulin	Dolostone	1	162'8"-163'5"	Unwaxed
14	Whirlpool	Sandstone	1	169'10"-170'4"	Waxed
15	Queenston	Shale	1	273'7"-274'3"	Waxed
16	Queenston	"	2	60'4"-60'11"	Waxed
17	Queenston	"	2	70'3"-70'10"	Unwaxed
18	Queenston	"	2	73'2"-73'9"	Waxed
19	Queenston	"	2	76'10"-77'4"	Unwaxed
20	Queenston	"	2	85'3"-85'10"	Unwaxed
21	Queenston	"	3	26'0"-26'6"	Waxed
22	Queenston	"	3	28'5"-28'11"	Unwaxed

\* rock type for the formation, not confirmed by thin section microscopy

\*\* samples and depth information supplied by Peto MacCallum Ltd.

TABLE 1: SAMPLE INFORMATION - IONA SEWER PROJECT ROCK CORE

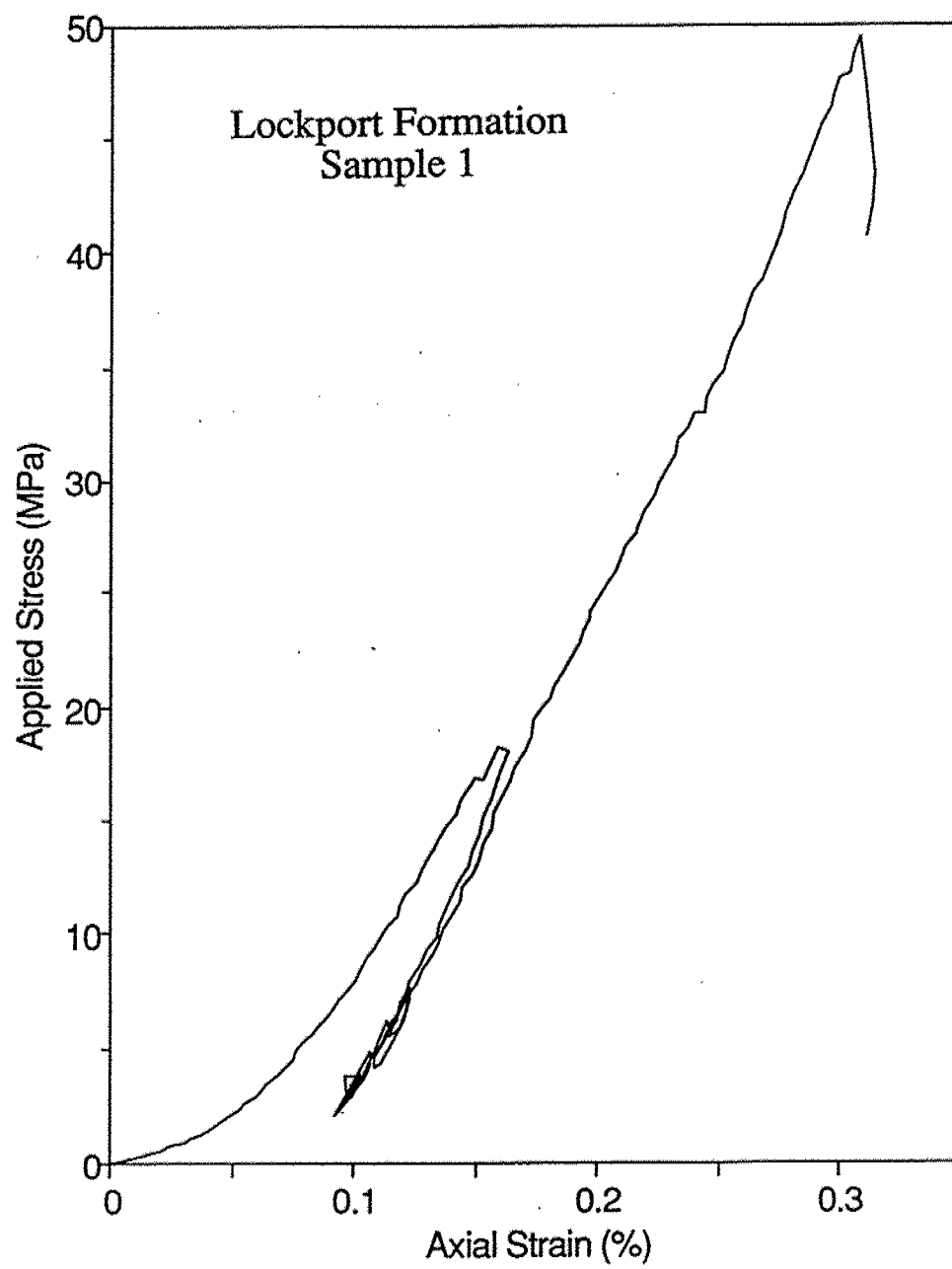
SAMPLE NUMBER	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	SONIC VELOCITY (km/s)	UNIAX. STRENGTH (MPa)	SECANT MODULUS (GPa)	TANGENT MODULUS (GPa)
1	2.84	2.56	4.56	49.44	12.32	18.44
2	1.34	2.60	4.56	113.95	22.25	44.40
3	0.37	2.43	5.18	25.72	13.77	25.40
4	0.22	2.19				
5	0.68	2.59				
6	1.61	2.00		76.83	21.95	33.50
7	0.55	2.96		112.23	31.98	47.07
8	0.61	2.12		86.39	22.79	34.98
9	5.00	2.16		9.55	2.72	3.33
10	3.59	2.56	3.00	7.04	1.81	2.69
11	4.20	2.27				
12	4.00	2.31				
13	0.43	2.83	5.10	41.88	6.58	20.78
14	5.50	2.12	4.26	94.17	27.19	39.62
15	2.45	2.68	3.34	13.30	5.84	8.64
16	1.74	2.56	2.99	8.61	3.28	3.78
17	0.47	2.52	3.65	19.07	11.05	15.88
18	3.51	2.51				
19	0.69	2.32	3.71	29.22	10.82	23.01
20	2.55	2.25				
21	4.66	2.21				
22	0.66	2.48	3.51	10.30	6.04	8.46

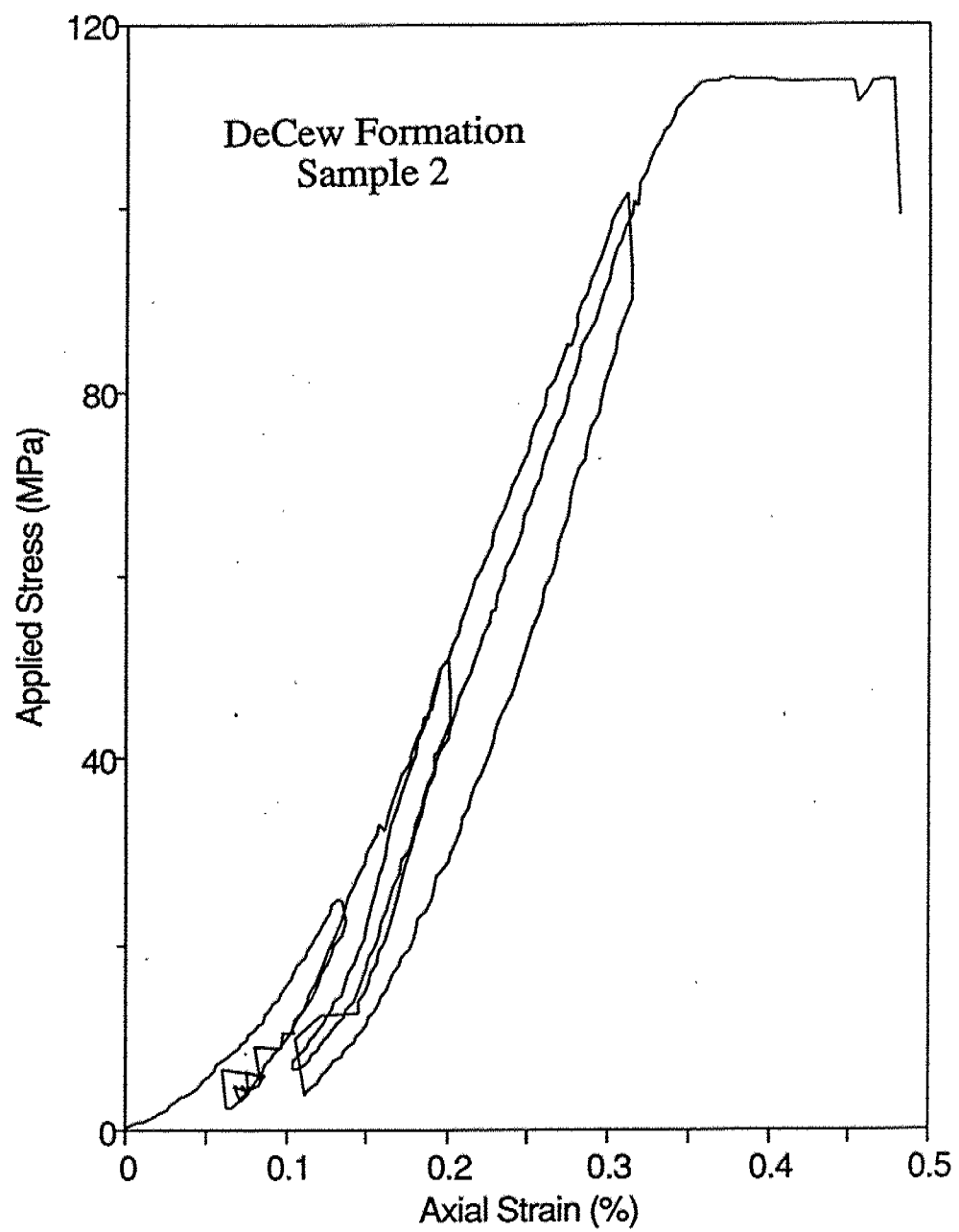
TABLE 2: TEST RESULTS

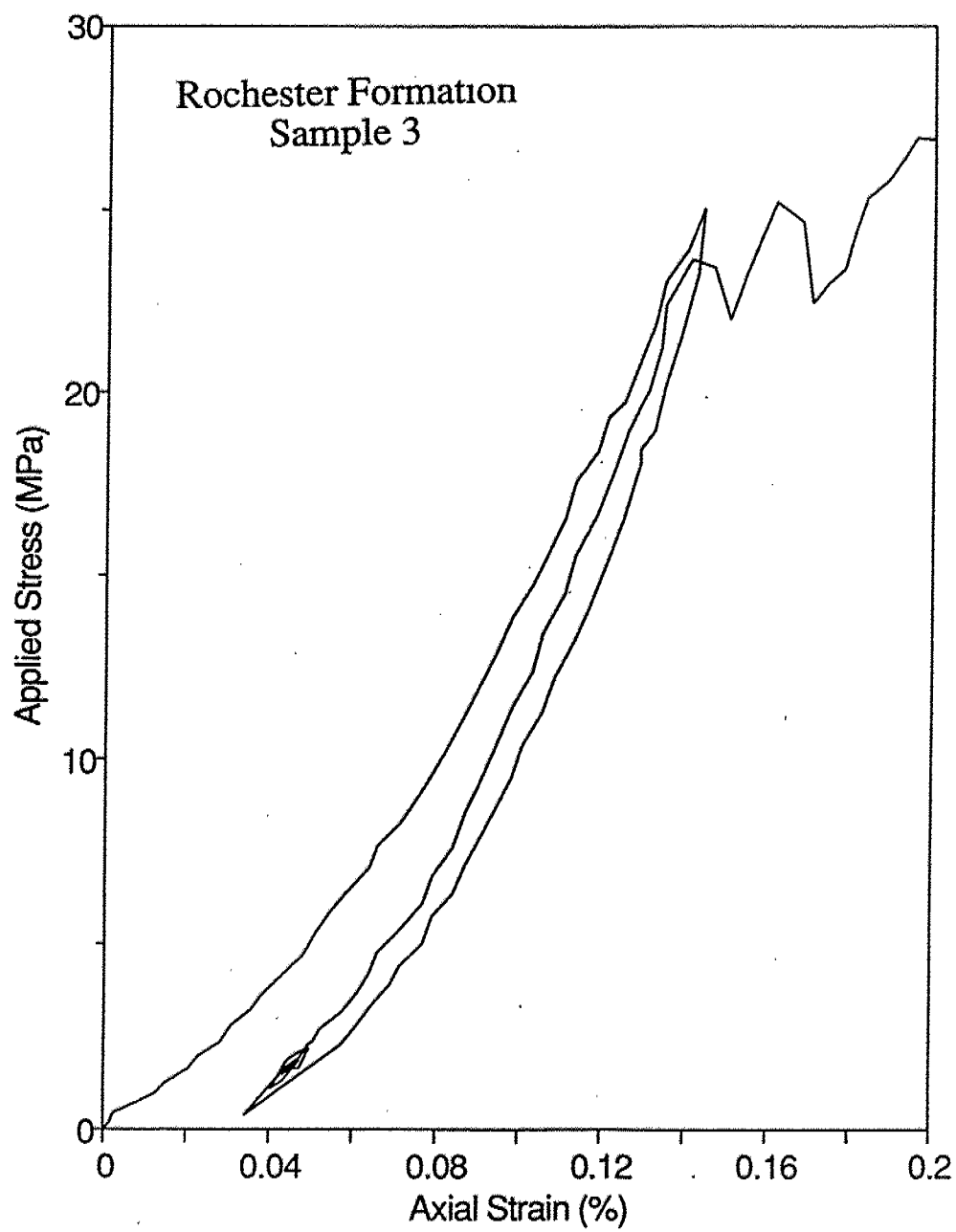


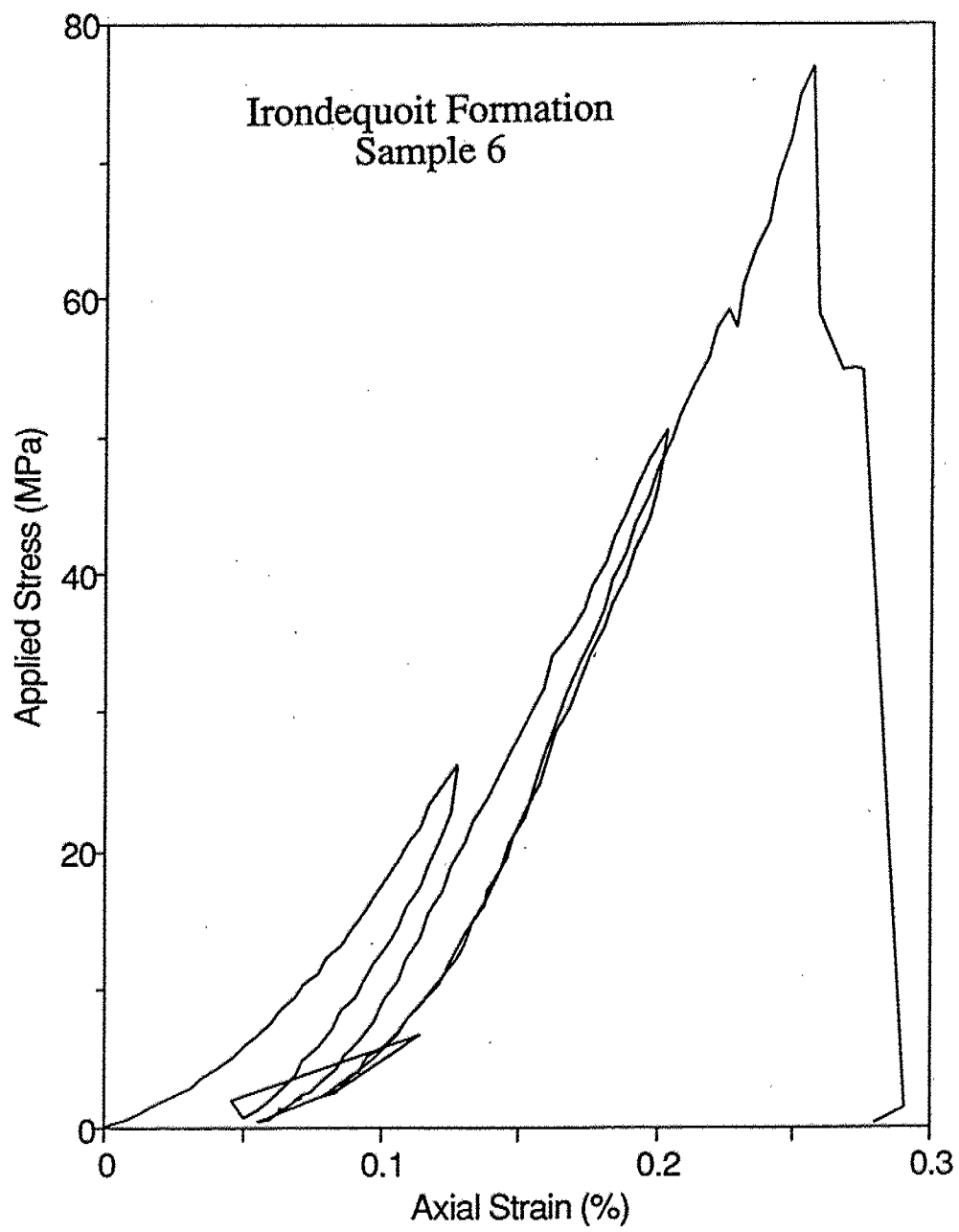
**APPENDIX 1**

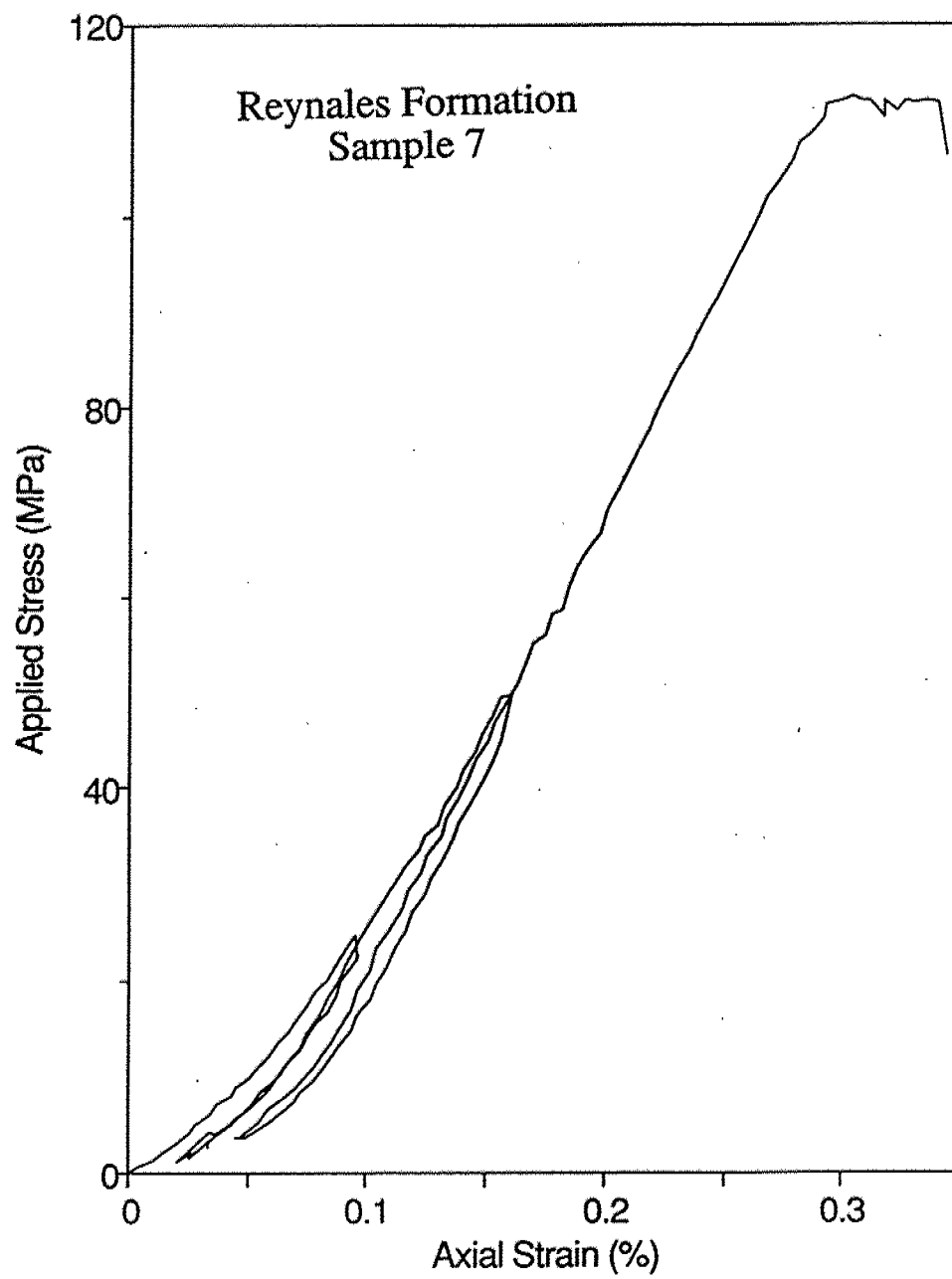
**STRESS-STRAIN GRAPHS**

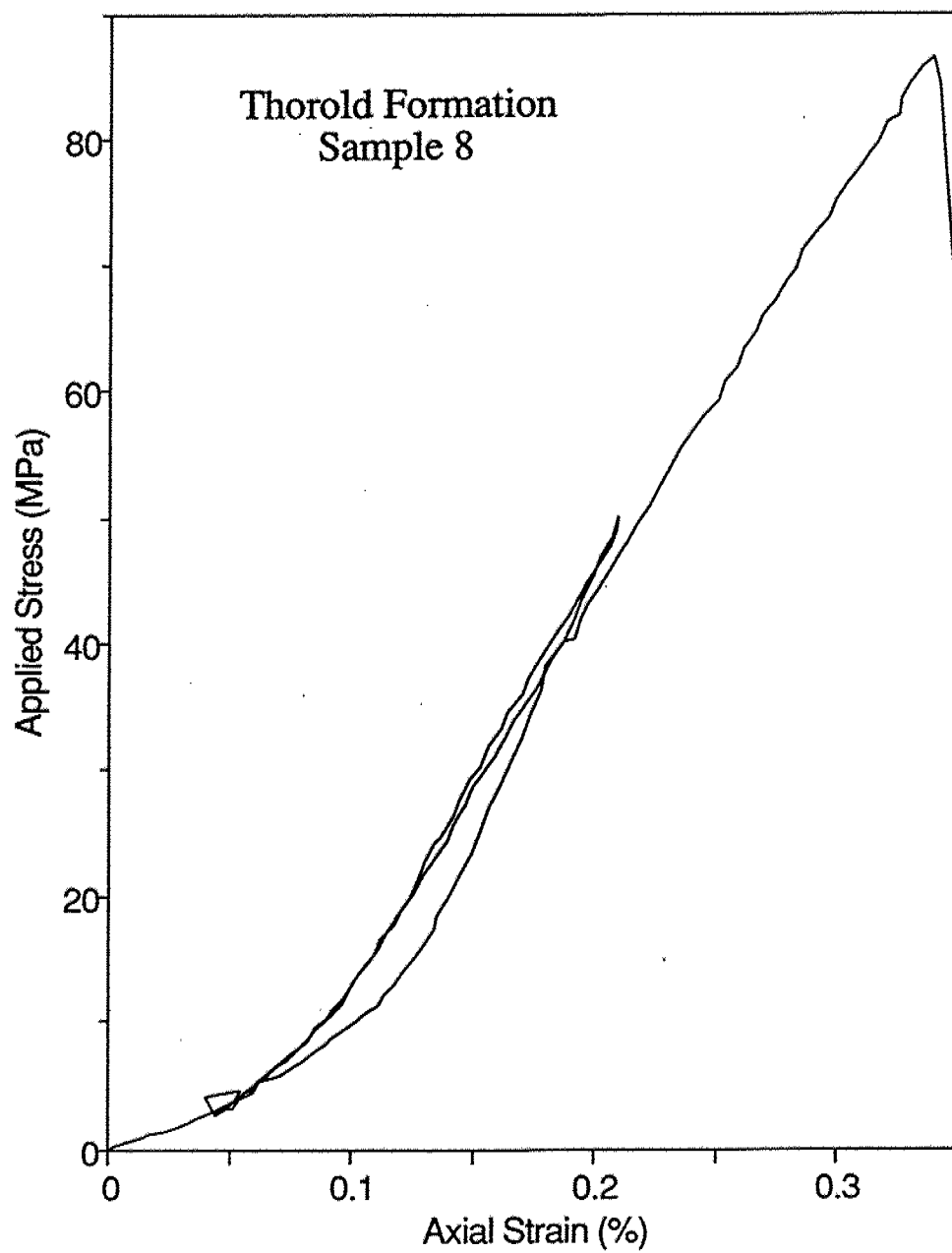


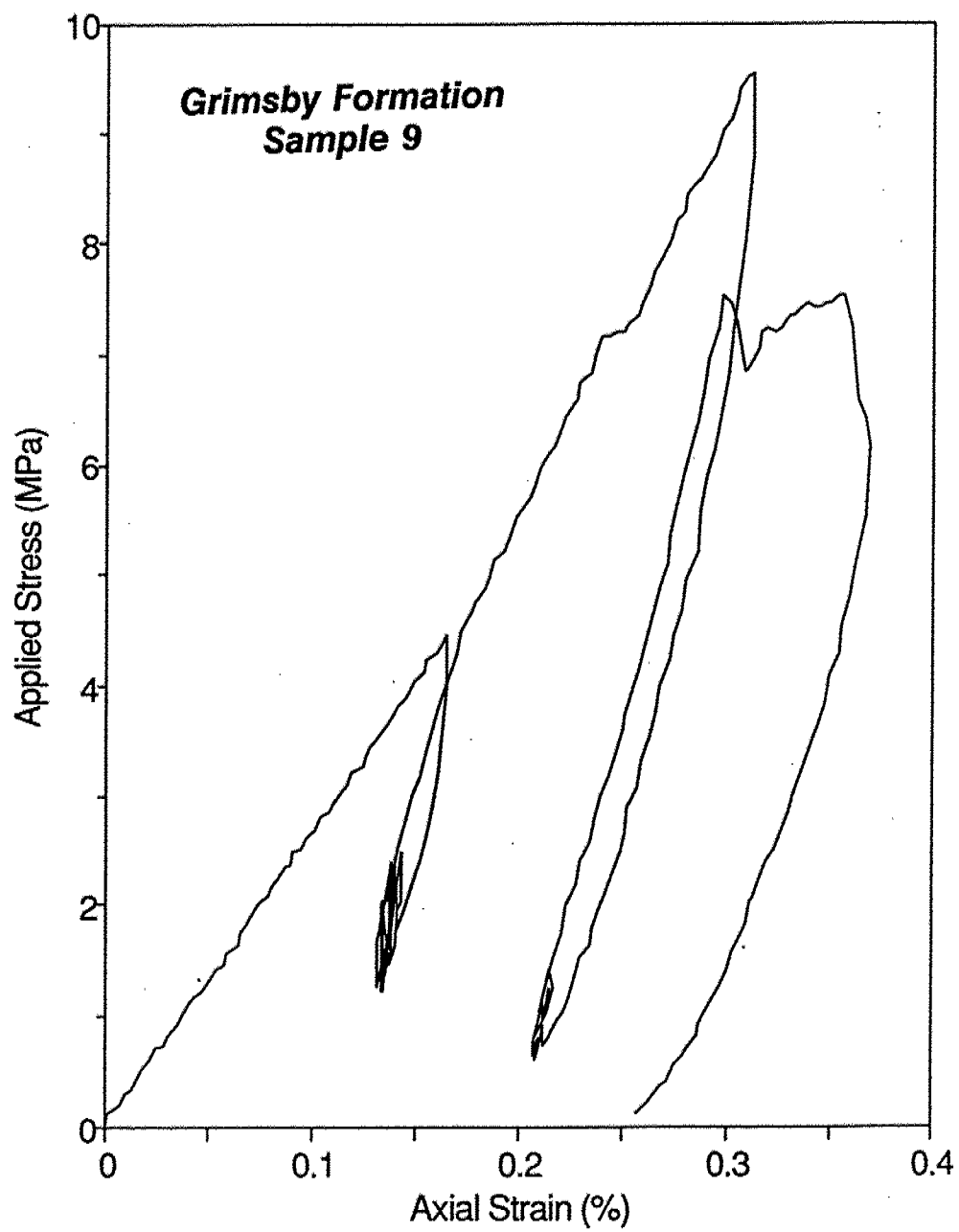




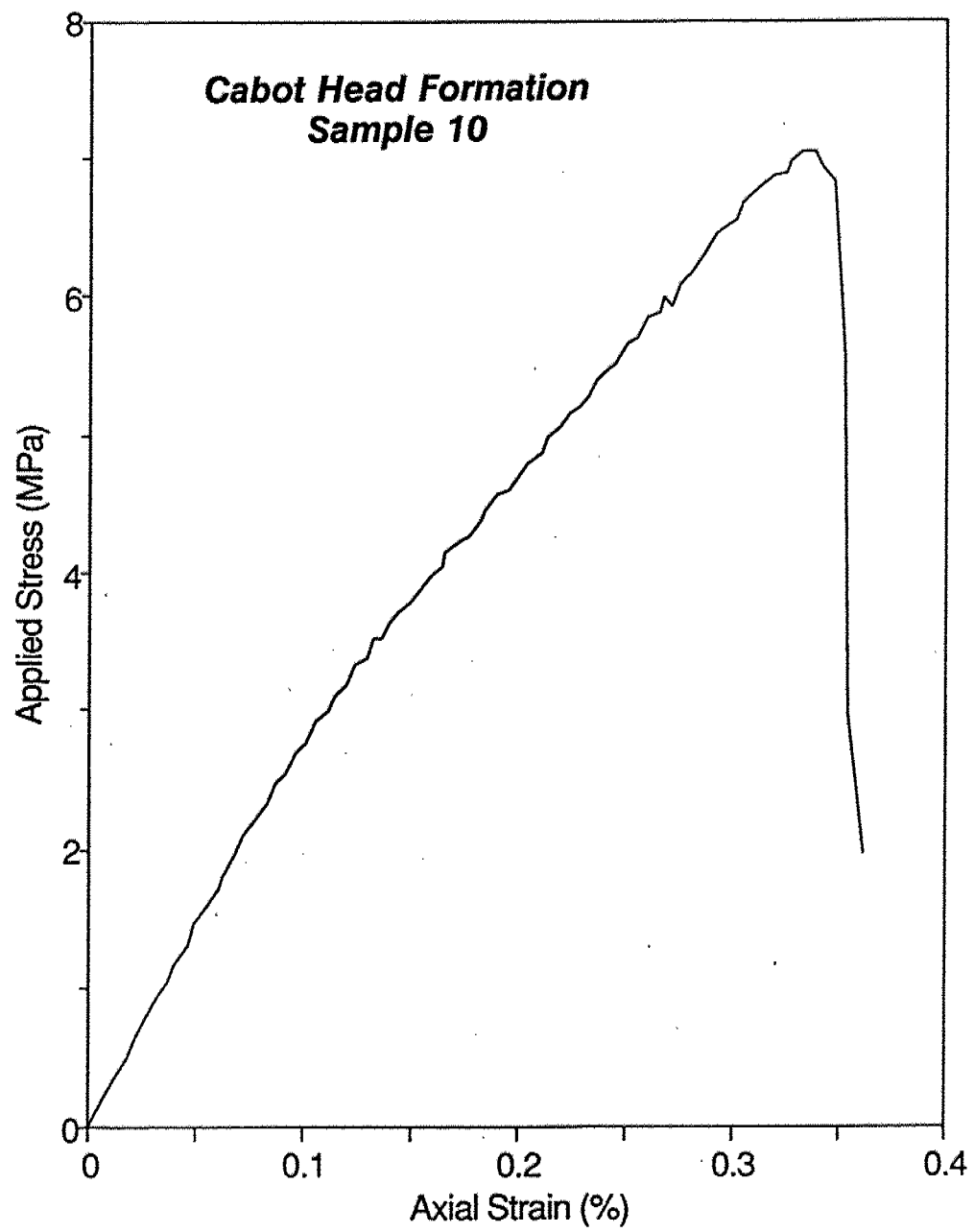


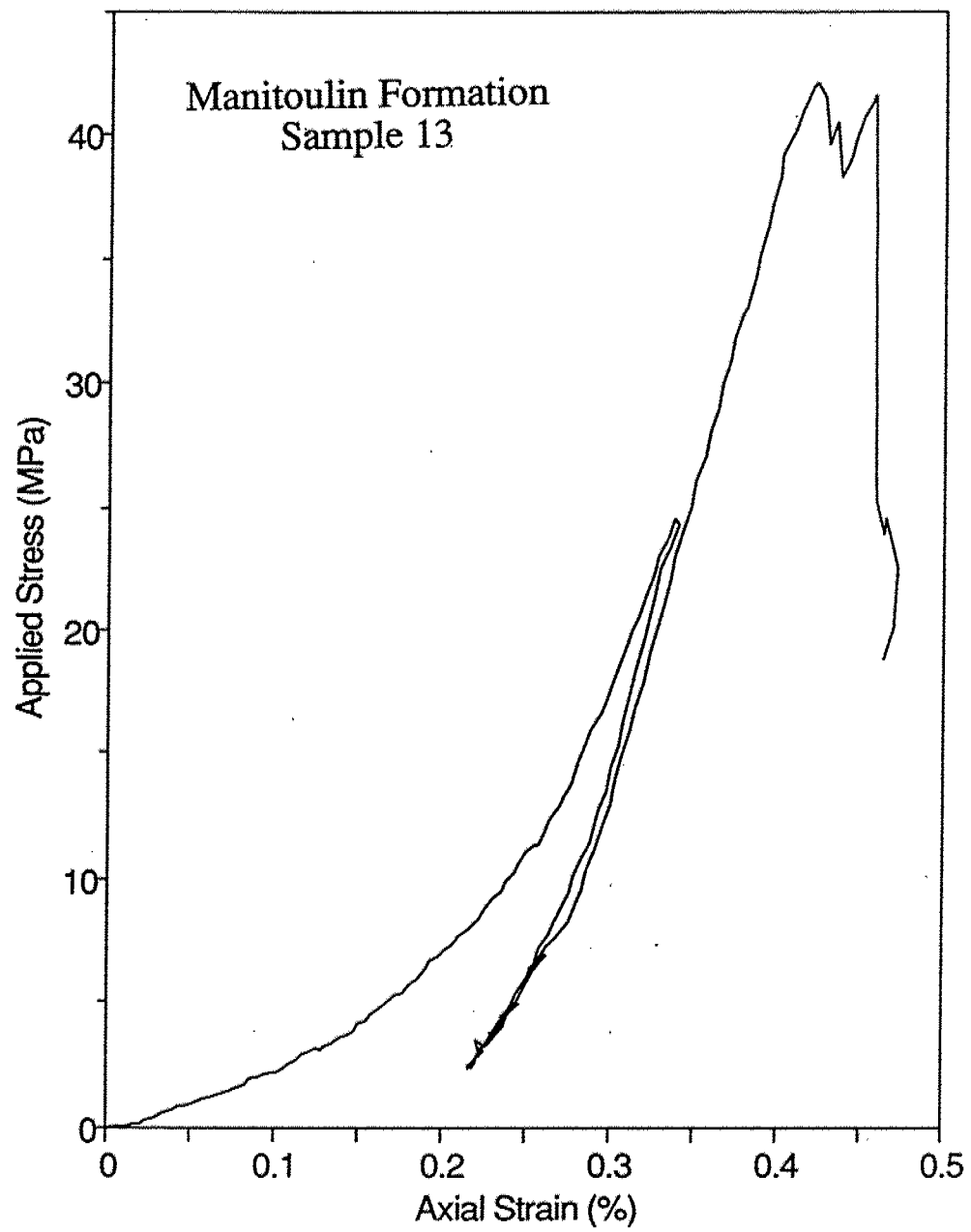


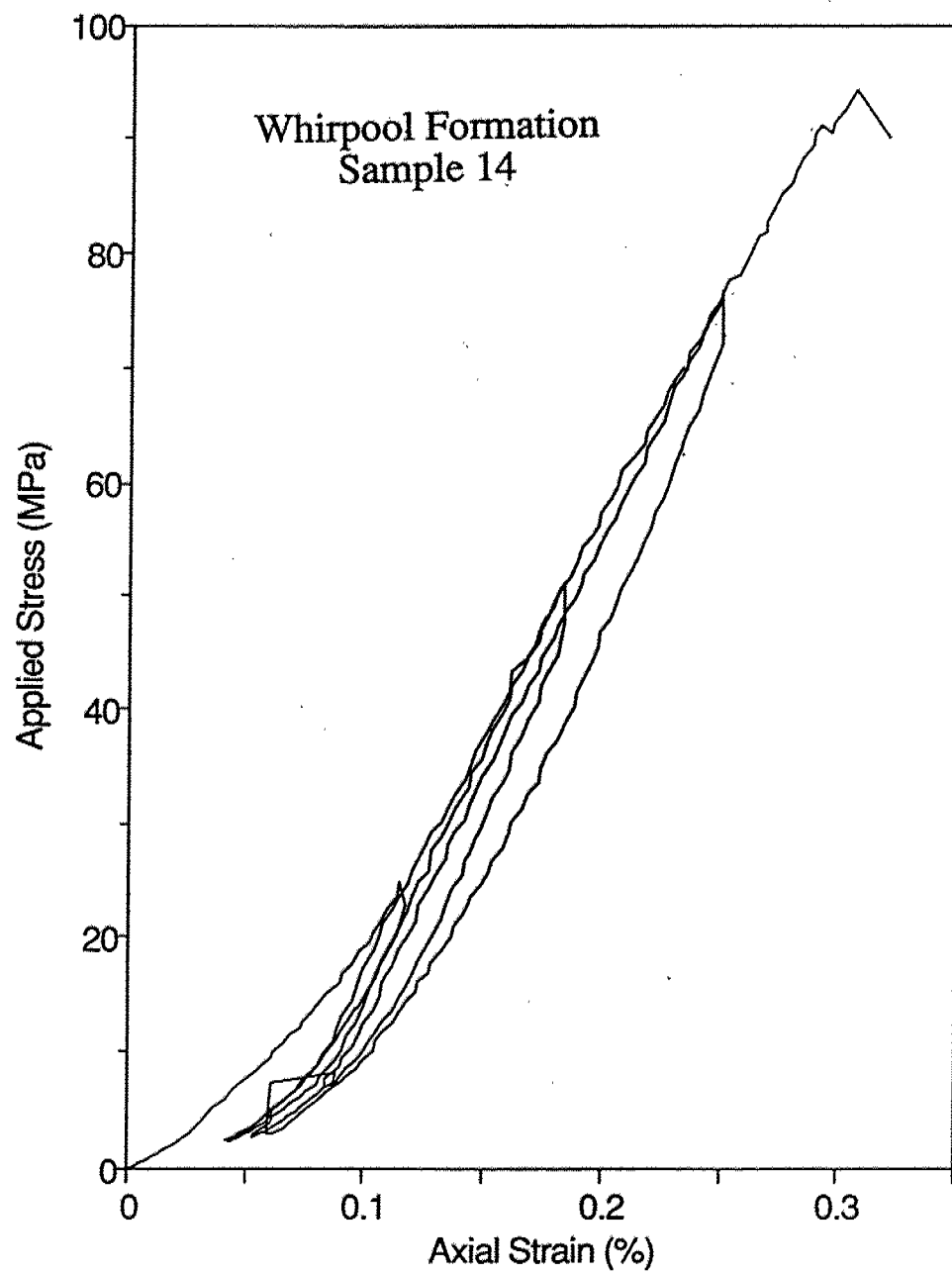


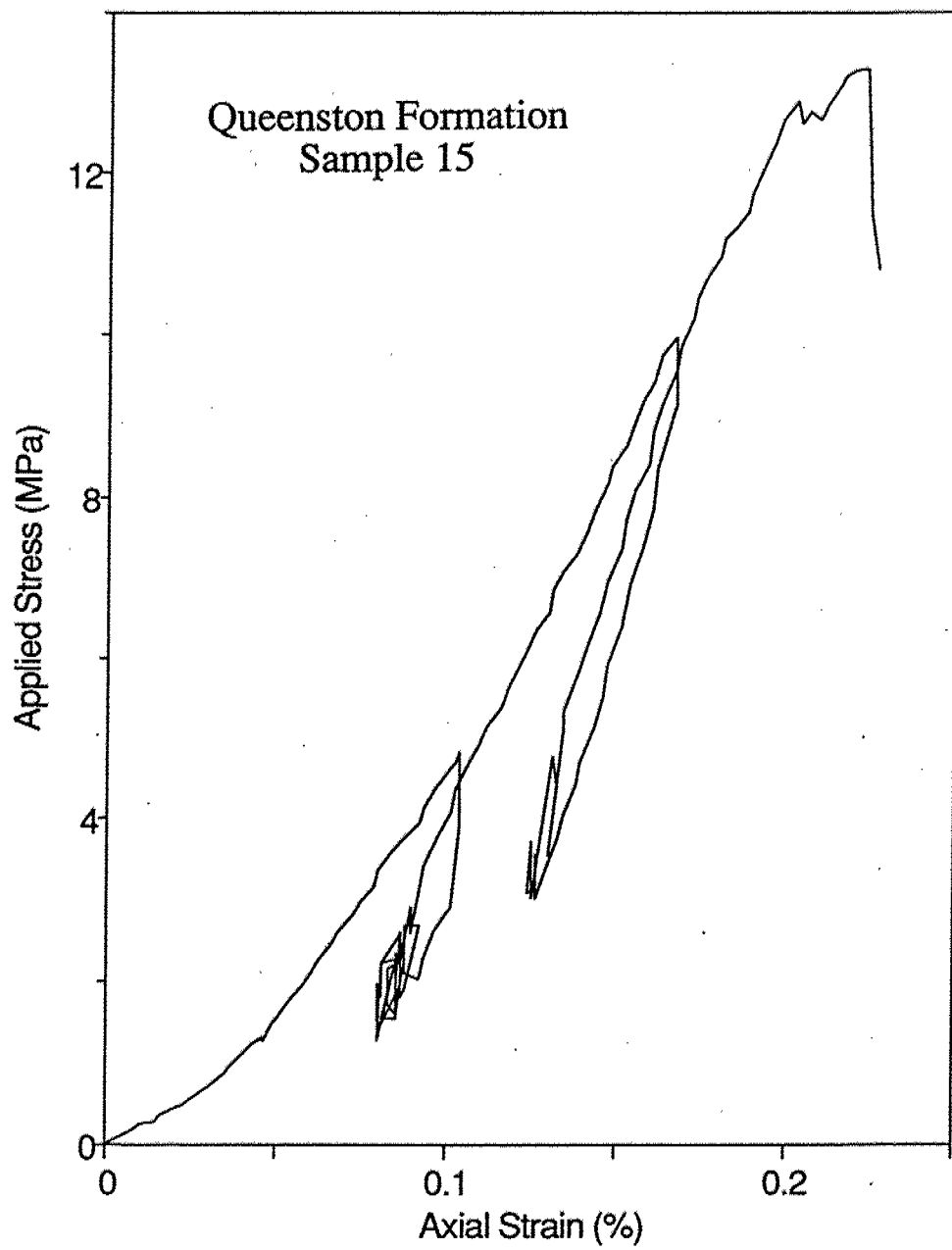


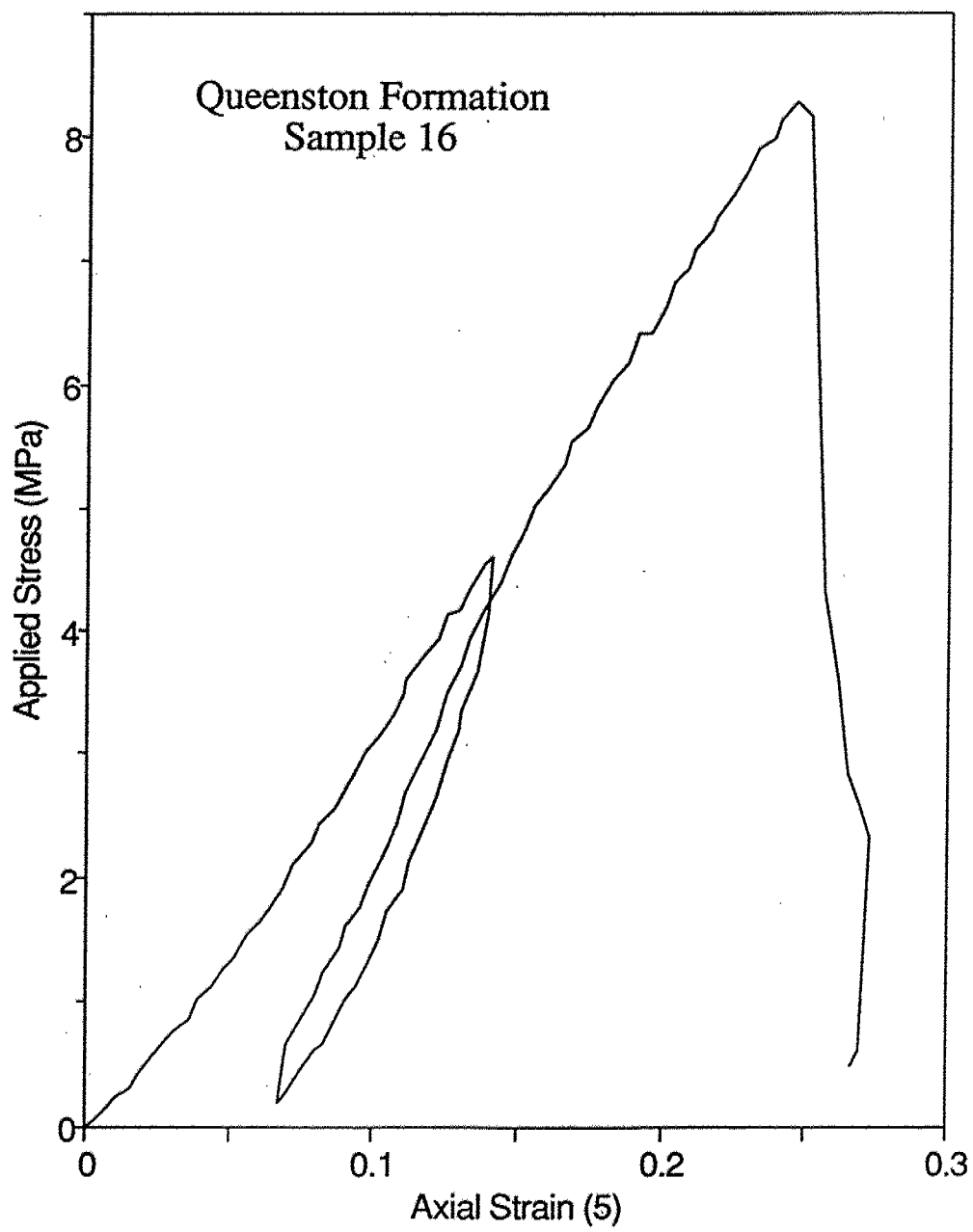


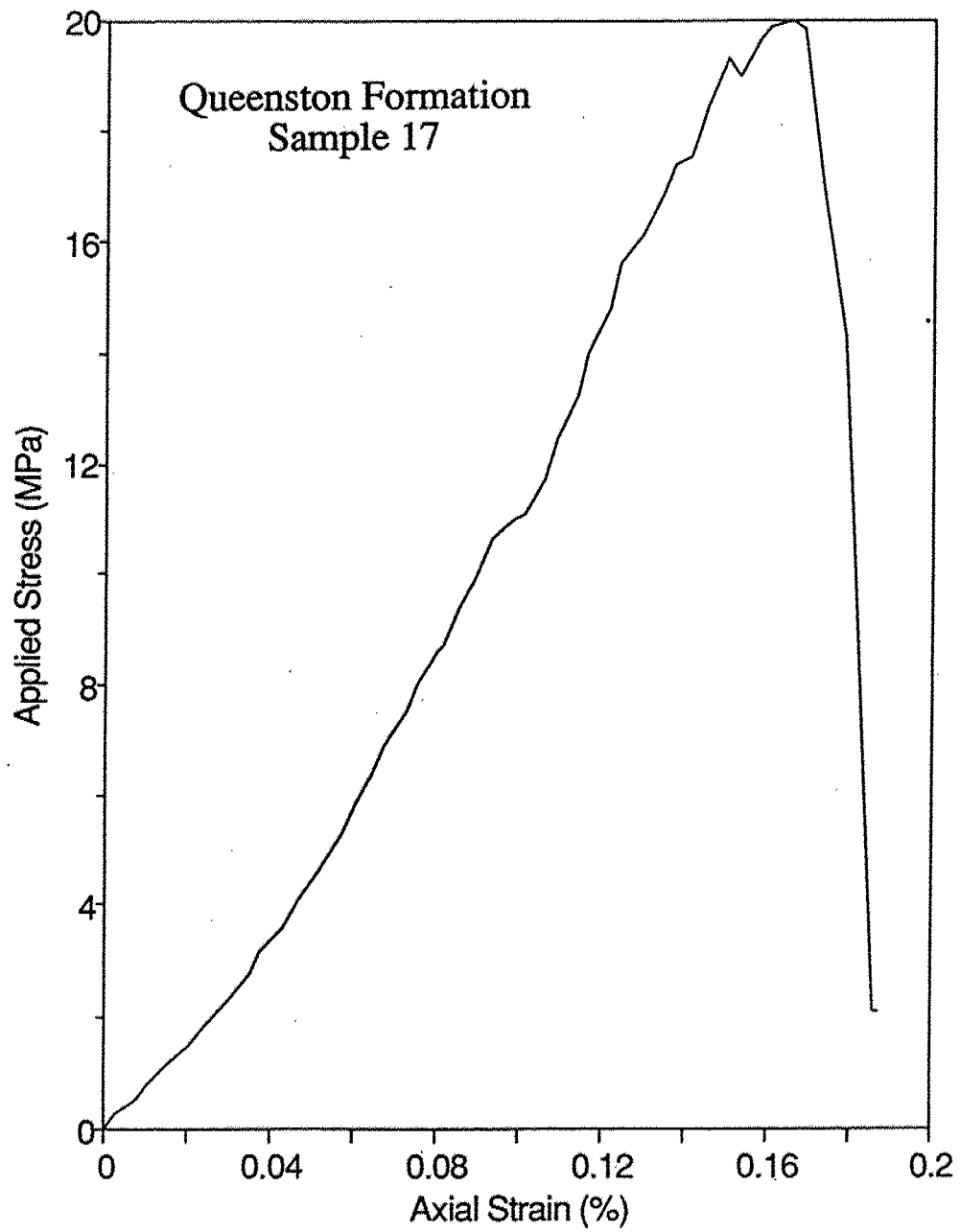


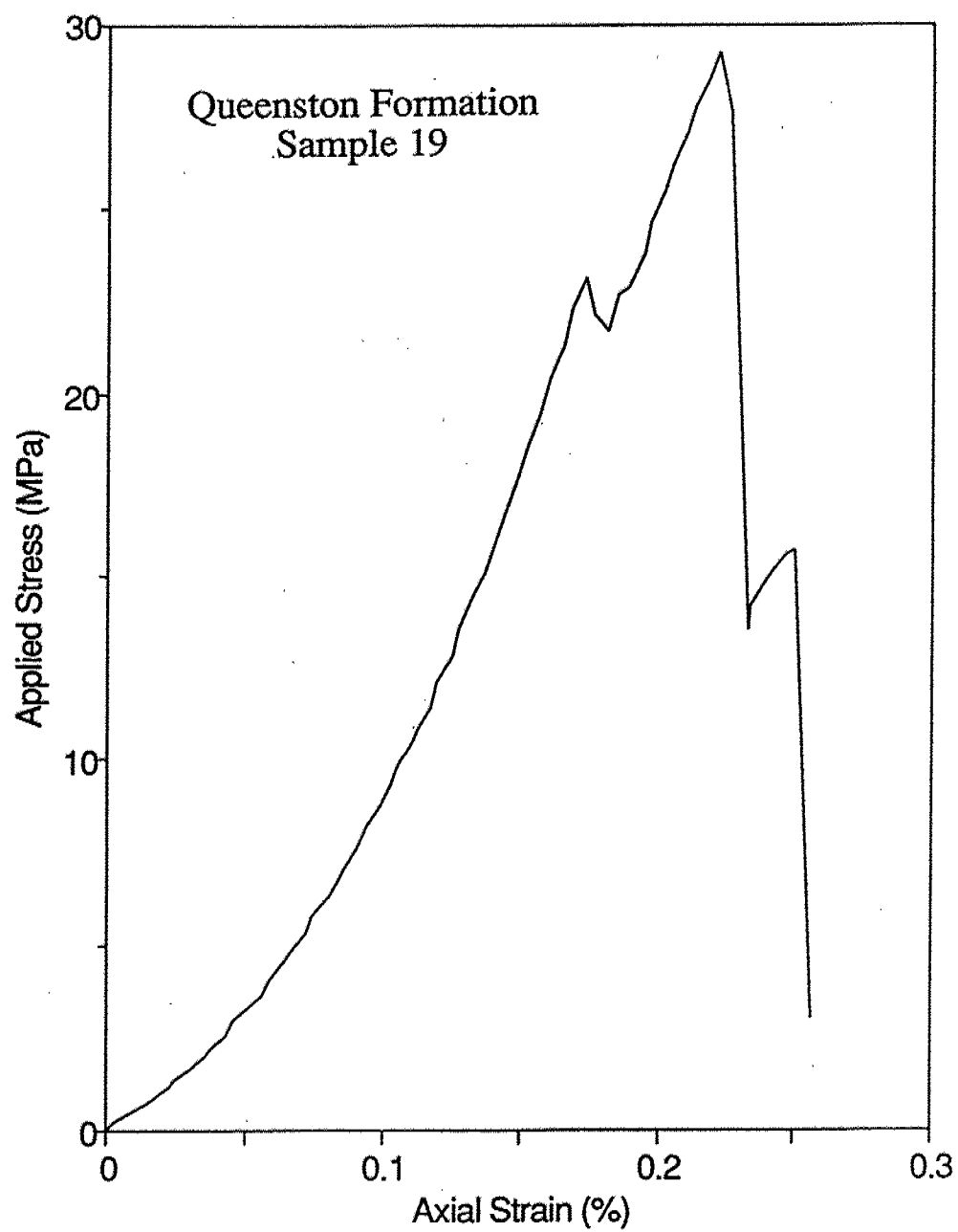


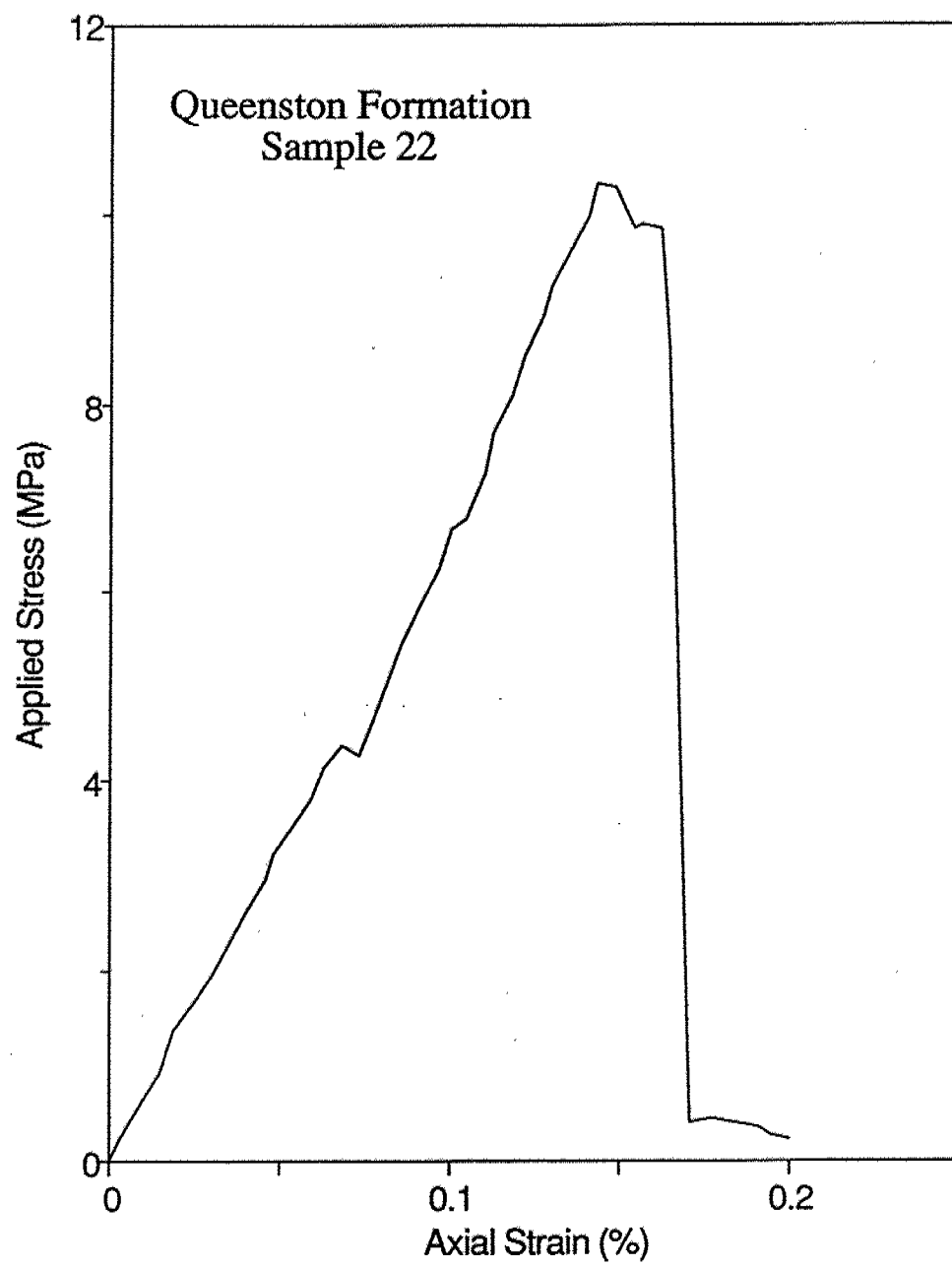














**APPENDIX B**

**Tunnel Liner Design Considerations**

**Provided by**

**Franklin Geotechnical Ltd.**



# **REPORT ON IONA SANITARY TRUNK SEWER HAMILTON, ONTARIO**

## **Input for Tunnel Liner Design**

Prepared for:

**PETO MACCALLUM LTD.**  
45 Burford Rd.  
Hamilton, Ontario  
L8E 3C6

Prepared by:

**FRANKLIN GEOTECHNICAL LTD.**  
The Stream  
R.R. #1, Orangeville, Ontario  
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tel: (519)941-3392

fax: (519)941-9773

Peto MacCallum Ltd.  
45 Burford Rd.  
Hamilton, Ontario  
L8E 3C6

June 3, 1991

Attention: Mr. Ty Garde, P.Eng.

Dear Ty,

### INVESTIGATION

Enclosed is our Report G678.2 giving the results of this investigation. We trust this meets with your approval, and will be happy to answer any questions that may arise.

Yours truly,

**FRANKLIN GEOTECHNICAL LTD.**

John A. Franklin  
President

**Distribution:**

PML - 2

FGL - 2

JAF:amp

/Enclosure

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### **2. VISCOT**

### **3. INPUT DATA**

- 3.1 Tunnel Data
- 3.2 Stress Assumptions
- 3.3 Rock Properties

### **4. RESULTS OF NUMERICAL MODELLING**

### **5. CONCLUSIONS**

### **6. REFERENCES**

#### **FIGURES**

- 1 Finite Element Model
- 2 Absolute Displacements vs Time After Excavation
- 3 Distribution of Horizontal Stresses Beneath Tunnel Invert

## 1. INTRODUCTION

Peto MacCallum Ltd. has engaged Franklin Geotechnical Ltd. as subconsultants to assist in rock mechanics aspects of an investigation for the Iona Sanitary Trunk Sewer Extension in Hamilton, Ontario.

Our recent Report G678.1 presented the results of tests on rock cores from the site investigation. This follow-up Report G678.2 contains responses to questions related to the design of the tunnel, as contained in Hamilton Wentworth Region's November 20, 1990 letter, and Addendum #1 of November 30, namely:

- Need for compressible materials around structures in shale (i.e. is the shale subject to convergence due to stress release?);
- Need for protection of shale after excavation;
- Modulus of Deformation and Poisson's Ratio of the rock;
- Ratio of fully mobilized horizontal pressure to vertical pressure acting on the tunnel lining;
- Estimate of convergence versus time for the tunnel and shafts;
- Feasibility of reducing liner stresses by delaying the installation of the liner;
- Estimation of pressure distribution around tunnel and shaft liners for an assumed delay time.

We discussed the rock mechanics requirements with Mr. Harvey Johnson and Mr. Douglas Dunbar of R.V. Anderson & Associates Ltd., and in our letter of 13th February offered to investigate interaction of the rock and liner using a finite element creep model similar to the one used in design of the Beulah Tunnel in North Dakota.

The computations were carried out under the supervision of Prof. Dusseault and by postgraduate students Milutin Petrovic and Pavel Vasak in the Earth Sciences Department at the University of Waterloo.

## 2. VISCOT

The numerical analyses have made use of VISCOT, a two-dimensional, non-linear, transient viscoelastic and viscoplastic finite element code for modelling the time-dependent mechanical behaviour of a rock mass.

The original version of VISCOT was developed by INTERA Environmental Consultants, Inc. for the National Waste Terminal Storage (NWTS) program of the United States Department of Energy, and was an adaptation of subroutines given in Owen and Hinton (1980).

The version of VISCOT that we adapted to study the Iona tunnel in shale had been developed by Professor Leo Rothenburg and employed by Rothenburg and Dusseault to simulate the behaviour of cavities mined into rock salt. The salt version employed a viscoelastic temperature- and stress-dependent constitutive model based on experimental work on salt creep at the University of Waterloo. The original viscoplastic routines were omitted, being inappropriate for the simulation of rock salt behaviour.

To study a tunnel in shale with a liner installed at a specified time after excavation, the following modifications were necessary:

- Reactivation of the subroutines for modelling elasto-viscoplastic behaviour;
- Rewriting part of the code to permit introduction of liner elements after some specified time.

The elasto-viscoplastic behaviour simulation was tested using the one-dimensional bar test cases published by Owen and Hinton (1980) and in the VISCOT manual.

To simulate lining of the tunnel, two compiled versions of VISCOT were used in parallel. The first version simulated creep of the unlined tunnel, and the second used an updated grid with liner elements added, taking as input the stress and strain results from the first run.

### 3. INPUT DATA

#### 3.1 Tunnel Data

The tunnel will be driven through the Queenston Shale Formation, probably by drill-and-blast methods. As presently envisaged, and as modelled using Viscot, it will have a circular, 3.6 m diameter cross-section, and a 300 mm thick concrete liner with compressive strength 35 MPa and Young's Modulus 30 GPa, Poisson's Ratio 0.17 (R.V. Anderson communication of 13th February, 1991).

#### 3.2 Stress Assumptions

Vertical stress was assumed to be gravitational, increasing linearly with depth. Rock cover was assumed as about 40 m, and with a stress gradient of 26 kPa/m for sedimentary rocks, this gives a vertical stress prior to excavation of 1 MPa at the tunnel crown.

In the absence of stress measurements for this project (which are difficult when the rock is a low strength shale), as a first approximation, the horizontal stresses were assumed equal and constant at 7.0 MPa (Figure 1). A constant horizontal stress over a limited range of relatively shallow depths is realistic, and the value of 7 MPa is typical of horizontal stresses that have been measured in Ontario shales. Values of up to 8.2 MPa were reported by Morton et al. (1975) for the Dundas Shale, and of 1.6 MPa by Franklin Trow Associates (1975) for the Collingwood Shale at the Easterly Filtration Plant tunnel in Scarborough. The value of 7 MPa is about one half of the 14 MPa which is a typical maximum for the stronger dolostones of southern Ontario.

#### 3.3 Rock Properties

The Queenston Shale samples tested (see Report G678.1) had an average uniaxial compressive strength of 15.8 MPa, and an average Young's Modulus of 11.9 GPa. Typical values reported by Franklin (1983) for the Queenston Shale are 8.7 (7.2 - 9.6) MPa for uniaxial compressive strength, and 1.3 (0.5 - 2.3) GPa for laboratory measurements of Young's Modulus. Hence the Iona tunnel

samples tested were 1.8 times as strong and 9.1 times as stiff as the previously reported "typical" values.

Time-dependent deformation (creep) consists of an elastic component and a viscoplastic component which are estimated using a constitutive model for the shale. No data are available from triaxial creep tests on Queenston Shale. Therefore, the "fluidity parameter" in VISCOT was estimated by calibrating the model to give a vertical convergence of 100 mm in one month, as measured at the Easterly Filtration Plant Intake Tunnel. Creep could be terminated by adjusting a strain hardening parameter, but for the results reported here, creep was allowed to continue at a decreasing rate indefinitely.

The behaviour monitored in the Easterly Filtration Plant intake tunnel in Scarborough (Franklin Trow Associates Ltd. Report F101, 1975) is considered the closest available comparison for the behaviour of the Iona tunnel in Queenston Shale, although the Collingwood shale at Scarborough is somewhat stronger and stiffer.

Estimation of the fluidity parameter should be checked by monitoring convergences in the Iona tunnel during construction, and if possible also by laboratory creep testing on samples of Queenston Shale. Creep tests on Queenston Shale are in progress at the University of Waterloo as part of an unrelated research project, but the results are unlikely to be ready in time to be useful for the Iona project.

It may be relevant to note that suitable creep test data were absent also for design of the liner for the Beulah Tunnel in North Dakota. Franklin Trow Associates Ltd., working for R.V. Anderson Associates Ltd., then made use of the best available alternative, which at the time was creep data for the London Clay in England (Hanafy, Emery and Franklin, 1976, 1977).

#### 4. RESULTS OF NUMERICAL MODELLING

Figure 2 shows the anticipated time-convergence behaviour of the tunnel, with primary support (shotcrete) to prevent shale loosening and drying-out, but before installation of a concrete liner. Figure 2 demonstrates that Viscot is capable of realistically simulating the parabolic shape of convergence-time graphs such as obtained in shale tunnels in southern Ontario.

The displacements shown are absolute, measured with respect to a fixed datum 18 m below the invert (Fig. 1). Horizontal convergence reaches about 40 mm (2 x 20 mm inward-positive wall displacements) after about 30 days, and continues at a decreasing rate. Vertical convergence includes about 20 mm of floor heave and 28 mm of roof sag for a total of about 48 mm after 30 days.

Following tunnel excavation in shale, a plastic (yielded) zone develops in which shear stress concentrations exceed the strength of the rock. Stresses are transferred into the surrounding unyielded rock, because the yielded material is no longer capable of sustaining shear stresses. Figure 3 shows the tangential stress profile beneath the invert, as the plastic zone develops with time. Because of the 7 MPa far-field horizontal stress, the invert is the most severely stressed location.

The solid line, which represents the initial elastic condition immediately after excavation, shows a stress concentration of about  $\times 2$  (the maximum value is about 14 MPa close to the invert). The stress concentration becomes insignificant about 6 m below the invert.

The various broken lines show stress distributions computed at various times after excavation. Values further than 4 m below the invert show a consistent trend towards stabilization with decreasing amounts of stress buildup as time elapses. The innermost ring of finite elements, however, shows anomalous values of stress that are considered unrealistic.

The anomaly is the result of a flaw found in the viscoplasticity subroutines of VISCOT. The stresses in the ring of elements directly adjacent to the opening continue to increase unrealistically with time. This increase in stress results from the stiffening of these elements. The flaw can be corrected relatively easily, given time. For the present, however, stress buildup on the concrete liner cannot be simulated realistically because it is exactly at this location that the anomaly occurs.

## 5. CONCLUSIONS

The following responses to the questions posed by the terms of reference are therefore based largely on experience in previous tunnels, rather than on the results of numerical modelling. This is not unusual: for example, in the recent guide to cavern engineering published by the Geotechnical Control Office of Hong Kong, the consulting engineers Berdal Strome point out that although large caverns are often analyzed to determine stresses and stability, the results are used mainly to extrapolate experience to cover conditions not previously encountered. An empirical approach governs the design even of these large and expensive rock caverns. They stress that "mathematical analysis is no substitute for experience".

We nevertheless hope to complete the calculations working alongside R.V. Anderson design engineers. The limit of resources available in the site investigation budget has been reached. Further analysis might be financed either from research or from project funding, but this remains to be determined. In the meantime, our responses to the questions posed in the Terms of Reference are as follows:

- The shale will converge as a result of the concentration of stresses around the tunnel. We anticipate a maximum pre-excavation horizontal stress of between 2 MPa and 7 MPa, accompanying a vertical stress of 1 MPa. The stresses will be magnified by a factor of about  $\times 2$  in the vicinity of the tunnel. Hence the levels of stress will approach or exceed locally the uniaxial compressive strength of the shale which is estimated at between 10 MPa and 16 MPa. In practice this presents little or no problem if the shale is shotcreted. A plastic yield zone will develop, although the thickness of this zone is likely to be limited to about 1 m if the shale is confined by appropriate primary support.

The Regional Municipality of Hamilton Wentworth have adopted the routine of installing a compressible layer between concrete liners and rock, as a result of earlier problems in the Red Hill Creek and other cut-and-cover projects. A compressible buffer would help here also, although it is not so easy to implement in the case of a circular fully below-ground tunnel. A sprayed foam liner might be worth investigating.

- The Queenston shale has a relatively low durability and should be prevented from drying out. In the tunnel, the immediate application of a shotcrete (fibercrete) primary liner is



recommended, which will serve the multiple purposes of preventing drying, inhibiting loosening, and permitting controlled displacements and arching to develop. The liner also assists in monitoring ongoing movements.

- In further calculations we recommend assuming a Modulus of Deformation of 10 GPa and a Poisson's Ratio of 0.3 for the rock.
- The ratio of fully mobilized horizontal pressure to vertical pressure acting on the tunnel lining remains to be computed.
- Convergence is likely to develop in the shotcreted tunnel to a maximum of about 40 mm over a period of about 1 month, becoming slower thereafter. The rates of tunnel liner displacement are likely to be similar to those shown in Fig. 2. Much less convergence is expected in the case of the shaft, which because of the dolostone and other hard rock beds, should not experience the same problem of rock squeeze.
- Installation of the primary shotcrete liner should not be delayed. Shotcrete should be applied daily to keep pace with excavation. The final liner can be installed on completion of the tunnel provided that rates of convergence appear to have stabilized to acceptable values. Further delays are unlikely to be beneficial.
- Further calculations might assist in estimating the reductions in liner pressures as a function of delay time between excavating and lining. However, these calculations are seldom reliable, particularly in the absence of creep test data. The best alternative is to use tunnel monitoring to calibrate the creep law, and to re-run the calculations during construction as an aid to deciding whether delays are justified.

## 6. REFERENCES

EMERY J.J., HANAFY E.A. and FRANKLIN J.A., 1977. Finite element simulation of tunnels in squeezing ground. Proc. Int. Symp. on the Geotechnics of Structurally Complex Formations, Capri, Italy, Vol.1, pp. 219-228.


EMERY J.J., HANAFY E.A. and FRANKLIN J.A., 1977. Creep movements associated with excavations in rock. Proc. Conf. on Large Ground Movements and Structures, Cardiff, Wales, 26p.

FRANKLIN J.A., 1983, Evaluation of shales for construction projects - an Ontario Shale Rating System. Ontario Ministry of Transportation, Rept. R.R. 229, 99 p.

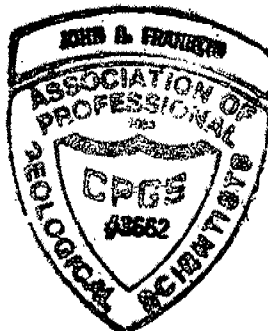
HANAFY E.A., EMERY J.J. and FRANKLIN J.A., 1976. Tunnel lining strategies. Proc. 3rd Symp. Eng. Applications of Solid Mechanics, Toronto, Vol. 2, pp. 179-202.

OWEN, D.R.J. and HINTON, E., FINITE ELEMENTS IN PLASTICITY: Theory and Practice, 1980, Pineridge Press, Swansea, U.K., 594 p.

Respectfully submitted,  
FRANKLIN GEOTECHNICAL LTD.



John A. Franklin, Ph.D., P.Eng., CPGS,  
President



## FIGURES



## CASE 1: 7 MPa UNIFORM

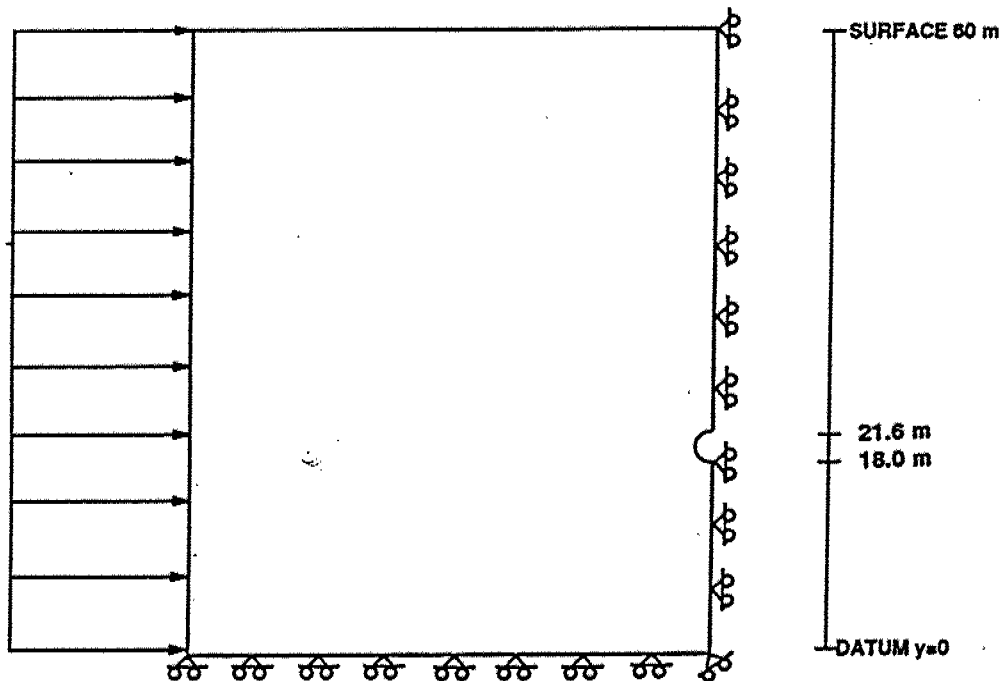

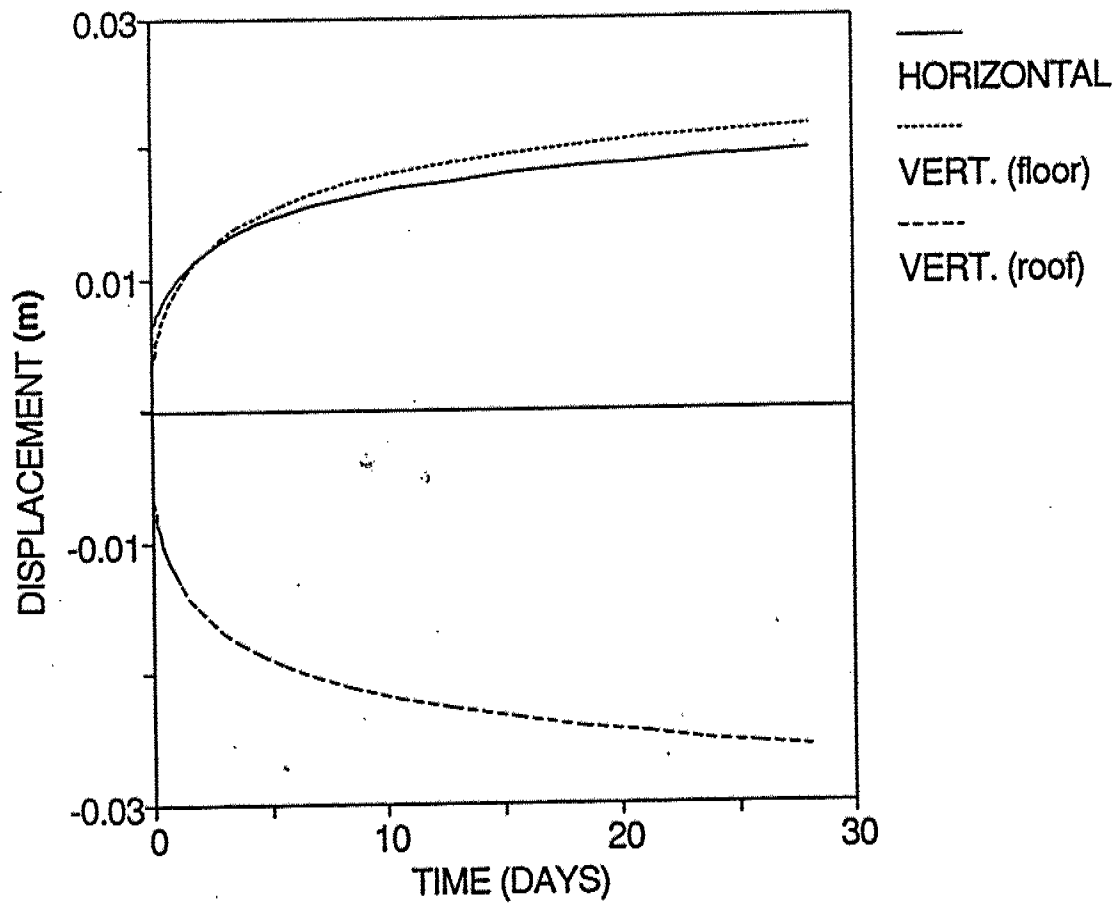



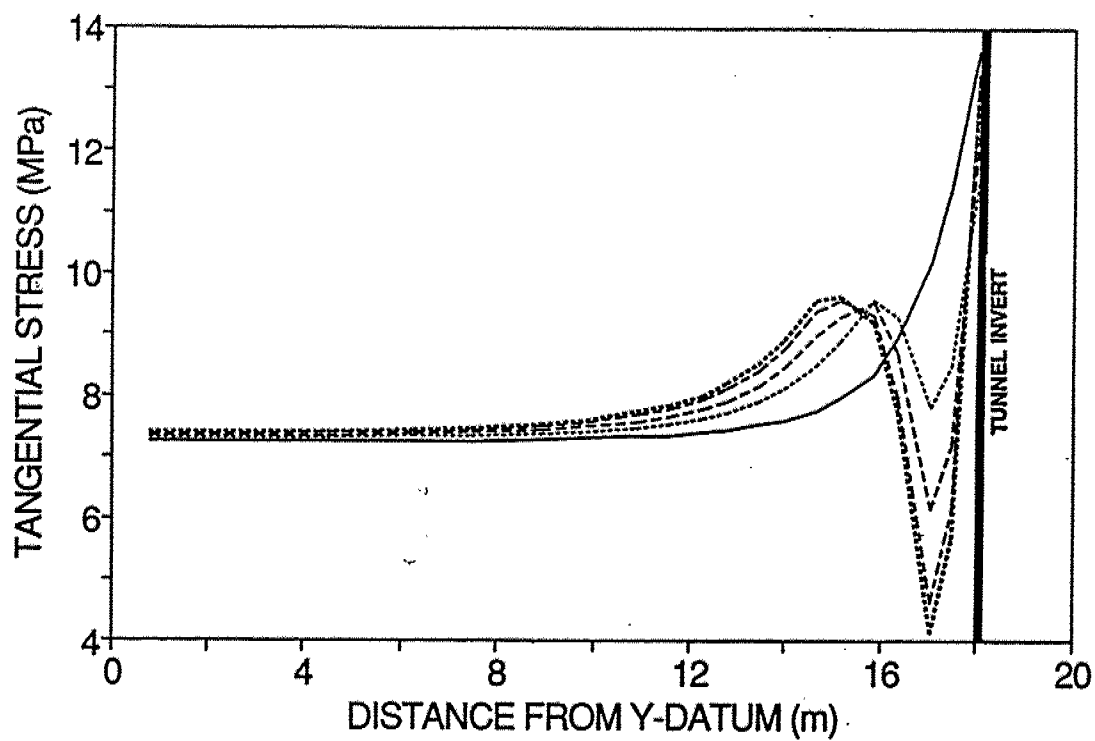
FIG 1: FINITE ELEMENT MODEL

Drawn by:	NAME	DATE		franklin geotechnical ltd.	
	PV	MAY91		PROJECT:	FIGURE:
	Checked:			G678.2	1
	Revisions:				




**FIG 2: ABSOLUTE DISPLACEMENTS VS TIME AFTER EXCAVATION**

Drawn by:	NAME	DATE		franklin geotechnical ltd.	
	PV	MAY91		PROJECT: G678.2	FIGURE: 2
	Checked:				
	Revisions:				



**FIG 3: DISTRIBUTION OF HORIZONTAL STRESSES BENEATH TUNNEL INVERT**

Drawn by:	NAME	DATE		franklin geotechnical ltd.	
	PV	MAY91		PROJECT: G678.2	FIGURE: 3
	Checked:				
	Revisions:				

**LOG OF BOREHOLE NO. 1**

1 of 7

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 & 27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE			SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE							
	GROUND ELEVATION 190.32											
0.15	TOPSOIL: Dark brown, silty clay, low organic		190									
	CLAY: Very stiff, khaki brown, silty clay with trace of sand, medium plastic, W.T.P.L.		189	1	SS	19*						
1.5			188									
	commence rotary coring			2	SS	225 mm & bouncing						
2.80												
3.0	DOLOSTONE: Tan to light grey, aphanitic to fine-grained, massive to thick bedded, numerous white chert nodules from 12 to 75 mm diameter, random vugs lined with chert/calcite in upper section, fair becoming excellent quality		187		RC							
			186	3	NQ	1750	95	54	100			
4.5			185		RC							
			184	4	NQ	3050	100	66	100			
6.0												
	Lockport Formation - Goat Island Member		183									
7.5			182									
			181	5	NQ	3050	100	67	100	2.8	2.56	49.4
9.0			180									
10.5			179		RC							
12.0			178	6	NQ	3050	100	93	100			
			177									
13.10	DOLOSTONE: Grey, medium-grained massive to thickly bedded, excellent quality, Lockport Formation - Gasport member		176									
14.45	DOLOSTONE: Decew Formation											
15.0	BOREHOLE CONTINUED ON NEXT SHEET		175									
16.5												

NOTES: \* 1) Within overburden, value denotes 'N' values in blows/0.3 m.  
2) R.Q.D. (Rock Quality Designation) is the total length of NQ core segments longer than 100 mm divided by the drill run length, expressed as a percent.

CHECKED BY: *TJ*

**LOG OF BOREHOLE NO. 1**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 &  
27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN(mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
	GROUND ELEVATION 190.32												
15.0	DOLOSTONE: Medium grey, aphanitic, massive to medium bedded, excellent quality		175		RC								
			174	7	NQ	3050	100	88	100	1.3	2.60	114.0	
16.5	16.95 Decew Formation												
	SHALE AND LIMESTONE: Dark grey interbedded shale and limestone fine-grained - highly fractured good to excellent quality, Rochester Formation		173		RC					0.4 0.2 0.7	2.43 2.19 2.59	25.72	
16.0	17.95		172	8	NQ	3050	100	92	100	1.6	2.00	76.8	
	LIMESTONE: Light grey, medium-grained, crystalline, porous, stylolitic, excellent, quality, Irondequoit Formation		171										
19.5	19.45												
	DOLOSTONE: Tan to light grey, fine-grained to aphanitic, massive, shale partings throughout, good to excellent quality, Reynales Formation		170		RC					0.6	2.96	112.2	
21.0			169	9	NQ	3050	100	90	100				
			168										
22.5	SANDSTONE: White to light grey medium to fine-grained sandstone, small scale cross bedding, layers of grey to green shale in lower section, good quality Thorold Formation		167										
24.0			166		RC					0.6	2.12	86.4	
24.60													
25.5	SHALE AND SANDSTONE: Interbedded red and green shale and sandstone, massive to thinly bedded, abundant shell fragments, shale deteriorates rapidly upon exposure, low strength		165	10	NQ	3050	100	88	100				
			164										
27.0			163	11	NQ	3050	100	88	100	5.00	2.16	9.6	
			162										
28.5	Grimsby Formation		161										
30.0	BOREHOLE CONTINUED ON NEXT SHEET		160										
31.5													

NOTES:

CHECKED BY: T.J.

## LOG OF BOREHOLE NO. 1

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE March 25, 26 &  
27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

BORING METHOD Rotary Diamond Coring, NQ Core Size

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (m)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAXIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH IN METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
	GROUND ELEVATION 190.32												
30.0	Grimsby Formation Con't		160		RC								
30.70													
	SHALE: Grey, occ. sandstone/ limestone layers, low strength, fair quality		159	12	NQ	3050	100	80	100				
31.5			158										
32.0													
33.0	Cabot Head Formation		157		RC								
34.0													
34.5			156	13	NQ	3050	100	67	100	3.6	2.56	7.0	
35.0													
36.0			155										
37.0													
38.0			154		RC								
39.0													
40.0			153	14	NQ	3050	100	73	100				
41.0													
42.0			152										
43.0													
44.0			151		RC								
45.0	becoming poor quality, clayey seams												
46.0			150	15	NQ	3050	100	45	100	4.2	2.27		
47.0													
48.0			149										
49.0													
50.0			148		RC								
51.0													
52.0	becoming fair quality												
53.0			147	16	NQ	3050	100	61	100	4.0	2.31		
54.0													
55.0			146										
56.0													
57.0													
58.0													
59.0													
60.0	BOREHOLE CONTINUED ON NEXT SHEET		145										
61.0													
62.0													
63.0													
64.0													
65.0													

NOTES:

CHECKED BY: T.J.



## LOG OF BOREHOLE NO. 1

PROJECT IONA SANTIARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 & 27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
45	GROUND ELEVATION 190.32												
	Cabot Head Formation		145		RC								
			144	17	NQ	3050	100	53	100				
			143										
40			142		RC								
			141	18	NQ	3050	100	81	100				
49.15	DOLOSTONE: Buff to light grey, dolostone with interbedded shale good quality Manitoulin Formation		140							0.4	2.83	41.9	
50.35	SANDSTONE: White, massive to finely bedded, quartz rich, separation of core along thin shale fragments parallel to bedding, fair quality Whirlpool Formation		139		RC								
			138	19	NQ	3050	100	64	100	5.5	2.12	94.2	
			137										
53.75	SHALE: Red and green mottled, massive, occ. calcite infilling, poor quality		136		RC								
54	Queenston Formation		135	20	NQ	3050	100	30	100				
			134										
57	becoming fair quality		133		RC								
			132	21	NQ	3050	100	71	100				
			131										
60	BOREHOLE CONTINUED ON NEXT SHEET		130										

NOTES:

CHECKED BY: T.J. Garde

**LOG OF BOREHOLE NO. 1**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 &  
27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (m)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAXIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
60	GROUND ELEVATION 190.32												
	QUEENSTON FORMATION		130		RC								
			129	22	NQ	3050	100	85	100				
	becoming good quality		128										
63			127		RC								
			126	23	NQ	3050	100	88	100				
			125										
66			124		RC								
	become fair quality		123	24	NQ	3050	100	70	100				
			122										
69			121		RC								
			120	25	NQ	3050	100	73	100				
			119										
72			118		RC								
			117	26	NQ	3050	100	54	100				
			116										
75	BOREHOLE CONTINUED ON NEXT SHEET		115										

NOTES:

CHECKED BY: *TJ*

**LOG OF BOREHOLE NO. 1**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE March 25, 26 &  
27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

BORING METHOD Rotary Diamond Coring, NQ Core Size

TECHNICIAN M. Rapsey

SOIL PROFILE			SAMPLES		RUN(mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER								
75	GROUND ELEVATION 190.32											
	QUEENSTON SHALE		115									
	becoming good quality		114	27	RC	3050	100	83				
					NQ							
			113									
78												
			112									
					RC							
			111	28	NQ	3050	100	64				
			110									
81												
	becoming excellent quality		109									
					RC							
			108	29	NQ	3050	100	95	2.5	2.68	13.3	
			107									
84												
			106									
					RC							
			105	30	NQ	3050	100	96				
			104									
87												
			103									
					RC							
			102	31	NQ	3050	100	100				
			101									
90												
	BOREHOLE CONTINUED ON NEXT SHEET		100									

NOTES:

CHECKED BY: *TJG*

**LOG OF BOREHOLE NO. 1**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive Royal Avenue, Hamilton, Ontario



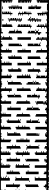
BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 & 27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE		SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAXIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION								
	GROUND ELEVATION 190.32										
90	Queenston Shale		100								Tunnel Obvert 100.2
			99	RC							
			32	NQ	3050	100	95	100			
			98								
93	becoming fair quality		97								Tunnel Invert 95.8
			96	RC							
			33	NQ	3050	100	55	100			
			95								
96	becoming good quality		94								
			93	RC							
			34	NQ	3050	100	85	100			
			92								
99	99.15 BOREHOLE TERMINATED AT 99.15 m DEPTH		91								

NOTES:

CHECKED BY: *TJG*

## LOG OF BOREHOLE NO. 2

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 17, 18 &  
19, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
	GROUND ELEVATION 120.00												
0.15	TOPSOIL: Dark brown silty clay, low organic												
	CLAY TILL: Brown, silty clay, some sand, medium plastic		119	1	SS	19*							
1.35	SHALE: Red and grey mottled shale, poor rock quality, low strength, strength increases with depth		118		RC								
	Queenston Formation		117	2	NQ	3050	100	33	100				
			116										
			115		RC								
6	becoming fair rock quality		114	3	NQ	3050	83	66	100				
			113										
			112		RC								
9			111	4	NQ	1500	100	59	100				
			110		RC								
			109	5	NQ	2315	86	55	100				
			108		RC								
12	becoming good rock quality		107	6	NQ	3050	98	85	100				
			106										
			105										
15	BOREHOLE CONTINUED ON NEXT SHEET												

- NOTES: \* 1) Within overburden, value denotes 'N' values in blows/0.3 m.  
2) R.Q.D. (Rock Quality Designation) is the total length of NQ core segments longer than 100 mm divided by the drill run length, expressed as a percent.

CHECKED BY: T.J.

**LOG OF BOREHOLE NO. 2**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 17, 18 &  
19, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
15.0	GROUND ELEVATION 120.00												
	Queenston Formation												
			104	7	RC								
					NQ	1675	100	86	100				
16.5	becoming excellent rock quality		103	8	RC								
					NQ	2440	98	98	100	1.7	2.56	8.61	
18.0			102										
			101										
19.5	becoming fair rock quality				RC								
			100	9	NQ	3050	96	73	100	0.5	2.52	19.1	
21.0			99										
			98										
22.5	becoming excellent rock quality				RC					3.51	2.51		
			97	10	NQ	3050	100	96	100	0.7	2.32	29.2	
24.0			96										
			95										
25.5	becoming fair				RC								
			94	11	NQ	3050	100	73	100	2.6	2.25		
27.0			93										
28.05	BOREHOLE TERMINATED AT 28.05 m DEPTH		92										

Tunnel Obvert 99.0

Tunnel Invert 94.6

NOTES:

CHECKED BY: TA

**LOG OF BOREHOLE NO. 3**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE April 2 & 3, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE			SAMPLES		RUN (mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )	UNIAXIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	NUMBER	TYPE								
	GROUND ELEVATION 106.02											
	Borehole advanced without sampling to bedrock surface											
	From cuttings - Clay Till											
1.5												
3.0												
3.3	Commence Rotary Drilling											
	SHALE: Red, massive, excellent rock quality becoming good											
			102	RC								
4.5			1	NQ	1220	92	100	100				
			101	RC								
6.0	Queenston Formation		100	2	NQ	3050	100	90	100			
			99									
7.5			98	RC								
			97	3	NQ	3050	100	76	100	4.7	2.21	
9.0									0.66	2.48	10.30	
9.30			96					0				
10.5	very poor rock quality, very weak shale		95	RC								
			94	4	NQ	3050	100	88	100			
12.0	becoming good rock quality		93									
			92	RC								
13.5	becoming excellent rock quality		91	5	NQ	1525	100	93	100			
15.0			90									
	BOREHOLE TERMINATED AT 15.40 m DEPTH											

Tunnel Obvert 98.6

Tunnel Invert 94.2

NOTES: 1) R.Q.D. (Rock Quality Designation) is the total length of NQ core segments longer than 100 mm divided by the drill run length, expressed as a percent.

CHECKED BY: *TJ*

## LOG OF BOREHOLE NO. 4

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE March 21, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

BORING METHOD Continuous Flight Solid Stem Augers

TECHNICIAN M. Rapsey

[illegible]

NOTES:

CHECKED BY: T.J.



**LOG OF BOREHOLE NO. 5**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

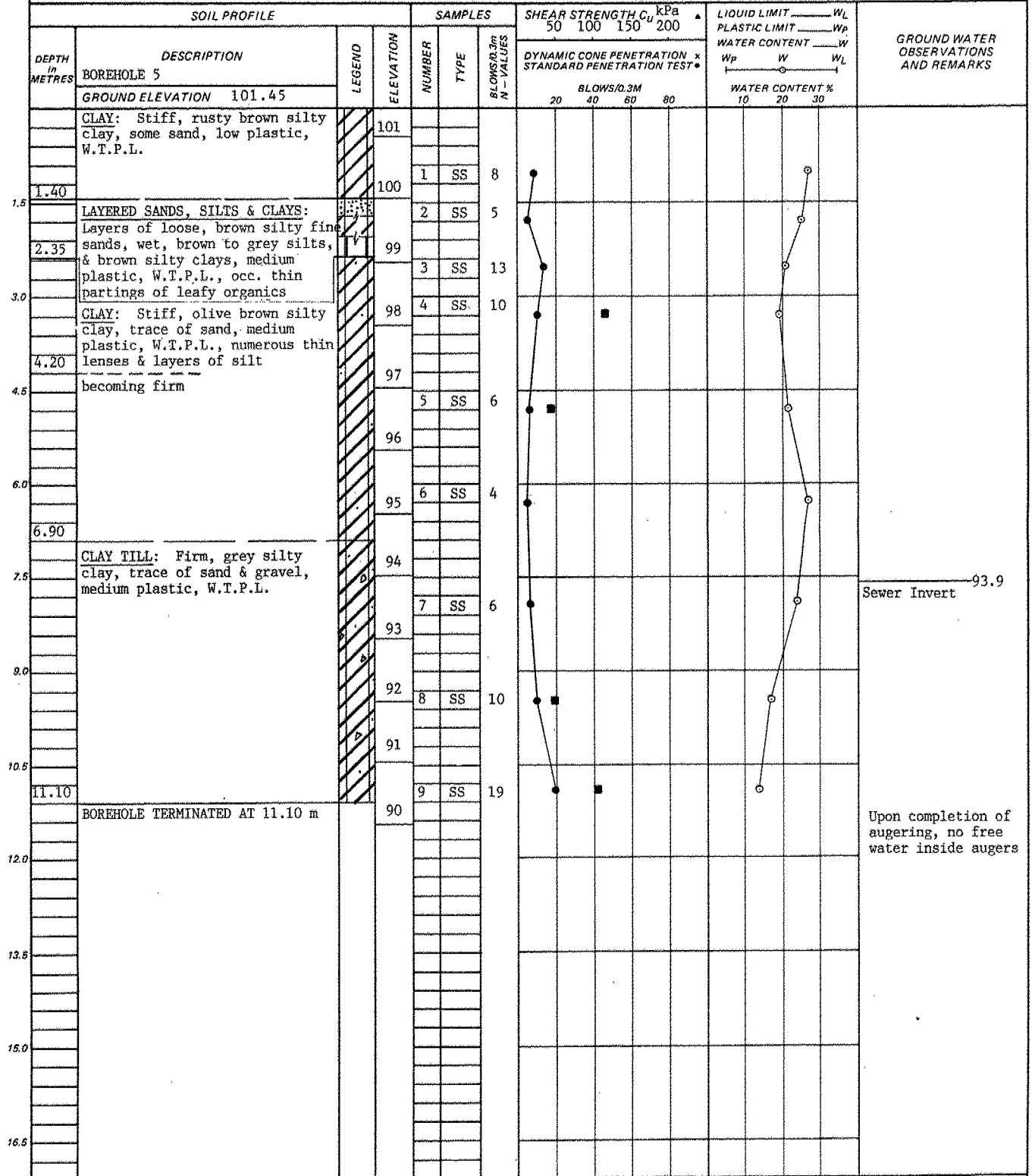
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE January 31/91

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey



Sewer Invert -93.9

Upon completion of augering, no free water inside augers

NOTES:

CHECKED BY: T.J.

**LOG OF BOREHOLE NO. 6**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

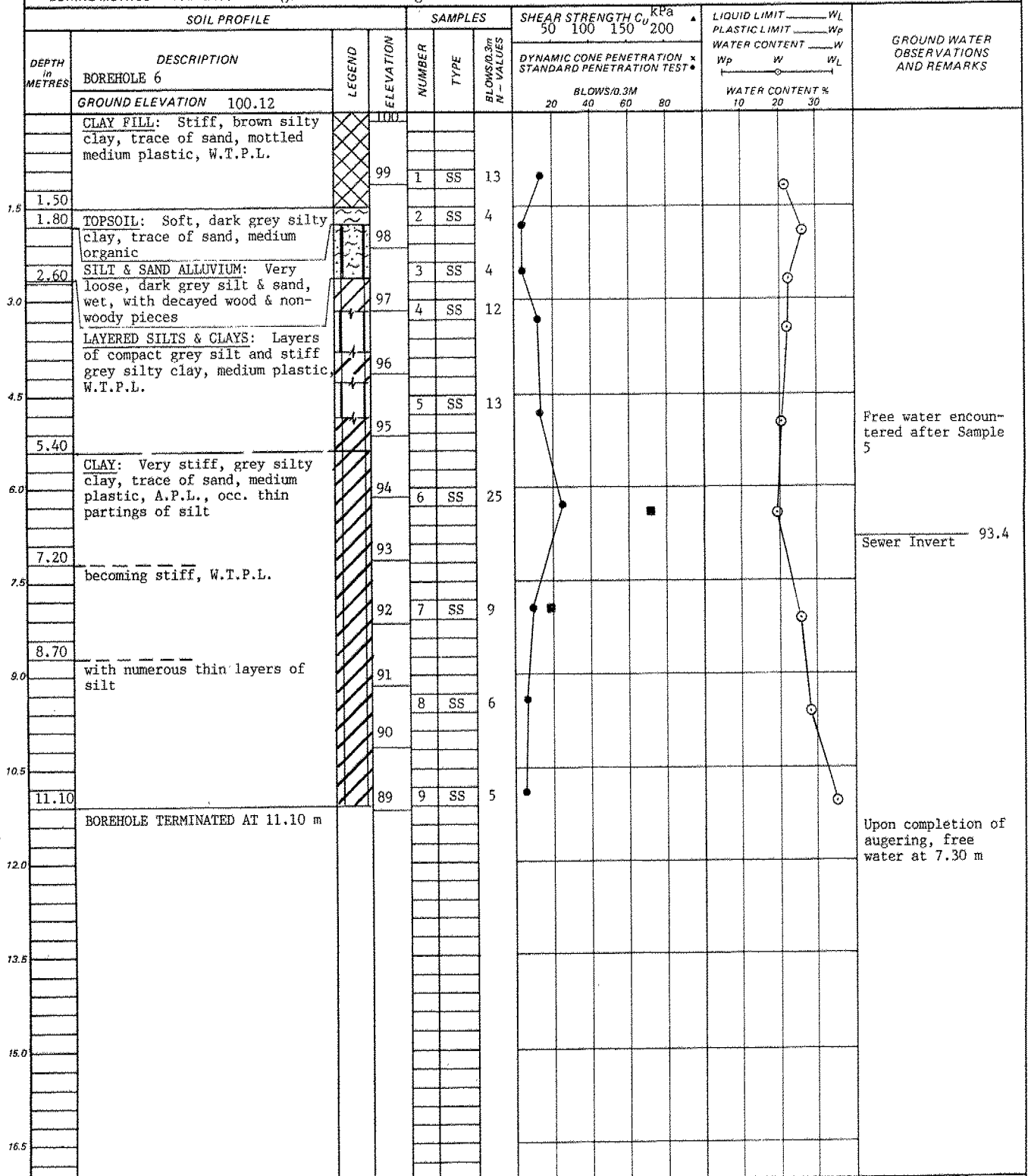
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE January 30/91

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsy



NOTES:

CHECKED BY: *[Signature]*

**LOG OF BOREHOLE NO. 7**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

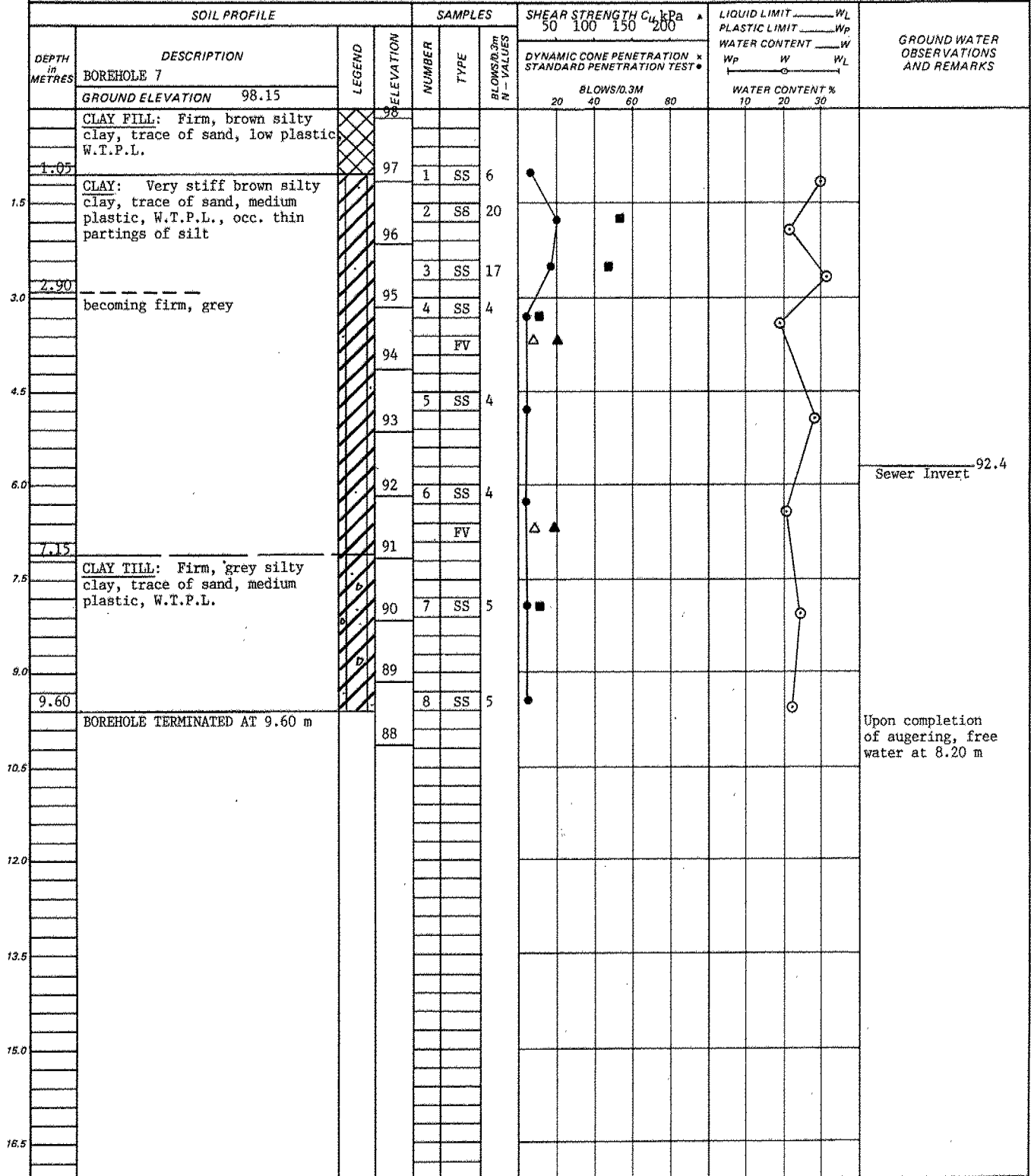
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE January 30/91

OUR PROJECT NO. 91HP007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: T.J.

## LOG OF BOREHOLE NO. 8

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

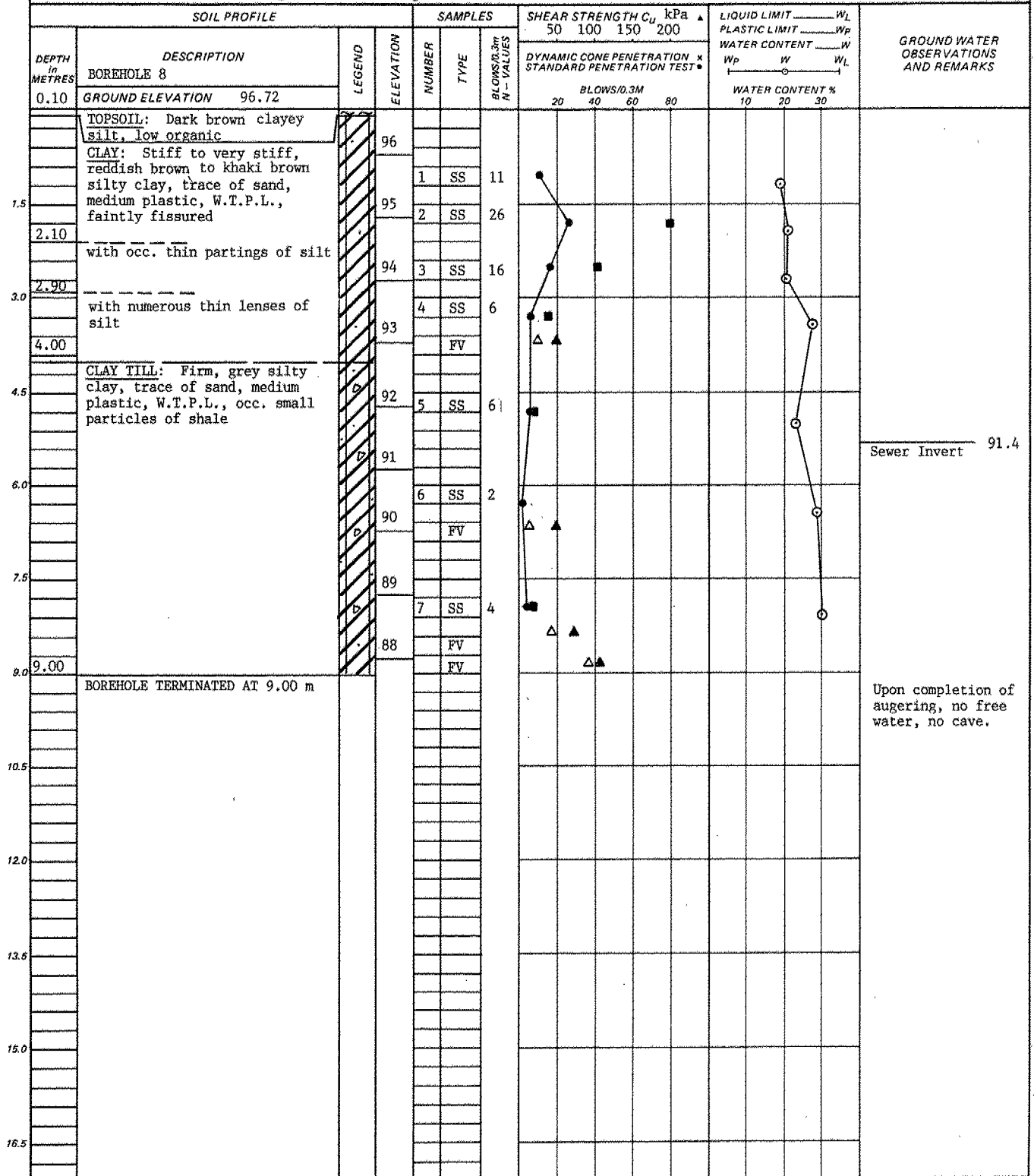
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE January 30/91

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: *TJG*

## LOG OF BOREHOLE NO. 9

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

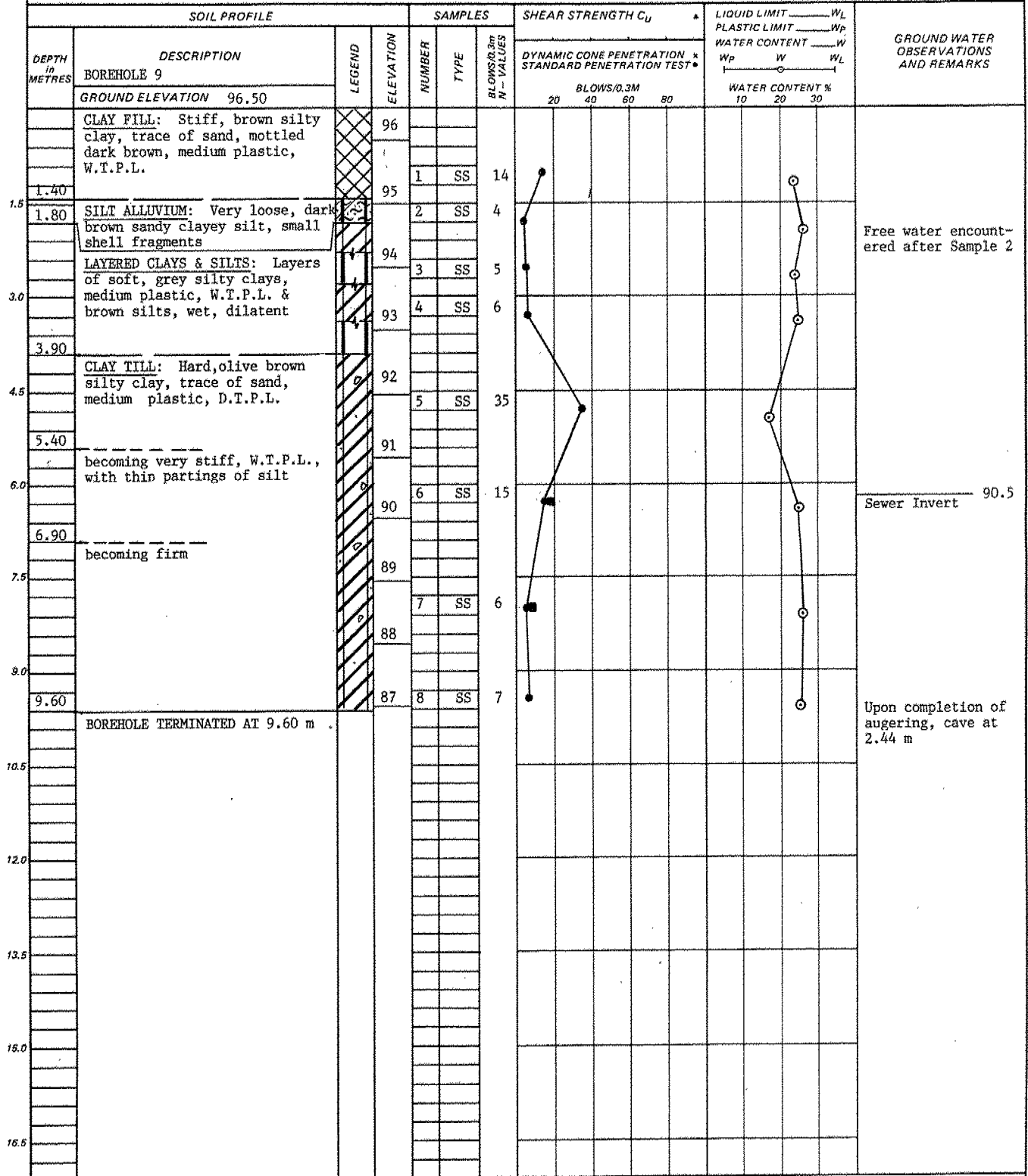
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE Jan. 30/91

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: *TJG*

**LOG OF BOREHOLE NO. 10**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

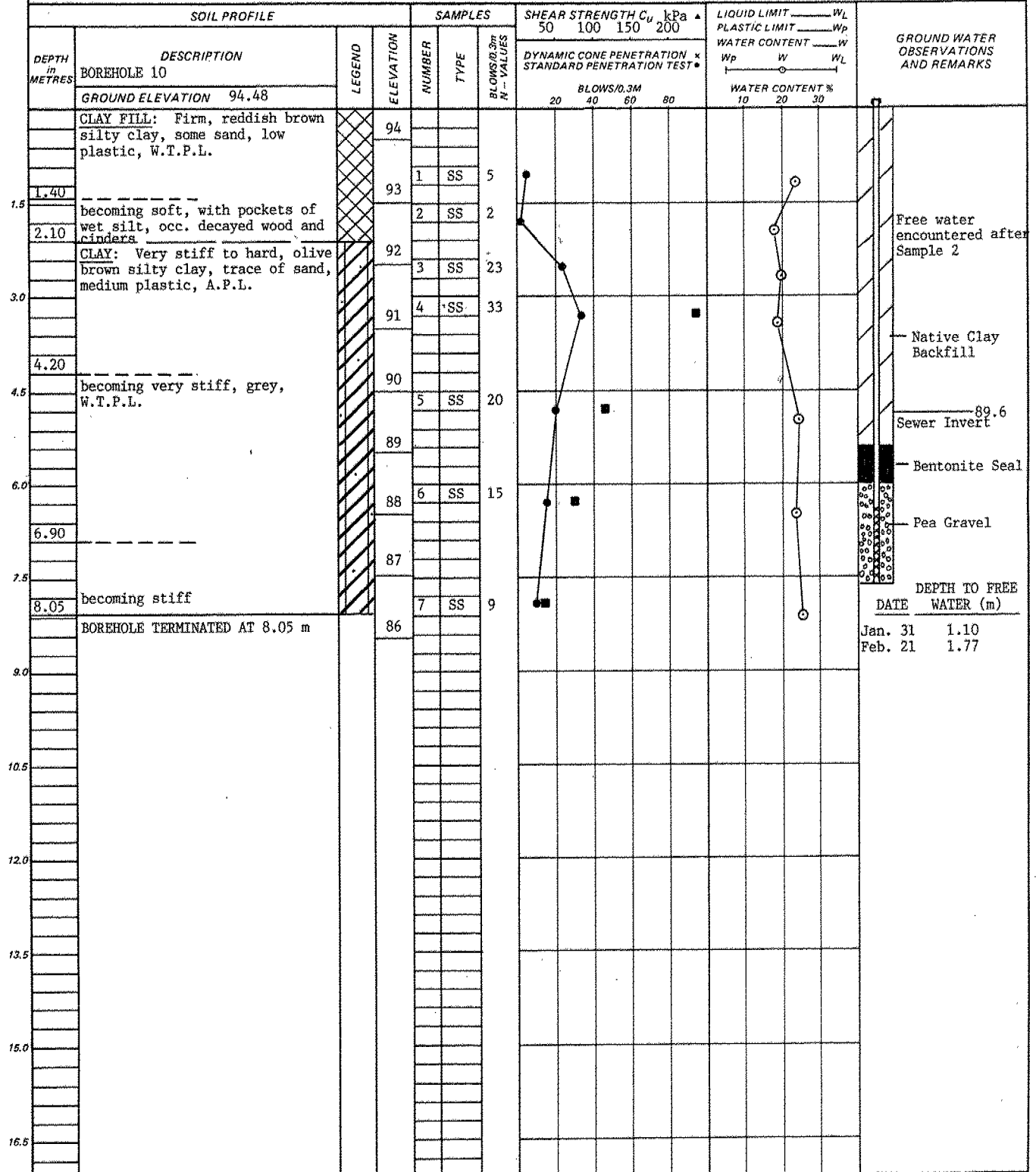
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE January 31/91

OUR PROJECT NO. 91HF007

ENGINEER T.G. Garde

TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: *TG*

**LOG OF BOREHOLE NO. 11**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

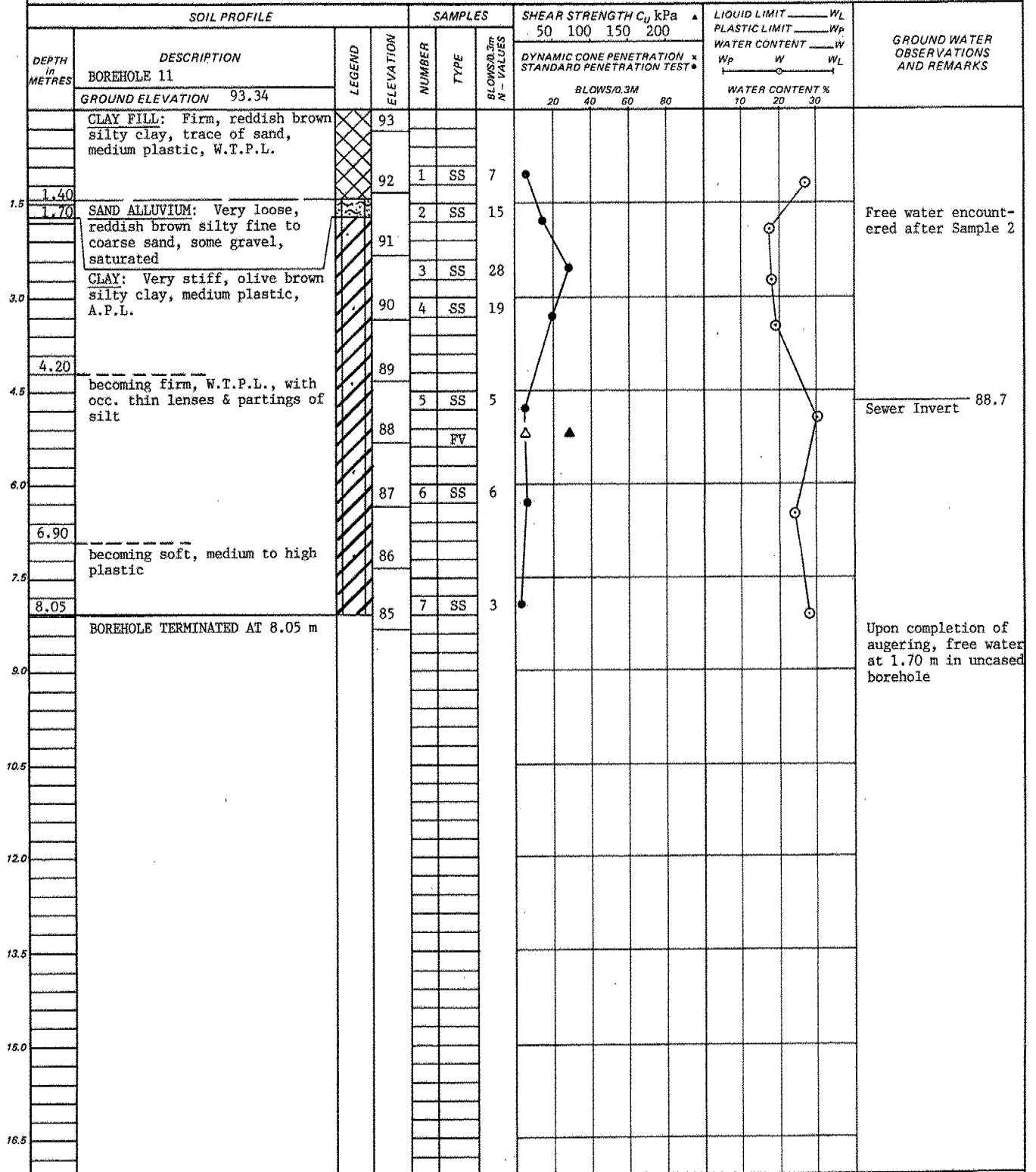
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT NO. 91HF007

BORING DATE January 31/91

ENGINEER T.G. Garde

TECHNICIAN M. Rapsey

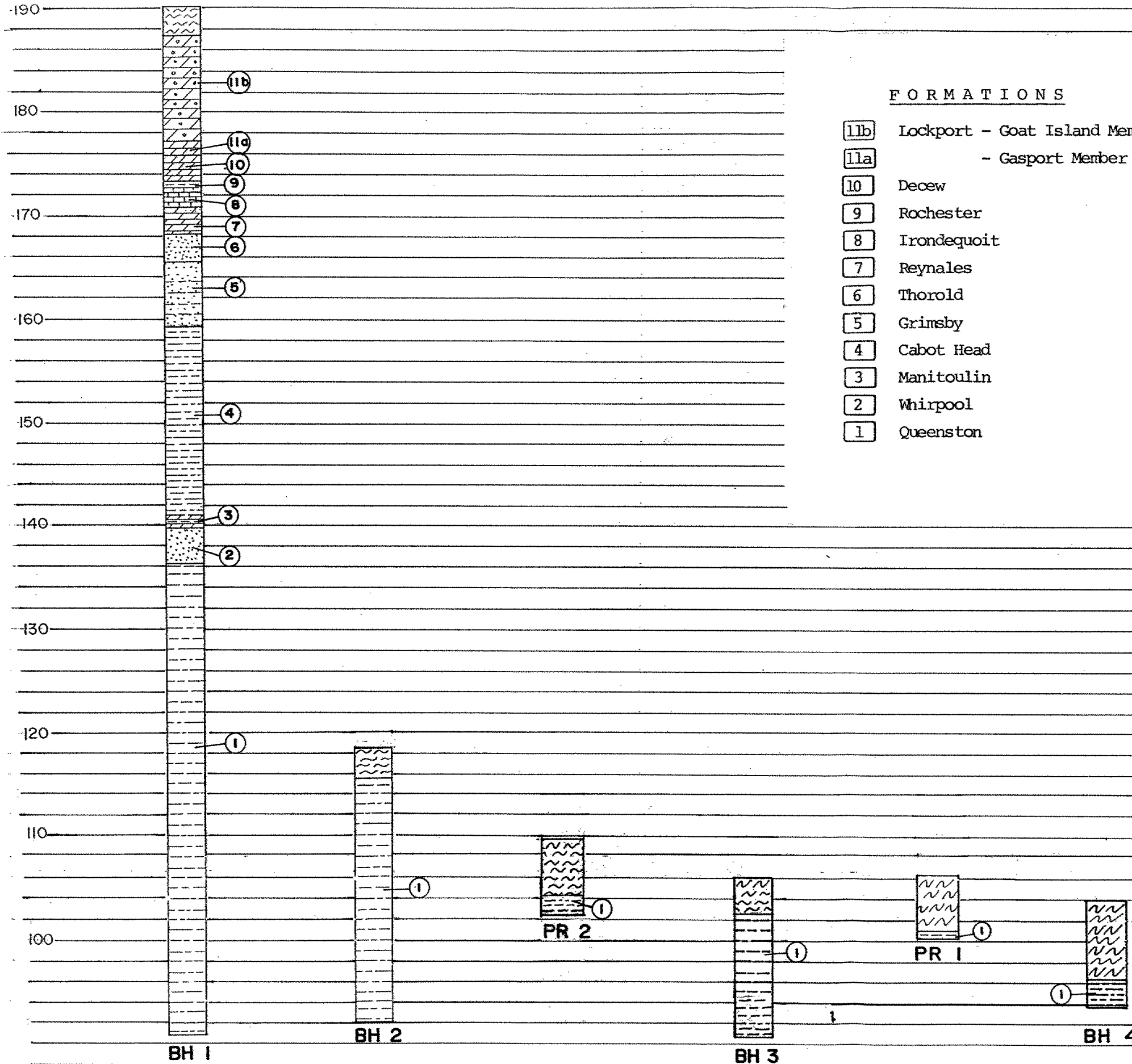


NOTES:

CHECKED BY: *TG*

SOUTH → NORTH

ELEVATION (m.)



# FORMATIONS

- 11b Lockport - Goat Island Member
- 11a - Gasport Member
- 10 Decew
- 9 Rochester
- 8 Irondequoit
- 7 Reynales
- 6 Thorold
- 5 Grimsby
- 4 Cabot Head
- 3 Manitoulin
- 2 Whirpool
- 1 Queenston

## LEGEND:

- DOLOSTONE WITH CHERT NODULES
- DOLOSTONE
- SHALE AND LIMESTONE
- DOLOSTONE AND SHALE
- SANDSTONE WITH CROSS BEDDING
- SANDSTONE AND SHALE
- SHALE

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

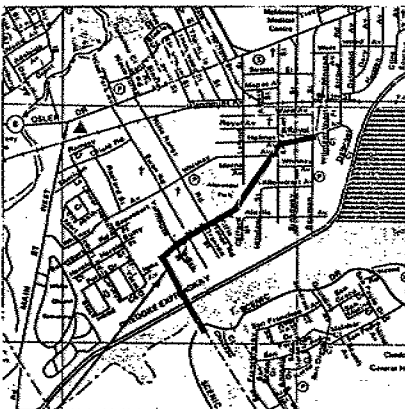
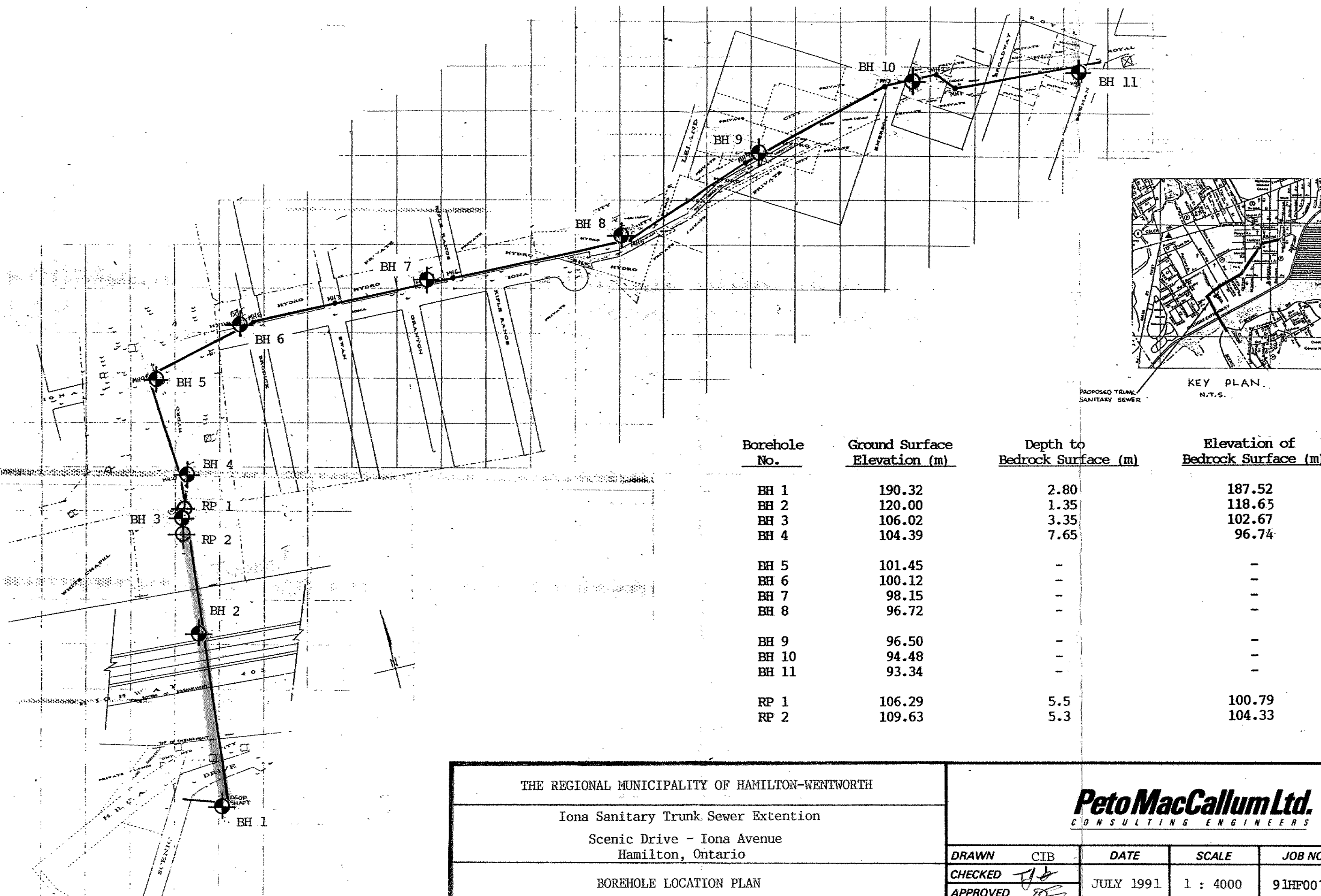
Geotechnical Investigation for Iona Sanitary  
Trunk Sewer Extension  
Hamilton, Ontario

SOIL AND ROCK PROFILE

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS

DRAWN	CIB	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED	TJA	JULY 1991	N.T.S.	91HF007	2
APPROVED	RE				

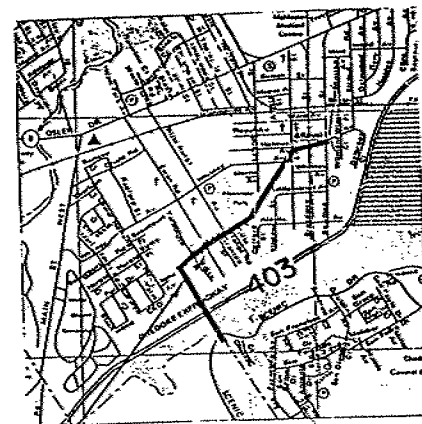
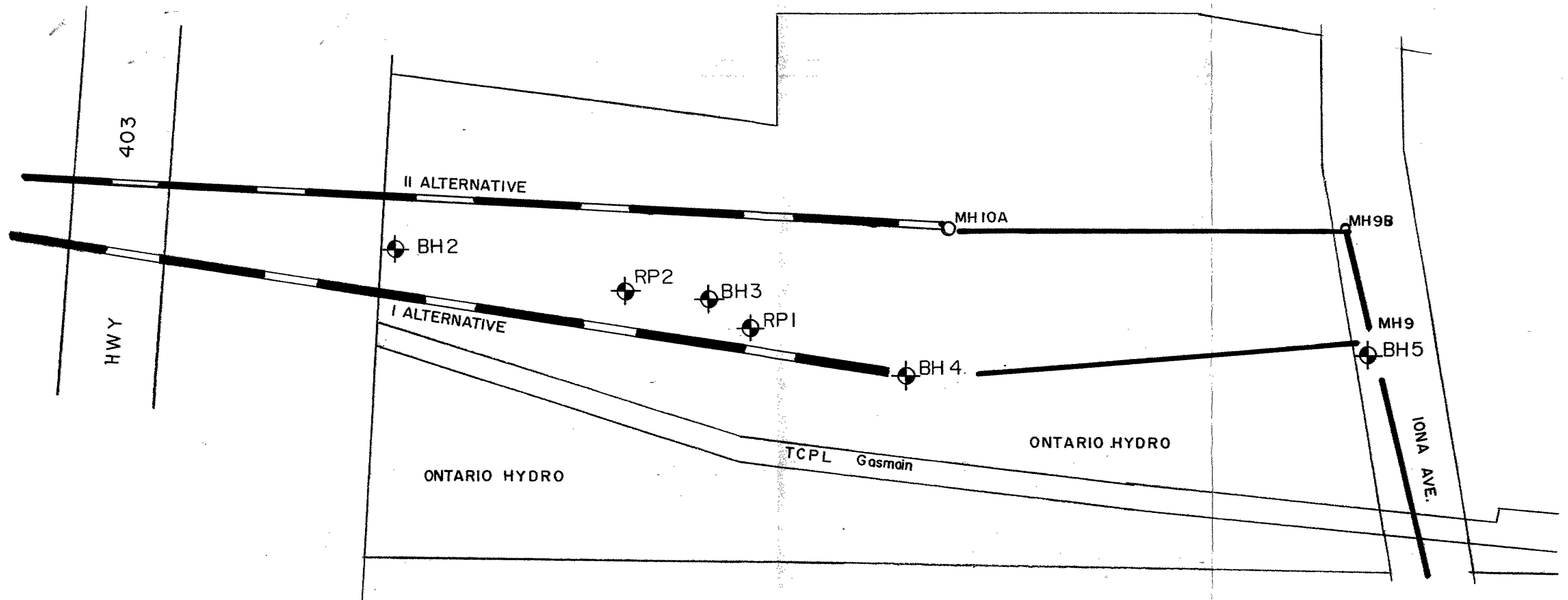




KEY PLAN  
N.T.S.

Borehole No.	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Elevation of Bedrock Surface (m)
BH 1	190.32	2.80	187.52
BH 2	120.00	1.35	118.65
BH 3	106.02	3.35	102.67
BH 4	104.39	7.65	96.74
BH 5	101.45	-	-
BH 6	100.12	-	-
BH 7	98.15	-	-
BH 8	96.72	-	-
BH 9	96.50	-	-
BH 10	94.48	-	-
BH 11	93.34	-	-
RP 1	106.29	5.5	100.79
RP 2	109.63	5.3	104.33

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH		<b>Peto MacCallum Ltd.</b> CONSULTING ENGINEERS					
Iona Sanitary Trunk Sewer Extention							
Scenic Drive - Iona Avenue		DRAWN	CIB	DATE	SCALE	JOB NO.	DRAWING NO.
Hamilton, Ontario		CHECKED	<i>[Signature]</i>	JULY 1991	1 : 4000	91HF007	1
BOREHOLE LOCATION PLAN		APPROVED	<i>[Signature]</i>				



PROPOSED TRUNK  
SANITARY SEWER

KEY PLAN  
N.T.S.

Borehole No.	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Elevation of Bedrock Surface (m)
BH 2	120.00	1.35	118.65
BH 3	106.02	3.35	102.67
BH 4	104.39	7.65	96.74
BH 5	101.45	-	-
RP 1	106.29	5.5	100.79
RP 2	109.63	5.3	104.33

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH				
IONA SANITARY TRUNK SEWER EXTENSION HWY 403-IONA AVE.				
HAMILTON, ONTARIO				
BOREHOLE LOCATION PLAN				
<b>Peto MacCallum Ltd.</b> CONSULTING ENGINEERS				
DRAWN <i>TK</i>	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED <i>TG</i>	NOV '91	1:1000	91HF007	2
APPROVED <i>TG</i>				

SUPPLEMENTARY ROCK CORING  
IONA SANITARY TRUNK SEWER  
HAMILTON, ONTARIO  
FOR  
THE REGIONAL MUNICIPALITY OF  
HAMILTON-WENTWORTH

Letters dated:

June 23, 1992  
July 9, 1992  
August 20, 1992  
September 2, 1992  
September 16, 1992  
November 24, 1992

Distribution:

4 cc: Client  
1 cc: PML Hamilton  
1 cc: PML Toronto

Job No. 91HF007A

# *Peto MacCallum Ltd.*

CONSULTING ENGINEERS

June 23, 1992

Our Ref: 91HF007A

Mr. H. Johnson, P.Eng.  
R.V. Anderson Associates Limited  
Suite 401, 1210 Sheppard Avenue East  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Supplementary Rock Coring  
Iona Sanitary Trunk Sewer  
Hamilton, Ontario

Further to your facsimile transmission dated June 17, 1992, we are pleased to provide our cost estimate for additional coring at the above noted project site.

We understand that the additional boreholes are required for further evaluation of the shale quality. It is noted that the shale was of very poor quality between elevations 91.6 to 93.2 at rock probe RP 2 and elevation 95.2 to 96.7 in borehole BH 2.

The estimated costs to locate and drill boreholes at the locations indicated in your fax, including engineering supervision, piezometer installation and reporting are as follows:

At RP 1 - approximately 5.5 m of overburden  
augering and 10 m of rock coring.....\$ 4 350.00

Between BH 2 and RP 2 - approximately 6 m of  
overburden augering and 20 m of  
rock coring.....\$ 5 900.00

Monitoring and reporting of water levels can be carried out in conjunction with measurements in the existing piezometers.

...2

***PetoMacCallum Ltd.***  
CONSULTING ENGINEERS

June 23, 1992, P2

91HF007A

Should you require any additional information, please do not hesitate to contact this office.

Sincerely

Peto MacCallum Ltd.



*ln* Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

MRA:hb



***Peto MacCallum Ltd.***  
C O N S U L T I N G   E N G I N E E R S

July 9, 1992

Our Ref: 91HF007A

The Regional Municipality of  
Hamilton-Wentworth  
c/o Mr. H. Johnson, P.Eng.  
1210 Sheppard Avenue East  
Suite 401  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Iona Sanitary Trunk Sewer  
Hamilton, Ontario

We have reviewed your April 6, 1992 letter concerning this project and are pleased to make the following comments.

Franklin Geotechnical Ltd. was retained by Peto MacCallum Ltd. to provide specialist advice concerning the rock mechanics aspects of the tunnel design. The February 13, 1992 letter from Franklin Geotechnical Ltd. (FGL) to R.V. Anderson Associates Limited (RVA) described the procedures that would be adopted during this study.

The magnitude of the rock pressure exerted on the secondary tunnel liner during convergence of the rock following excavation of the tunnel is, in general, dictated by:

- i) The in situ stresses in the rock mass.
- ii) The mechanical properties of the rock mass.
- iii) The type and properties of the tunnel support.
- iv) The method of excavation and construction of the tunnel.

The in situ stress in the rock mass and the mechanical properties of the rock were described in our report dated August, 1991.

...2

H. Johnson, July 9, 1992, P2

91HF007A

In order to assess the fully mobilized horizontal and vertical pressures acting on the secondary tunnel liner, it is necessary that the type of liner and excavation technique be known. Use of traditional liner design concepts are, in our opinion, not applicable to the highly stressed rocks that exist in Southern Ontario.

Assessment of the pressures acting on the liner at the design stage is an interactive process that must consider the four (4) factors noted above, particularly the interaction between the liner and the rock. Computer modelling techniques are available to evaluate the time dependent stresses that will be imposed on the secondary tunnel liner by the rock. However, the creep characteristics of Southern Ontario rocks are extremely variable; monitoring of the movements following excavation of a tunnel is an essential input for meaningful results to be provided by the computer model.

The following comments are intended to be used during preliminary design of the tunnel, selection of a suitable construction procedure and liner type. When this is complete, a more refined estimate of the rock pressures acting on the secondary tunnel liner can be provided by using computer modelling techniques.

The vertical component of stress in the rock mass before tunnelling increases with depth below ground surface. The vertical stress ( $\sigma_v$ ) can be computed from the following equation:

$$\sigma_v = \gamma_t h$$

where h = depth below ground surface

$$\begin{aligned}\gamma_t &= 20.5 \text{ kN/m}^3 \text{ in overburden} \\ &= 26.0 \text{ kN/m}^3 \text{ in rock}\end{aligned}$$

The actual vertical component of rock stress acting on the secondary tunnel liner will be much less than the in situ ground stress, due to arching in the rock mass above the tunnel, inward movements that take place due to the horizontal stresses that exist in the rock, as well as the primary liner (rock bolts, shotcrete). The transfer of stresses from the rock mass to the tunnel is a time dependent process. The mobilized vertical component of stress will depend on the effectiveness of the primary liner as well as the time lapse between excavation of the tunnel and installation of the liner

...3

H. Johnson, July 9, 1992, P3

91HF007A

For preliminary design, a value of zero should be used for the vertical component of stress in the rock adjacent to the tunnel before placement of the secondary liner.

The horizontal component of stress could be as high as the strength of the rock. Therefore, a value of 15 MPa is recommended during preliminary design of the liner.

Deformation of the bedrock and convergence of the tunnel is a time dependent process governed by the creep characteristics of the rock, the time delays between excavation and installation of the liner as well as the rigidity of the liner.

Based on our experience with previous tunnels in Southern Ontario, we expect the horizontal convergence to be in the order of 20 to 60 mm during the first 30 days after excavation. Ultimate closure over the next year or two could be twice this value.

Since the properties of the rock, particularly the time dependent movements and consequent stresses imposed on the secondary liner are quite variable, the secondary liner is normally designed to withstand the water pressures and the thrust of the TBM if this construction technique is employed. The recommended construction procedure is to install the primary liner immediately after excavation and monitor convergence in the tunnel as excavation proceeds. The final liner should be installed when excavation is complete and monitoring indicates the rate of convergence has reached an acceptable level. Convergence monitoring can also be used to evaluate the displacements and stresses that will be imposed on the secondary liner.

In regards to the two shafts, a horizontal stress value of up to 10 MPa is recommended in the Queenston shale and up to 15 MPa in the upper dolostone sequences. Closure of these shafts is expected to be much less than the tunnel, less than 20 mm in the Queenston Shale and 10 mm in the dolostone sequences. Fifty percent of the closure should occur within 30 days following excavation.

Deformation measurements should be carried out during excavation of the tunnel and shaft to confirm the magnitude of predicted convergences, as well as the creep characteristics and stress levels in the rock.



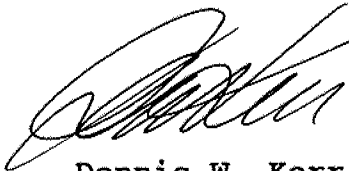
H. Johnson, July 9, 1992, P4

91HF007A

We trust the contents of this letter are sufficient to enable you to complete preliminary design. If further information is required, however, we strongly recommend that a meeting be called to deal with any further questions you may have.

Sincerely

Peto MacCallum Ltd.



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton



DWK:rp

- 1 cc: Region of Hamilton-Wentworth; J. Koshurba
- 1 cc: R.V. Anderson Associates; H. Johnson
- 1 cc: Franklin Geotechnical Ltd.

***PetoMacCallum Ltd.***  
C O N S U L T I N G   E N G I N E E R S

September 2, 1992

Our Ref: 91HF007A

The Regional Municipality of  
Hamilton-Wentworth  
c/o Mr. H. Johnson, P.Eng.  
R.V. Anderson Associates  
1210 Sheppard Avenue East  
Suite 401  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Groundwater Impact Summary  
Iona Sanitary Trunk Sewer  
Hamilton, Ontario

This letter summarizes our comments regarding the anticipated impact of the Iona Sewer project on groundwater conditions in the Niagara Escarpment.

A detailed discussion of the geology and groundwater conditions at the site, our assessment of the impact of the project on groundwater, as well as recommendations to minimize the impact are provided in our engineering report dated August 6, 1991. A nested piezometer was installed at the top of the escarpment on July 16, 1992, and supplementary testholes were drilled along the tunnel portion of the sewer north of Hwy 403 during the period June 9 to July 21, 1992. This information was summarized in our letter dated August 20, 1992.

A site plan showing the sewer alignment, borehole locations and piezometer/water level information is provided on Drawing 2.

We conclude that the proposed sewer drop shaft and tunnel will have a negligible long term impact on existing groundwater levels. Construction period impacts are expected to be minor/localized. A summary of our assessment is provided below.

Proposed Sewer Layout

The proposed sewer installation in the escarpment vicinity will consist of a drop shaft extending some 94 m from the top of the escarpment and a tunnel section proceeding northerly under the escarpment face, the Highway 403 corridor and the slope below the highway.

...2

H. Johnson, September 2, 1992, P2

91HF007A

The drop shaft will extend through some 3 m of overburden clay till and then bedrock consisting of approximately 28 m of dolostone, limestone and sandstone, overlying two major shale formations. The lateral tunnel will be installed entirely within the lower shale unit, the Queenston formation.

#### Existing Groundwater Conditions

Groundwater levels measured in the two lower piezometers of the piezometer nest installed at the drop shaft location were about 23 m below the ground surface (elevation 167). The water level in the upper piezometer (zone of measurement 5 to 30 m) was 26.5 m below grade, near elevation 164. These measurements indicate a small upward hydraulic gradient in the upper rock formation and hydrostatic conditions below a depth of 30 m.

The Hwy 403 corridor is near elevation 142 at the proposed sewer crossing. It is probable that some seepage from the groundwater table occurs from the escarpment face above the highway. Seepage at higher levels on the escarpment face often noted during the winter/spring season probably represents surface water infiltrating through discontinuities and fractures in the upper dolomite/limestone formations.

Groundwater levels measured in three piezometers installed below Highway 403 ranged between 4 to 9 m below the ground surface. Seepage was not observed along this section of the slope during the geotechnical investigations although some groundwater discharge may occur into the small creek that exists at the base of the slope north of the proposed tunnel. The clay till overburden and Queenston shale bedrock in this section are considered to be relatively impermeable.

#### Anticipated Impact of Construction on the Groundwater Table

The rock units below the measured water table consist primarily of relatively impermeable shales. Minor sandstone units and a thin dolostone formation also exist. Although rock core samples showed no significant water bearing fractures or joints, some seepage into the drop shaft should be expected, with a potential for temporary lowering of the local water table. Some seepage from the bedding planes and fractures in the dolostone/limestone sequence above the measured water level may also occur. Water entering the shaft above Hwy 403 could result in a minor reduction in the normal discharge on to the escarpment face.

H. Johnson, September 2, 1992, P3

91HF007A

The possibility of encountering local water bearing fractures/joints in the rock and the potential for seepage from the upper dolostone/limestone sequences were identified in our geotechnical report. Recommendations were made to locally grout/seal off water bearing zones during construction and to permanently grout the annular space between the vertical drop shaft and surrounding rock.

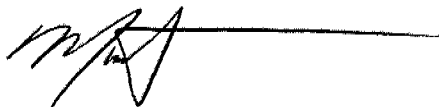
The lateral tunnel section will extend through relatively impermeable Queenston shale and hence any seepage during construction is not expected to affect groundwater levels. An impermeable barrier placed around the tunnel section would control long term seepage.

Provided these measures are implemented, installation of the trunk sewer is expected to have minimal impact on the groundwater table both in the short term during construction as well as long term following completion of construction.

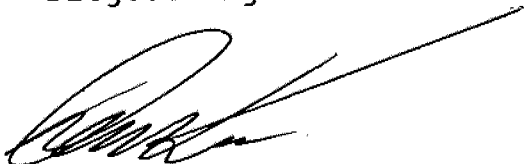
Should you require any additional information, please do not hesitate to contact this office.

Sincerely

Peto MacCallum Ltd.



Murray R. Anderson, P.Eng.  
Project Engineer



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton



MRA:rz

4 cc: R.V. Anderson Associates; H. Johnson  
1 cc: Region of Hamilton-Wentworth; J. Koshurba

# *PetoMacCallum Ltd.*

*C O N S U L T I N G   E N G I N E E R S*

September 16, 1992

Our Ref: 91HF007A

The Regional Municipality of  
Hamilton-Wentworth  
c/o Mr. H. Johnson, P.Eng.  
R.V. Anderson Associates  
1210 Sheppard Avenue East  
Suite 401  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Supplementary Comments  
Groundwater Impact Mitigation  
Iona Sanitary Trunk Sewer  
Hamilton, Ontario

This letter provides our supplementary recommendations to minimize the long-term impact of this project on existing groundwater conditions.

A detailed discussion of the geology and groundwater conditions at the site, our assessment of the impact of the project on groundwater, as well as recommendations to minimize the impact were provided in our engineering report dated August 6, 1991.

A nested piezometer was installed at the proposed drop shaft location on July 16, 1992, and supplementary testholes were drilled/piezometers installed along the tunnel portion of the sewer north of Highway 403 during the period June 9 to July 21, 1992. Factual data from this work was provided in a letter dated August 20, 1992 and included in a summary letter dated September 2, 1992.

In view of the subsequent groundwater level measurements and the site geology, we believe that measures to minimize the long-term impact of the shaft/tunnel should include the following:

- i) The annular space between the vertical drop shaft and surrounding rock should be permanently grouted along the entire length of the shaft in rock.
- ii) An impermeable barrier should be placed around the tunnel section in the Queenston shale to prevent long-term seepage between the shale and tunnel wall.

...2

**PetoMacCallum Ltd.**  
CONSULTING ENGINEERS

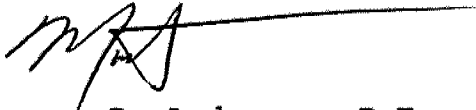
H. Johnson, September 16, 1992, P2

91HF007A

Should you require any additional information, please do not hesitate to contact this office.

Sincerely

Peto MacCallum Ltd.



Murray R. Anderson, P.Eng.  
Project Manager



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton



MRA:rz

4 cc: Region of Hamilton-Wentworth; J. Koshurba  
1 cc: R.V. Anderson Associates; H. Johnson

# ***Peto MacCallum Ltd.***

C O N S U L T I N G   E N G I N E E R S

November 24, 1992

Our Ref: 91HF007A

Mr. Stephen Szigeti, P.Eng.  
The Regional Municipality of  
Hamilton-Wentworth  
Roads Department  
71 Main Street West  
Hamilton, Ontario  
L8N 3T4

Dear Mr. Szigeti

Iona Trunk Sanitary Sewer  
Hamilton, Ontario

We have reviewed the November 4, 1992 letter from R.V. Anderson Associates as requested.

We note that design of the liner on an empirical basis is planned. The design/construction procedure calls for:

- . Excavation of the tunnel section and application of shotcrete to the surface of the rock.
- . Monitoring of time dependent movements.
- . Application of a 50 to 100 mm thick layer of compressible polyurethane spray foam material about 30 days after excavation. The time delay and foam thickness will be subject to the results of the closure monitoring.
- . Construction of a secondary concrete liner designed to withstand the groundwater pressures.

Adoption of an empirical approach to design of the tunnel liner system is consistent with that recommended in our August 1991 report and subsequent correspondence. The computer modelling referred to in our July 9, 1992 letter was to be based on closure measurements in the tunnel after excavation. Computer modelling based on actual closure measurements was considered to be the only practical means of providing the design data requested by R.V. Anderson Associates Limited.

...2

S. Szigeti, November 24, 1992, P2

91HF007A

In regards to the empirical design concepts outlined in the November 4, 1992 letter referred to previously, we make the following comments:

- i) The most practical means of designing tunnels in Southern Ontario rock is to use an empirical approach.
- ii) Placement of a compressible material between the rock and concrete has been successfully employed on several tunnels excavated in Southern Ontario rocks.

Problems with excessive movement have been experienced with this technique however if the design calls for a change from tunnelling to a cut and cover procedure near the end of the tunnel.

Greater movements near the north limit of this tunnel should be anticipated if a cut and cover technique is adopted. The construction methodology specified by the Region should be employed if sections of the tunnel in rock are constructed using cut and cover techniques.

- iii) Convergence of the rock is unlikely to be complete 30 days after completion of excavation. Closure of the tunnel should be monitored to permit assessment of the rate and magnitude of movements likely to occur after construction of the liner, the thickness of compressible material required to accommodate these movements, the consequent stress imposed on the liner and the optimum time to construct the secondary liner.

Concreting should be delayed until excavation of the tunnel is complete and start at the location where excavation commenced to permit the maximum time lapse between excavation and tunnelling.



S. Szigeti, November 24, 1992, P3

91HF007A

- iv) Preliminary design of the secondary liner should be based on the groundwater pressures. Final design should be based on the water pressures as well as the stresses likely to be imposed on the liner deduced from the convergence measurements coupled with visual examination of the primary liner.
- v) Closure of the tunnel could continue to be monitored after concreting to confirm that the design assumptions were appropriate.
- vi) If grouting between the liner and rock is planned, it should be limited to filling major voids that exist between the liner and the rock that may result in local stress concentrations; a low water cement ratio should be specified.

Reference is made to Section 6.2.1 in our August 1991 report for further comments to be considered during construction.

Should you require any additional information, please do not hesitate to contact this office.

Sincerely

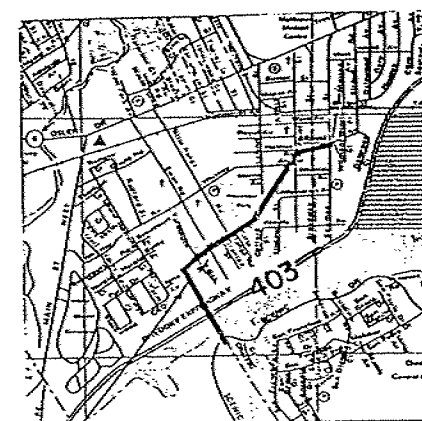
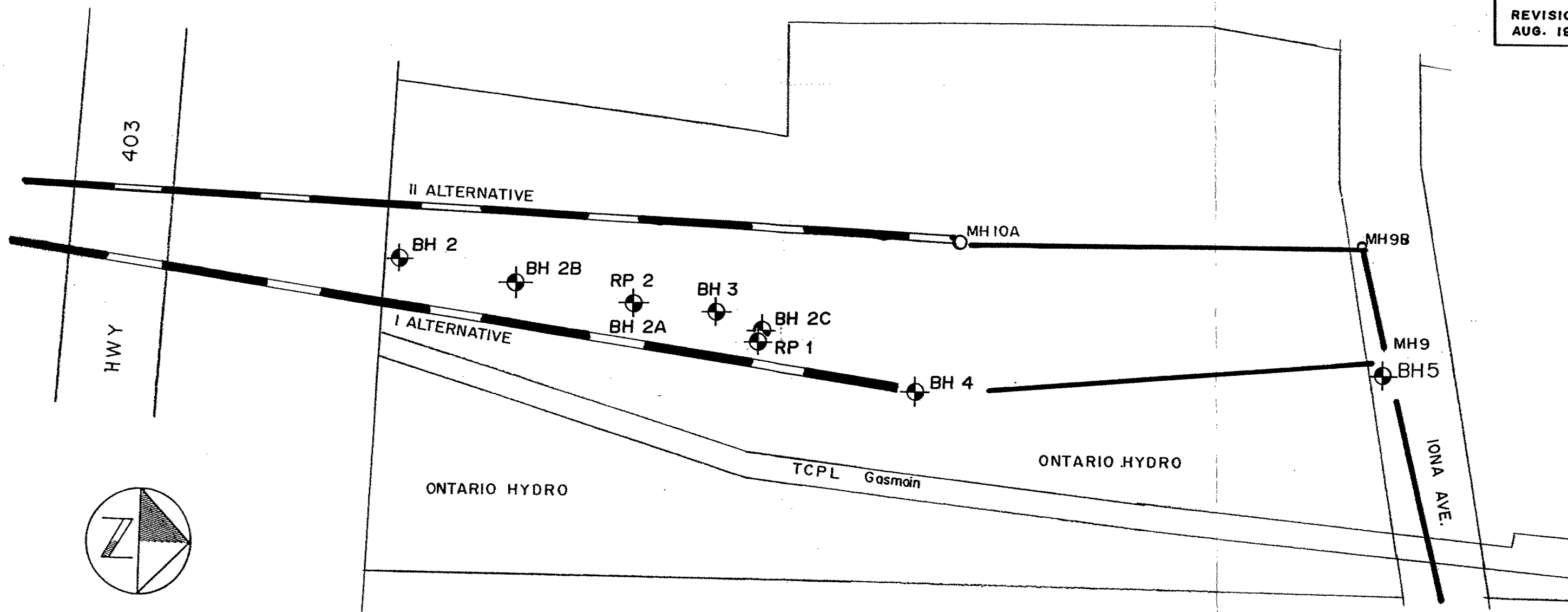
Peto MacCallum Ltd.



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

DWK:rz

4 cc: Region of Hamilton-Wentworth  
1 cc: R.V. Anderson Associates; H. Johnson



KEY PLAN  
N.T.S.

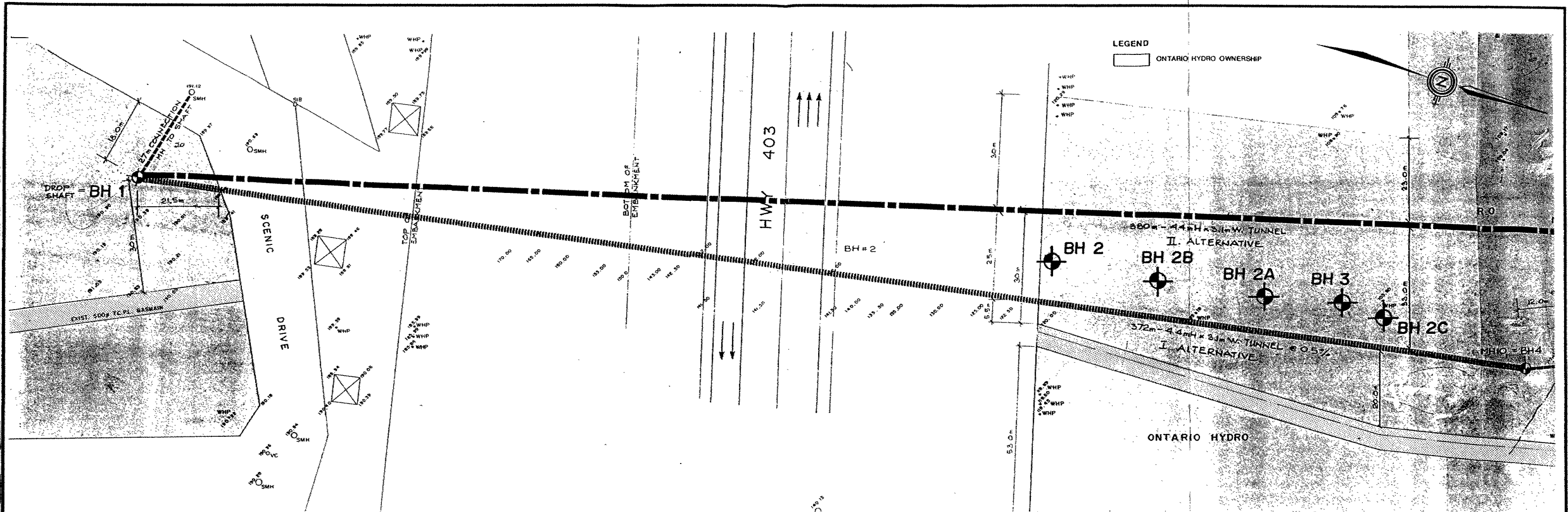
PROPOSED TRUNK  
SANITARY SEWER

Borehole No.	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
2	120.00	1.35	118.65
2A (RP2)	113.34	7.30	106.04
2B	115.21	2.15	113.06
2C (RP1)	105.82 (106.29)	6.90	98.92
3	106.02	3.35	102.67

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH  
IONA SANITARY TRUNK SEWER EXTENSION  
HWY 403-IONA AVE.  
HAMILTON, ONTARIO  
BOREHOLE LOCATION PLAN

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS

DRAWN <i>TK</i>	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED <i>TG</i>	NOV '91	1:1000	91HF007A	1
APPROVED <i>TG</i>				



Borehole No.	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
1	190.32	2.80	187.52
2	120.00	1.35	118.65
2A	113.34	7.30	106.04
2B	115.21	2.15	113.06
2C	105.82	6.90	98.92
3	106.02	3.35	102.67
4	104.39	7.65	96.74

Piezometer No.	Piezometer Details		Measured Water Level	
	Screen Depth (m)	Bentonite Seal Position (m)	Depth (m)	Elevation
1a	97.5 - 100.5	64.0 - 67.0	23.3	167.0
1b	58.0 - 61.0	34.6 - 40.7	23.2	167.1
1c	27.5 - 30.5	0.0 - 4.5	26.5	163.8
2A	18.0 - 19.5	6.7 - 7.3	9.1	104.2
2B	19.7 - 22.7	12.3 - 13.6	6.2	109.0
2C	14.3 - 15.8	7.3 - 8.5	4.1	101.7

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

IONA SANITARY TRUNK SEWER EXTENSION  
HIGHWAY 403 — IONA AVENUE  
HAMILTON, ONTARIO

BOREHOLE LOCATION PLAN

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS

DRAWN	CIB	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED	<i>[Signature]</i>	AUG. 1992	1:1000	91HF007A	2
APPROVED	<i>[Signature]</i>				

# PetoMacCallum Ltd.

CONSULTING ENGINEERS

## LOG OF BOREHOLE NO. 2B (cont'd)

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

OUR PROJECT NO. 91HP007A

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE July 13/92

ENGINEER M.R. Anderson

BORING METHOD Continuous Flight Solid Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE			SAMPLES		RUN(mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )		GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE							
	GROUND ELEVATION 115.21											
			100									
			99									
16.5			98	10	RC							
			97	11	RC NQ	760	87	80	100			
18.0			96	12	RC NQ	450	56	33	100			
18.40	becoming poor quality, 'muddy' zone between 18.40 to 18.60 m		95									
19.5			94	13	RC NQ	1525	100	30	100			
20.40	becoming good quality, occasional gypsum seams		93	14	RC NQ	1525	100	80	100			
21.0			92	15	RC NQ	760	100	80	100			
22.5												
22.70	Borehole terminated at 22.70 m											
23.0												
24.5												
26.0												
27.5												
29.0												
29.5												

Tunnel Obvert 97.8

Poor quality zone

Tunnel Invert 94.5

NOTES:

CHECKED BY: *M.T.*

**LOG OF BOREHOLE NO. 2C**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

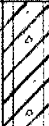



BORING DATE July 17/92

OUR PROJECT NO. 91HF007A

ENGINEER M.R. Anderson

BORING METHOD Continuous Flight Solid Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE							
	GROUND ELEVATION 105.82											
1.5	Borehole advanced without sampling to bedrock surface		105									Native backfill
			104									
			103									
3.0			102									
			101									
4.5	met refusal and attempted to core; shale fragment in clay till		100									
6.0			99									
6.90												
7.5	SHALE: Successive zones of excellent to poor quality, red shale, occasional grey layering		98	1	RC NQ	1525	100	100	100			Bentonite Seal Tunnel Obvert Poor quality zone Tunnel Invert Filter sand
			97									
9.0			96	2	RC NQ	1465	100	23	100			
			95									
10.5			94	3	RC NQ	1525	100	48	100			
			93									
12.0			92	4	RC NQ	1525	100	73	100			
			91									
13.5			90	5	RC NQ	1525	100	33	100			
14.00												
15.0	75 mm thick 'muddy' zone											
15.80												
16.5	Borehole terminated at 15.80 m											

NOTES:

CHECKED BY: *[Signature]*

## LIST OF ABBREVIATIONS

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N', - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 0.3m INTO THE SUBSOIL. DRIVEN BY MEANS OF A 63.5kg HAMMER FALLING FREELY A DISTANCE OF 0.76m.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS. 0.3m INTO THE SUBSOIL. THE DRIVING ENERGY BEING 475 J PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3 m</u>	<u>c kPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3 m</u>
VERY SOFT	0 - 2	0 - 12	VERY LOOSE	0 - 4
SOFT	2 - 4	12 - 25	LOOSE	4 - 10
FIRM	4 - 8	25 - 50	COMPACT	10 - 30
STIFF	8 - 15	50 - 100	DENSE	30 - 50
VERY STIFF	15 - 30	100 - 200	VERY DENSE	> 60
HARD	> 30	> 200		
W.T.P.L. WETTER THAN PLASTIC LIMIT		D.T.P.L. DRIER THAN PLASTIC LIMIT		
A.P.L. ABOUT PLASTIC LIMIT				

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W	THINWALL OPEN
W.S	WASHED SAMPLE	T.P	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S	OESTERBERG SAMPLE
A.S	AUGER SAMPLE	F.S	FOIL SAMPLE
C.S	CHUNK SAMPLE	R.C	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

### SOIL TESTS

Q <sub>u</sub>	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Q <sub>cu</sub>	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q <sub>d</sub>	DRAINED TRIAXIAL		

Δ, Δ - Undisturbed and remoulded undrained shear strength determined from insitu vane test.

□ - Undrained shear strength determined from pocket penetrometer test.

# PetoMacCallum Ltd.

CONSULTING ENGINEERS

## LOG OF BOREHOLE NO. 2A

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE June 8 & 9/92

OUR PROJECT NO. 91HF007A

ENGINEER M.R. Anderson

BORING METHOD Continuous Flight Solid Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE		SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION							
	GROUND ELEVATION 113.34									
	Borehole advanced without sampling to bedrock surface									
7.30	Commenced Rotary Drilling		106							Native Backfill
7.5										Betonite Seal
8.60	SHALE: Poor quality red shale, low strength, with grey layering		105	1	RC	1525	100	28	100	
9.0	Queenston Formation becoming fair quality		104	2	RC	1525	100	53	100	
			103		NQ					
10.40	becoming good quality		102	3	RC	1525	95	90	100	Filter Sand
10.5			101	4	RC	1525	98	75	100	
12.0			100		NQ					
13.5			99	5	RC	1525	100	93	100	
15.0			98		NQ					
			97	6	RC	1525	100	80	100	Tunnel Obvert 97.7
16.50	becoming very poor quality		96	7	RC	1525	93	13	100	Very poor quality zone
18.0	becoming poor to fair quality		95		NQ					
			94	8	RC	1525	97	50	100	Tunnel Invert 94.4
19.50	Borehole terminated at 19.50 m		93		NQ					

NOTES:

CHECKED BY: *[Signature]*

**LOG OF BOREHOLE NO. 2B**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE July 13/92

OUR PROJECT NO. 91HF007A

ENGINEER M.R. Anderson

BORING METHOD Continuous Flight Solid Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE			SAMPLES		RUN(mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER							
	GROUND ELEVATION 115.21		115								
	Borehole advanced without sampling to bedrock surface		114								
1.5			113								
2.15	becoming shale		112								
3.00	Commenced Rotary Drilling		111	1	RC NQ	1525	100	60	100		
	SHALE: Fair quality red shale, low strength, occasional grey layers		110								
4.50	Queenston Formation		109	2	RC NQ	1525	100	47	100		
	becoming poor quality, occasional vertical fractures		108								
6.0			107	3	RC NQ	1525	100	45	100		
7.5			106								
9.0			105	4	RC NQ	1525	100	38	100		
	becoming good quality		104								
10.5			103	5	RC NQ	1525	100	80	100		
12.0			102	6	RC NQ	1525	95	82	100		
12.90			101	7	RC NQ	1220	100	58	100		
	becoming fair quality		100								
14.80			99	8	RC NQ	1370	100	59	100		
15.0	becoming excellent quality			9	RC NQ	1370	100	93	100		
16.5				10	RC NQ	1525	100	96	100		

Native Backfill

Bentonite Seal

Filter Sand

NOTES:

CHECKED BY: *[Signature]*



***PetoMacCallumLtd.***  
CONSULTING ENGINEERS

August 20, 1992

Our Ref: 91HF007A

The Regional Municipality of  
Hamilton-Wentworth  
c/o Mr. H. Johnson, P.Eng.  
R.V. Anderson Associates  
1210 Sheppard Avenue East  
Suite 401  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Supplementary Geotechnical Investigation  
Iona Sanitary Trunk Sewer  
Hamilton, Ontario

We are pleased to present the results of the supplementary geotechnical investigation completed in connection with the above noted project.

The supplementary work was carried out to further evaluate the nature/extent of the zone of very poor quality shale identified in borehole 3 during the original investigation, and to obtain groundwater level measurements in the tunnel/shaft area.

Supplementary Coring

On June 8 and 9, 1992, a supplementary borehole designated borehole 2A was put down at the location of rock probe 2. This borehole was advanced unsampled to the bedrock surface and thin coring of the shale bedrock was carried out to total depth of 19.5 m.

After review of the core information from borehole 2A, authorization was received to drill two additional boreholes. Borehole 2B was put down approximately halfway between boreholes 2 and 2A, and borehole 2C was drilled near rock probe 1. Core samples of the shale were recovered between 3.0 to 22.7 m depth in borehole 2B, and between 6.9 to 15.8 m depth in borehole 2C.

The borehole locations are shown on Drawing 1, Revision 1 dated August 1992.

...2

H. Johnson, August 20, 1992, P2

91HF007A

The ground elevations at boreholes 2A, 2B and 2C were established by Peto MacCallum Ltd. relative to the ground elevation at boreholes 2 and 3 previously defined by The Regional Municipality of Hamilton-Wentworth surveyors. The elevations were not determined relative to a fixed benchmark and are therefore considered to be approximate.

The elevation at borehole 2A (rock probe 2) previously established by The Regional Municipality of Hamilton-Wentworth was inconsistent with the elevations at boreholes 2 and 3. The elevation at borehole 2A has been adjusted accordingly.

The results of the coring operations including rock classifications and rock quality designation (RQD) are provided on the attached borehole logs.

The adjusted ground and bedrock surface elevations at the boreholes are summarized on Drawing 1.

We note that the bedrock depths previously defined by rock probes 1 and 2 may not be reliable. It is possible that these probes were terminated on shale fragments within the clay till overburden. A shale fragment was contacted at 4.6 m depth in borehole 2C; this was believed to be bedrock but coring proved otherwise.

The zone of poor to very poor quality shale encountered in borehole 3 was also encountered in boreholes 2A, 2B and 2C. The elevation of this zone was as follows:

<u>Borehole No.</u>	<u>Zone of Poor Quality Shale</u>	
	<u>Depth (m)</u>	<u>Elevation</u>
2	----	----
2A	16.5 - 18.0	96.8 - 95.3
2B	18.4 - 20.4	96.8 - 94.8
2C	8.7 - 10.6	97.1 - 95.2
3	9.3 - 10.8	96.7 - 95.2

H. Johnson, August 20, 1992, P3

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Groundwater Levels

Piezometers were installed in boreholes 2A, 2B and 2C for measurement of groundwater levels. A nested piezometer installation was constructed adjacent to borehole 1 at the drop shaft location. Details of the piezometer installations were as follows:

<u>Borehole No.</u>	<u>Date of Installation</u>	<u>Screen Depth (m)</u>	<u>Bentonite Seal Position (m)</u>
1a	July 16	97.5 - 100.5	64.0 - 67.0
1b	July 16	58.0 - 61.0	34.6 - 40.7
1c	July 16	27.5 - 30.5	0.0 - 4.5
2A	June 9	18.0 - 19.5	6.7 - 7.3
2B	July 15	19.7 - 22.7	12.3 - 13.6
2C	July 21	14.3 - 15.8	7.3 - 8.5

The water levels measured in the piezometers on August 7, 1992 are summarized below:

<u>Borehole No.</u>	<u>Measured Water Level</u>	
	<u>Depth (m)</u>	<u>Elevation</u>
1a	23.3	167.0
1b	23.2	167.1
1c	26.5	163.8
2A	9.1	104.2
2B	6.2	109.0
2C	4.1	101.7

H. Johnson, August 20, 1992, P4

91HF007A

Should you require any additional information, please do not hesitate to contact this office.

Sincerely

Peto MacCallum Ltd.



Murray R. Anderson, P.Eng.  
Project Manager



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

MRA:rz

4 cc: Region of Hamilton-Wentworth; J. Koshurba  
1 cc: R.V. Anderson Associated; H. Johnson

G.I.-30 SEPT. 1976

GEOCRES No. 30M4-88DIST. 4 REGION \_\_\_\_\_

W.P. No. \_\_\_\_\_

CONT. No. \_\_\_\_\_

W. O. No. 94-11001

STR. SITE No. \_\_\_\_\_

HWY. No. 403LOCATION Iona Sewer ProjectNo of PAGES -OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

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

GEOTECHNICAL INVESTIGATION  
IONA SANITARY TRUNK SEWER EXTENSION  
SCENIC DRIVE TO ROYAL AVENUE  
HAMILTON, ONTARIO  
FOR  
THE REGIONAL MUNICIPALITY OF  
HAMILTON-WENTWORTH

Distribution:

4 cc: Client  
1 cc: R.V. Anderson Associates Limited  
1 cc: PML Hamilton  
1 cc: PML Toronto

Job No. 91HF007

August, 1991

# ***PetoMacCallumLtd.***

*C O N S U L T I N G   E N G I N E E R S*

August 6, 1991

Our Ref: 91HF007

Mr. P. Stumpo  
The Regional Municipality of  
Hamilton-Wentworth  
Department of Engineering  
71 Main Street West  
Hamilton, Ontario  
L8N 3T4

Dear Mr. Stumpo

Geotechnical Investigation  
Iona Sanitary Trunk Sewer Extension  
Scenic Drive to Royal Avenue  
Hamilton, Ontario

We are pleased to present the results of our geotechnical investigation for the proposed sanitary trunk sewer extension project briefly described above. Written authorization to proceed with this investigation was provided to Mr. L. Franco, Director of Administration, the Regional Municipality of Hamilton-Wentworth in a letter dated January 22, 1991.

The project involves the design and construction of a sanitary trunk sewer extending from the top of the Niagara Escarpment at Scenic Drive northerly to Iona Avenue and then east to Royal Avenue, in Hamilton, Ontario. An approximate 94 m deep drop shaft is to be constructed through the Niagara escarpment and connected by a tunnel under Hwy. 403 to an open cut section which begins about 125 m south of Iona Avenue.

Subsurface conditions at the drop shaft location consisted of approximately 2.8 m of silty clay overburden mantling dolostone bedrock. The bedrock stratigraphy at the borehole location was typical of the Niagara escarpment and comprised eleven geological formations consisting of interbedded carbonates, shales and sandstones. Queenston shale was encountered at 53.8 m depth below grade (elevation 136.6) at the test hole.

Subsurface conditions encountered in the boreholes within the tunnel/driving shaft location comprised surficial topsoil overlying clay/clay till mantling Queenston shale bedrock. The overburden thickness increased from south to north from 1.4 m at borehole 2 to 7.6 m at borehole 4.

...2

P. Stumpo, August 6, 1991, P2

91HF007

Variable clay, clay fill and topsoil layers were contacted surficially at the borehole locations within the open cut section. The fill and topsoil were generally underlain by silt and/or sand alluvium which overlies a clay/clay till unit. A discontinuous sand and/or clays and silts deposit was encountered locally below the upper units.

The level of free water could not be measured in tunnel/shaft section due to the use of drilling water during rock coring. However groundwater seepage through highly permeable zones of fractured bedrock must be anticipated. Free water was not identified in test holes 5 and 8, but it was noted at depths of 1.5 m in test holes 9 to 11 and at 4.5 m in test hole 6 during drilling. Upon completion of drilling free water was measured at depths of 1.1 to 2.4 m in boreholes 9 to 11 and at depths of 7.3 and 8.2 m in test holes 6 and 7.

It is envisioned that construction of the drop structure will involve conventional rock excavation/blasting techniques. Excavation of the driving shaft through the native clay/clay till should be straightforward. Excavation through the Queenston shale bedrock to the driving shaft will be more difficult requiring large excavation equipment and possible blasting and jack-hammering. The tunnel section connecting the two shafts should be constructed using a conventional tunnel boring machine suited for soft rock excavation and/or standard mining/blasting techniques. The Queenston shale in the tunnel and shafts is expected to converge about 40 mm following excavation, over a period of about one month.

Construction of the open cut section should be relatively straightforward and slopes may generally be excavated at inclinations of 1 horizontal to 1 vertical, locally flatter where loose/soft/wet soils are present, provided sufficient space is available.

Where space limitations and/or excavation depths do not permit construction of sufficiently shallow slopes, such as at locations adjacent to existing hydro towers, braced excavations will be required.

The standard concrete cradle or granular bedding requirement of The Regional Municipality of Hamilton-Wentworth should be satisfactory.



P. Stumpo, August 6, 1991, P3

91HF007

The moisture contents of the native soils in the proposed open cut section are typically above optimum moisture content. Reuse of on-site soils for bulk fill purposes is considered feasible only if above normal post constructions settlements can be tolerated.

We trust this will be sufficient.

Sincerely

Peto MacCallum Ltd.



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

TJG:lh

5 cc: Client

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LOG OF BOREHOLES SHEETS

DRAWING 1 - Borehole Location Plan

DRAWING 2 - Soil and Rock Profile

DRAWING 3 - Earth Pressure Diagram (Singly Braced Cuts)

DRAWING 4 - Earth Pressure Diagram (Multi-Braced Cuts)

## **1.0 Introduction**

Peto MacCallum Ltd. has been retained by the Regional Municipality of Hamilton-Wentworth to carry out a geotechnical investigation for the proposed extension of the Iona sanitary trunk sewer.

The alignment of the proposed trunk sewer extends westerly along the Ontario Hydro right-of-way from the intersection of Royal Avenue and Bowman Street to the north-south Hydro corridor west of Brodick Street, and then southerly under Hwy. 403 to Scenic Drive at the top of the Niagara Escarpment, in Hamilton, Ontario. The relative location of the sewer alignment is shown on Drawing No. 1.

The section of sewer from Royal Avenue to about 150 m north of Hwy. 403 is to be constructed using conventional open cut trenching techniques. An approximate 100 m deep drop shaft and tunnel is planned to connect the existing sewer at the top of the escarpment with the proposed sewer installed using open cut techniques to within 150 m of Hwy. 403. Installation of the sewer in a tunnel bored through bedrock using conventional rock tunnelling design and equipment is planned.

The proposed sanitary trunk sewer will be 1200 mm in diameter in the open cut section and 4.4 m high by 3.1 m wide in the tunnel section. The diameters of the drop shaft and driving shaft have not been provided. The trunk sewer extends over a total length of about 1650 m.

The purpose of this study was to define the subsurface conditions and engineering properties of in situ soil/rock at the site in order to provide geotechnical engineering comments and recommendations pertinent to the design and construction of the proposed trunk sewer extension.

The scope of work and details of this investigation were defined in the proposal request prepared by the Regional Municipality of Hamilton-Wentworth, dated November 20, 1990. The study was expanded during the field work to enable more complete definition of the subsurface conditions and additional boreholes/rock probes were required in the vicinity of the driving shaft, north of the escarpment base.

Franklin Geotechnical Ltd. was retained to provide specialist advice pertinent to the rock mechanics aspects of the tunnel/shaft design.

## **2.0 Investigation Procedures**

The field work for this study, carried out during the period from January 30 to April 3, 1991, involved 11 test holes and 2 rock probes.

Boreholes 5 to 11, inclusive were advanced to depths of 8.1 to 11.1 m at selected locations along the open cut portion of the sewer that extends between Royal Avenue and the north-south Hydro corridor, west of Brodick Street on January 30 and 31, 1991. The test holes were drilled using continuous flight hollow and solid stem augers, powered by a track-mounted CME-55 drillrig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of our engineering staff.

Following the completion of these test holes, the field work was delayed pending approval from Ontario Hydro to carry out a subsurface investigation within the north-south Hydro corridor.

After this permission was obtained, the remaining test holes were completed during the period from March 18 to April 3, 1991. The test holes were advanced using a track-mounted Acker Soilmax (boreholes 1 and 3) and a CME-55 (boreholes 2, 4 and rock probes 1 and 2) drillrig.

Boreholes 1 to 4 were augered through the overburden to the bedrock surface and then continuously cored using NQ rotary diamond drilling equipment. Test holes 1 to 3 were advanced to between 2.7 and 4.7 m below the proposed invert elevation of the tunnel (15.4 to 99.2 m below existing grade).

Test hole 4 drilled at the location of the proposed driving shaft was terminated in shale bedrock at 10.3 depth, approximately 400 mm below the sanitary sewer invert elevation.

Sound bedrock was encountered 8.8 m below the ground surface (elevation 95.6) at borehole 4. Since the proposed level of the tunnel obvert at this location was about 98.4 m and the design requires that sound bedrock be present at least 2 m above the tunnel obvert, two rock probes, RP 1 and RP 2 were drilled at the request of R.V. Anderson Associates Limited to delineate the bedrock surface south of borehole 4 in order to determine a more suitable location for the shaft. The rock probes were advanced without sampling at locations selected by R.V. Anderson Associates Limited and Peto MacCallum Ltd. to depths between 6.3 and 7.2 m, and were terminated upon auger refusal on inferred bedrock. Borehole 3 was put down near this alternate shaft location.

The relative locations of all test holes and rock probes are indicated on the borehole location plan, Drawing 1, appended.

Representative samples of the overburden were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration, pocket penetrometer and field vane tests were conducted during drilling to assess the strength characteristics of the substrata. Continuous rock cores were obtained from the boreholes advanced below the bedrock surface. The groundwater conditions in the boreholes were closely monitored during the course of the field work.

It should be noted that about 40 m of drill rod could not be retrieved from test hole 1. This should be considered during design and installation of the drop shaft.

It should also be noted that during the advance of borehole 4, auger grinding occurred at about 6.9 m depth, 0.7 m above the bedrock surface. When the augers were pulled to permit sampling at a depth of 7.6 m the sound of flowing water could be heard coming from the base of the borehole indicating an underground service may be present at this depth/location. The drillrig was moved 3 m west and the test hole was completed.

A storm sewer was not shown on the underground service drawings provided to our office by the Region of Hamilton-Wentworth. We suggest a thorough search of available information be completed to determine the nature and extent of this possible underground service.

The programmed locations of the test holes were shown on the site plan and drawings provided by the Regional Municipality of Hamilton-Wentworth. The actual field locations of the boreholes were established by Peto MacCallum Ltd. as dictated by existing underground services and terrain restrictions. Test hole 2 was originally scheduled to be drilled in the grass median of Highway 403. However, MTO was hesitant to permit drilling on the highway median and consequently the borehole was moved to a location at the base of the north slope of the highway embankment to facilitate drilling. Ground surface elevations at the test holes were determined by a survey crew from the Regional Municipality of Hamilton-Wentworth.

### **3.0 Laboratory Testing**

All of the recovered soil samples were returned to our laboratory for detailed visual examination, classification, and routine moisture content determinations. The bulk unit weight of representative samples of the overburden soils was also determined. The test results are plotted on the appropriate Log of Borehole Sheet; the bulk unit weights are summarized on Table I.

Selected rock core samples from each geologic formation penetrated in borehole 1, and at least one core from the tunnel area in boreholes 2 and 3 were submitted to Franklin Geotechnical Ltd. for specialized testing of selected engineered properties including moisture content, dry unit weight, sonic velocity, uniaxial strength and secant/tangent moduli.



Select rock core samples were carefully preserved and waxed immediately following recovery from the borehole to prevent drying. Additional samples were requested by Franklin Geotechnical Ltd.; the subsequent samples had been preserved unwaxed in the core boxes.

The test procedures employed along with the test results are presented in Appendix A and summarized on the test hole logs.

#### **4.0 Site Description and Geology**

The route of the proposed sanitary trunk sewer crosses the Niagara Escarpment which forms a major topographic feature in this area. Consequently, significant variations in the general site description occur over relatively short distances.

The section above the escarpment at Scenic Drive is principally within an Ontario Hydro corridor. The ground surface is slightly undulating and grass covered.

The ground surface drops abruptly from about elevation 190.0 on top of the escarpment to 120.0 on the north edge of the Hwy. 403 embankment over a horizontal distance of about 125 m.

The ground surface extending north from the Hwy. 403 embankment to Iona Avenue and then easterly along the Hydro lands to Royal Avenue and Bowman Street is grass covered and gently slopes north and east with local undulations. The section immediately north of the Hwy. 403 embankment is more steeply inclined.

The escarpment is formed mainly of lower and middle Silurian carbonate sedimentary deposits. The upper portion is formed by a complex succession of dolomites interbedded with limestone and shales belonging to the Lockport Formation and Clinton Groups. Although the boundary varies through the region, the Caratact Group, consisting of interbedded sandstones, shales and limestone normally exists below elevation 165. The upper limit of the Queenston Formation is near elevation 135. The regional dip of these strata varies between one and two degrees below horizontal to the west or southwest.

The bedrock geology of the study area is detailed on Ontario Division of Mines Maps 2336 and 2343 titled "Palaeozoic Geology, Hamilton" and "Palaeozoic Geology, Grimsby", respectively.

According to the Ontario Department of Mines Map 2033, "Pleistocene Geology, Hamilton Area", surficial deposits below the escarpment are described as Halton clay or silt till; and/or offshore fine-grained sand and silty sand deposits of former glacial Lake Iroquois.

## **5.0 Subsurface Conditions**

### **5.1 GENERAL**

Reference is made to the appended Log of Borehole sheets for details of the soil and rock classifications, inferred stratigraphy, standard penetration test "N" values, pocket penetrometer and field vane shear strength test results, groundwater observations and the results of laboratory testing referenced in Sections 3.0.

A schematic overview of the bedrock type and overburden thickness along the tunnel alignment is presented on Drawing 2.

The stratigraphy along the sewer alignment is consistent with the geology of the area inferred from the geological maps.

## 5.2 TOP OF ESCARPMENT

Borehole 1 was advanced at the top of the escarpment.

An approximately 150 mm thick topsoil layer was encountered surficially in test hole 1. The topsoil was comprised of dark brown silty clay and judged to be low organic. Very stiff silty clay was present beneath the topsoil to 2.8 m below the ground surface; bedrock was encountered underlying the silty clay at this depth.

The bedrock lithology at the borehole location was typical of the Niagara Escarpment in the West Hamilton area and comprised interbedded Palaeozoic carbonate, shale and sandstone sedimentary strata. Eleven geological formations were identified in the rock core recovered from test hole 1, as detailed on the borehole log and summarized in the following table. The geologic formations are numbered sequentially from the bottom of the hole upward for ease of reference.

SUMMARY OF BEDROCK LITHOLOGY AND  
STRATA THICKNESS

11. LOCKPORT FORMATION

- a) Goat Island Member (10.3 m). Dolostone, aphanitic, with chert nodules, weathered on partings, close to moderately spaced discontinuities (50 to 90 mm), fair quality.
- b) Gasport Member (1.4 m). Dolostone, medium crystalline, close to wide spaced discontinuities, (50 to 3,000 mm), excellent quality.

10. DECEW FORMATION (2.5 m). Dolostone, good quality.

9. ROCHESTER FORMATION (1.0 m). Interbedded shale and limestone, excellent quality.

8. IRONDEQUOIT FORMATION  
(1.5 m). Limestone, medium crystalline, excellent quality.

7. REYNALES FORMATION  
(2.6 m). Dolostone, with thin shale layers, good to excellent quality.

6. THOROLD FORMATION  
(2.6 m). Sandstone with layers of shale, good quality.

5. GRIMSBY FORMATION  
(6.1 m). Interbedded sandstone and shale, massive to thinly bedded, good quality.

4. CABOT HEAD FORMATION  
(18.5 m). Shale, low to medium strength, poor to fair quality.

3. MANITOULIN FORMATION  
(1.2 m). Dolostone interbedded with shale, good quality.

2. WHIRLPOOL FORMATION  
(3.4 m). Sandstone, massive to finely bedded, fair quality.

1. QUEENSTON FORMATION  
(greater than 45.4 m). Red shale with green mottling, calcite infilling, typically fair to good quality, excellent between 80.9 and 93.0 m depth.

Some thin isolated clay seams and chert/calcite lined vugs were noted in the upper carbonate deposits and are likely a result of infilling of voids and/or fractures.

Numerous zones of relatively weak shale which alter rapidly to a clay state on exposure, were encountered within both the Cabot Head and Queenston formations.

The Rochester formation was highly fractured.

In general, the in situ moisture contents of the rock core samples from borehole 1 ranged from 1.3 to 5.5 percent. Moisture contents of the rock core stored in the core boxes ranged from 0.2 to 0.7. Dry unit weights for these specimens were between 2.0 and 3.0 g/cm<sup>3</sup> and uniaxial strength varied from 7.0 MPa for the Cabot Head shale to 114.0 MPa for the DeCew dolostone.

Refer to the test hole logs and Appendix A for more detailed description of the rock properties.

### 5.3 BASE OF ESCARPMENT - NORTH OF HIGHWAY 403

Test holes 2 to 4, and rock probes 1 and 2 were advanced between of the north limit of the Highway 403 embankment and the proposed driving shaft location.

In general, the subsurface conditions encountered at the test hole locations comprised surficial topsoil overlying clay/clay till mantling Queenston shale bedrock. The overburden thickness increased from south to north from 1.4 m at borehole 2 to 7.6 m at borehole 4.

The thickness of the surficial topsoil was between 150 to 200 mm. This material comprised dark brown silty clay, judged to have a low organic content.

The topsoil was underlain by firm to very stiff, silty clay/clay till with trace to some sand. The clay/clay till was judged to be low to medium plastic, at or wetter than the plastic limit.

The bedrock surface dips to the north and weak shale bedrock was contacted below the clay/clay till at depths between 1.4 to 7.6 m below the existing ground surface. Auger refusal on sound shale was encountered at 1.7 to 8.8 m depth (El. 118.3 to 95.6) in the test holes. The following table summarizes the ground surface and bedrock surface elevations as well as auger refusal elevations at the test holes and rock probes.

<u>Test Hole/ Rock Probe No.</u>	<u>Ground Surface Elevation</u>	<u>Bedrock Surface Elevation</u>	<u>Elevation of Auger Refusal/ Sound Bedrock</u>
BH 2	120.00	118.6	115.3 *
RP 2	109.63	104.3	102.4
BH 3	106.02	102.7	102.7 *
RP 1	106.29	100.8	100.0
BH 4	104.39	96.7	95.6

\* Elevations of sound shale inferred from core recovery/RQD

Rotary core drilling was carried out to advance the three boreholes below the zone of auger refusal to depths of 28.0, 15.4 m and 10.3 m in boreholes 2, 3 and 4, respectively. The rock quality was generally fair to good improving to good/excellent with depth. A localized zone of very poor quality Queenston shale was encountered

in borehole 3 within the proposed tunnel section between 9.3 and 10.8 m below grade (elevation 96.7 to 95.2).

The in situ moisture contents of Queenston shale core specimens from boreholes 2 and 3 ranged from 1.7 to 4.7 percent; the moisture content of core samples stored in the core box ranged from 0.47 to 2.55. The dry unit weight varied between 2.21 and 2.56 g/cm<sup>3</sup> and uniaxial compressive strength test results ranged from 8.6 to 29.2 MPa.

#### 5.4 OPEN CUT SECTION, WEST OF BRODICK STREET TO ROYAL AVENUE

Boreholes 5 to 11 were located along the open cut section of the sanitary truck sewer, on Iona Avenue and the Hydro right-of-way from Brodick Street to Royal Avenue. These test holes were advanced to the depths scheduled by the Regional Municipality of Hamilton-Wentworth, that ranged between 8.0 and 11.1 m below existing grade.

Variable clay, clay/fill and topsoil layers were contacted surficially at the borehole locations. The fill and topsoil were generally underlain by silt and/or sand alluvium which overlies a clay/clay till unit. A discontinuous layered sands and/or clays and silts deposit was encountered locally below the upper units.

Surficial clay fill was present at test holes 6, 7 and 9 to 11. This fill comprised a firm to stiff, silty clay with trace to some sand, and was judged to be low to medium plastic. In situ moisture contents varied from 21 to 30 percent, and the thickness of the fill ranged from 1.0 to 2.1 m.

Topsoil was contacted surficially in borehole 8. This unit was about 100 mm thick and comprised dark brown clayey silt, judged to have a low organic content. An approximate 300 mm thick topsoil layer was encountered locally below the surficial clay fill in borehole 6. An obvious topsoil layer was not identified in test hole 5.

Stiff silty clay with some sand was present in test hole 5 from the ground surface to 1.4 m depth.

The surficial materials were underlain by variable layered sands/silts/clays, sand and/or silt alluvium and native clay.

Silt and/or sand alluvium was present below the topsoil in borehole 6 and beneath the clay fill in boreholes 9 and 11. This material comprised very loose, variable sand/silt and thicknesses ranged from 300 to 800 mm. The in situ moisture content of one sample was 18 percent.

Layered sands/silts/clays were encountered below the alluvium in test holes 6 and 9, and below the surficial clay in borehole 5. These units comprised interlayered loose brown silty fine sand, loose to compact brown to grey silt and soft to stiff brown to grey silty clays. Thicknesses ranged from 1.0 to 2.8 m and moisture contents were typically between 22 and 26%.

A clay deposit was identified below the upper soil units except test hole 9. This unit was present below the layered sands/silts/clays in boreholes 5 and 6, clay fill in boreholes 7 and 10, surficial topsoil in borehole 8, and sand alluvium in borehole 11. The clay was generally stiff to very stiff, becoming stiff to firm with depth; locally very stiff to hard becoming very stiff to stiff in borehole 10. The undrained shear strength of



the upper 1.5 m of this deposit measured using a pocket penetrometer, typically ranged from 100 to 200 kPa. In situ vane shear strength test results conducted in the lower portion of this unit ranged from 50 to 75 kPa.

The in situ moisture content for this material was typically 25%, but ranged from 18 to 35% and generally increased with depth. The thickness of the clay layer varied from 3.9 to 6.1 m in test holes 5, 7 and 8, where the unit was fully penetrated. Boreholes 6, 10 and 11 were terminated in the clay at depths between 8.1 and 11.1 m.

Clay till was contacted underlying the clay unit in test holes 5, 7 and 8 and below the layered silts and clays in test hole 9. The till comprised silty clay with trace of sand and was judged to be medium plastic. The consistency of the clay till was generally firm to stiff, locally hard becoming very stiff to firm with depth in borehole 9. Undrained shear strengths of this material determined by field vane tests varied from 5.0 to 100 kPa. In situ moisture contents for this unit were typically 22 to 26% with locally lower values in test hole 5 and higher in test hole 8.

Boreholes 5 and 7 to 9 were terminated in the clay till at depths between 9.0 to 11.1 m.

None of the test hole locations programmed by the Regional Municipality of Hamilton-Wentworth were within the existing roadway sections crossed by the proposed sewer route. Therefore the depth of asphalt, concrete and granular material at these locations could not be measured during this study. Further field work, including coring of the roadway, is required to provide this information.

Free water was not identified in test holes 5 and 8 during drilling. It was noted at depths of 1.5 m in test holes 9 to 11 and at 4.5 m in test hole 6 during drilling. Upon completion of drilling free water was measured at depths of 1.1 to 2.4 m in test holes 9 to 11 and at depths of 7.3 and 8.2 m in test holes 6 and 7. The water level in the standpipe installed in borehole 10 was measured at 1.8 m depth on February 21, 1991. It is considered that the water is perched in the upper layered clays and silts/clay fill/sand alluvium above the clay/clay till over the eastern portion of the proposed alignment (boreholes 9 to 11).

Stabilized groundwater levels have not been determined and varying levels may be encountered during construction.

## **6.0 Engineering Considerations**

### **6.1 GENERAL**

Reference is made to the Regional Municipality of Hamilton-Wentworth Department of Engineering drawings for the Iona Sanitary Trunk Sewer, sheets 1 to 6, inclusive for details of the proposed sanitary sewer, drop structure and tunnel. As currently proposed, the sanitary trunk sewer will drop some 94 m through a drop shaft to invert elevation 96.3 m and then proceed northerly in tunnel section for approximately 370 m beneath the face of the escarpment, Highway 403, and the north-south Hydro corridor. The tunnel dimensions are to be 4.4 m high and 3.1 m wide and the tunnel invert is about 500 mm below the sewer invert. It will terminate at an approximate 10.5 m deep driving shaft to be constructed about 125 m south of Iona Avenue near the location of test hole 4 (MH 10). The diameters of the two shafts were not been provided.

From the north portal of the tunnel, a 1200 mm diameter sanitary sewer will follow northerly to Iona Avenue and then along the east-west Hydro right-of-way north of Iona Avenue terminating at Royal Avenue/Bowman Street. The planned length of this section is about 1270 m and invert elevations range from 94.4 m at the junction with the tunnel section to 88.6 m at the east end of the sewer.

## 6.2 SHAFTS AND TUNNEL

Based on the stratigraphy and bedrock lithology encountered at borehole 1, the subsurface conditions to the invert of the 94 m deep drop structure will comprise 2.8 m of silty clay over the various lithological units of the Niagara Escarpment between dolomite of the Goat Island member of the Lockport Formation to shale of the Queenston Formation.

The current design calls for the invert of the tunnel to drop from elevation 95.8 at the bottom of the drop shaft to 94.0 at the driving shaft. The sewer invert elevation drops from 96.3 to 94.4 along this section. Boreholes 2, 3 and probe holes 1 and 2 indicate the tunnelling operation will be carried out entirely within Queenston Shale provided it is terminated south of borehole 4.

The bedrock surface was contacted at elevation 96.7 in test hole 4. The design obvert elevation of the tunnel at this borehole location is 98.4, therefore the tunnel would be partially constructed in soil if advanced to the design location about 125 m south of Iona Avenue. We were advised by R.V. Anderson Associates Limited that the entire tunnel should be excavated in rock, with at least 2 m of competent bedrock above the tunnel obvert to

satisfy the present design requirements. The bedrock surface at test hole 4 was considered to be too deep to accommodate the intended tunnel and driving shaft construction.

The bedrock surface and sound bedrock elevations at borehole 3 are 102.7 and 101.3, respectively. The tunnel obvert is shown to be 98.7 on the design drawings, thus the requirement that at least 2 m of competent bedrock is present above the tunnel obvert appears to be satisfied at this location.

From our conversations with R.V. Anderson, we understand the driving shaft will be located near borehole 3. At this location approximately 3.3 m of clay/clay till overburden is anticipated to be encountered above Queenston Shale bedrock.

#### 6.2.1 EXCAVATION AND GROUNDWATER CONTROL

Excavation through the native clay/clay till overburden at the driving shaft location should be straightforward. Sidewalls in the soil may be sloped at an angle of 1 horizontal to 1 vertical if space permits. If steeper side slopes are required due to the space restrictions, then adequately designed and constructed temporary shoring will be required.

Excavation through the Queenston Shale bedrock to the invert elevation of the driving shaft will be more difficult requiring large excavation equipment and possibly implementation of standard rock excavation methods such as blasting and jack-hammering. The actual equipment required and method of excavation within the bedrock will be somewhat dependent upon the geometry of the cut and relative depth of excavation into the bedrock. The rock excavation should be carried out such that fracturing of the bedrock surface on which the proposed service is to be founded is minimized.

It is anticipated that construction of the 94 m deep drop shaft will also require blasting to advance the excavation, particularly through the competent shale, dolostone, limestone and sandstone formations.

Excavation/blasting of the shaft upwards from the base may be the most expeditious method of constructing the drop shaft but other techniques such as downward excavation with blasting or with rock coring equipment are possible. The preferred construction techniques will primarily be dictated by construction constraints, worker safety and economic considerations. Input from a specialist excavation contractor is recommended during the selection process.

Blasting must be carefully controlled to minimize overbreak. A preconstruction survey of the residential structures near the two shaft locations is recommended if blasting is carried out. Blast monitoring is recommended during construction to determine whether charges should be reduced or maintained based on accepted damage criteria or if a different excavation method should be utilized. The tunnel section between the two shafts may be excavated using a conventional tunnel boring machine suited to soft rock excavation and/or conventional mining/blasting techniques.

Alternate techniques for advancing both the shafts and horizontal tunnel may prove acceptable but should be reviewed by our office prior to construction.

It is recommended that monitoring of rock conditions be carried out during construction. The rock conditions in the shaft should be logged by an experienced and qualified engineering geologist.

The Queenston shale has a relatively low durability and should be protected from drying out by the application of shotcrete (fibrecrete) or similar material immediately following excavation.

Normally, the shale should stand unsupported. However discontinuous limestone/sandstone layers and/or major bedding planes may be encountered which could result in local instability. Since the pattern of such bedding can change rapidly along the line of the tunnel, any temporary support system would best be selected and modified by experienced construction personnel as the heading advances.

The shotcrete liner should adequately serve as the primary tunnel liner and permit controlled displacements and arching to develop. Additional support may be provided by installing rock bolts in conjunction with wire mesh and shotcrete or similar means where considered necessary, such as in major zones of weakness, jointing, along seams, hard bands, etc., particularly at the crown of the tunnel.

As mentioned, it is recommended that the primary shotcrete liner be applied daily to keep pace with the excavation.

The shale is expected to converge as a result of the concentration of stresses around the tunnel. Detailed comments and design recommendations in this regard are presented in the following section.

Monitoring of the convergence of the sides of the shaft and tunnel should be carried out at a number of points for as long a period of time as practical. If the in situ measurements during construction indicate greater movements than anticipated, the design of the structure should be reconsidered, cognizant of the actual movements.

The final liner should not be installed until the tunnel is completed and convergence appears to have stabilized based on the monitoring data.

It would be prudent to place a layer of compressible material between the rock surface and the permanent liner. This material may consist of non-compacted dry clay, styrofoam or sprayed foam. Alternatively, an open cavity can be utilized.

The presence of water bearing zones in the rock was not detected in the boreholes drilled along the shafts/tunnel section, primarily because of the presence of drilling fluid used during coring of the rock.

Essentially all of the water employed during coring was recycled, a significant loss of drilling water was not observed. In addition, sample recovery and the relatively high RQD values suggest that the bedrock in the shafts/tunnel section is not generally highly fractured. Therefore control of groundwater in the drop shaft and tunnel is not expected to be a major concern.

It should be noted however that seepage of water onto the face of the escarpment is often evident during wet spring conditions. Hence, some seepage is to be expected.

The potential for local relatively high concentrations of groundwater inflow into the shaft/tunnel should not be overlooked. During construction local permeable zones may be encountered in discontinuities/fracture zones such as the zone of very poor rock quality encountered in borehole 3 between 9.3 and 10.8 m below grade. Stress relief during excavation may also cause some seams to open up and permit seepage along bedding plains and joints. It

is possible that the 'flowing water' observed in test hole 4 is a void in the rock.

Low volume seepage should be adequately handled using conventional sump pumping techniques. High volume seepage may require pressure grouting within the more permeable zones as described in section 6.4.3.

#### 6.2.2 DESIGN CONSIDERATIONS

The following design considerations are provided as per the requirements presented in the Regional Municipality of Hamilton-Wentworth letter dated November 20, 1990. These responses are based on the research and test results completed by Franklin Geotechnical Ltd. The detailed findings of this work are provided in Appendix A and B.

- . The Modulus of Deformation for the rock is assumed to be 10 GPa.
- . Poisson's Ratio for the rock is assumed to be 0.3.
- . Convergence in the shotcreted tunnel, the driving shaft and the lower portion of the drop shaft which is through Queenston shale is estimated to be about 40 mm, occurring over a period of approximately 1 month, with reduced rates following that time. Much less convergence is expected for hard dolostone, limestone and sandstone beds in the portion of the drop shaft above the Queenston shale (above elevation 136.6). Convergence of the Cabot Head shale is expected to be about 30 mm, occurring over about 1 month. The rates of tunnel liner displacements should be similar to these shown on Figure 2 of Report



G678.2, presented in Appendix B. In addition to long-term convergence, local movements of about 25 mm should be anticipated for the tunnel and shaft walls due to the presence of unfavourably oriented joints or discontinuities.

- . A pre-excavation horizontal stress of between 2 MPa and 7 MPa is expected, accompanying a vertical stress of 1 MPa. The stresses should be magnified by a factor of about 2 times in the vicinity of the tunnel. Hence, the levels of stress may locally approach or exceed the uniaxial compressive strength of the shale which is estimated at between 10 to 16 MPa. This overstressing should present little or no problem provided the shale is shotcreted. A plastic yield zone may develop, although the thickness of this zone should be limited to about 1 m if the shale is confined by appropriate primary support.
- . An allowable rock bearing pressure of 5000 kPa may be used for conventional footing foundations founded on the Queenston shale at the proposed shaft invert elevations between 95.8 and 94.0 m.

### 6.3 OPEN CUT SECTION

Based on the subsurface conditions encountered in boreholes 4 to 11, inclusive, excavation to the proposed invert elevations along the open cut section of the sanitary trunk sewer should encounter variable fill/topsoil/silt/sand over clay/clay till, and locally shale near test hole 4. Invert levels typically range from 10 m at the tunnel portal at the west end of the project to 5 m at the east end.

6.3.1 EXCAVATION AND GROUNDWATER CONTROL

Excavation through native soils to the assumed invert depths should be relatively straight forward using conventional equipment. Where sufficient space is available open cut procedures may be used to install the underground services. Based on a simplified slope stability analysis, sidewalls in the very stiff, locally hard, to firm clay/clay till may be excavated at a slope of 1 horizontal to 1 vertical. A detailed stability analysis is recommended for slopes in the stiff to firm clay/clay till with slope heights greater than 5 m when design/construction details are finalized. Flatter side slopes may be necessary where the very loose to loose sand/silt alluvium and layered sand/silt/soft clay, are present, or if localized seepage zones are encountered.

We expect that braced excavations will be required adjacent to existing hydro towers and at all other locations where space limitations and/or excavation depths do not permit construction of sufficiently shallow slopes. Methods for calculating earth pressures and general design recommendations for singly and multi-braced excavations are provided on Drawing No 1 and 2. Rigid supporting walls should be constructed adjacent to settlement sensitive structures. Preliminary calculations indicate that base heave in deep braced excavations along the proposed sewer alignment should not be a problem, however, these findings should be confirmed when designs are completed for the open cut section.

All work should be carried out in accordance with The Occupational Health & Safety Act, 1981 and with local regulations.

#### 6.3.2 PIPE BEDDING

The founding soil at the proposed invert elevations is anticipated to be very stiff to firm clay/clay till/shale. In general, no problems are expected with respect to bearing capacity or settlement.

An allowable soil bearing pressure of 100 kPa is available for manholes/drop structures founded on the firm to stiff silty clay/clay till.

The standard concrete cradle or granular bedding requirement of The Regional Municipality of Hamilton-Wentworth should be satisfactory. It may be necessary to increase the bedding thickness if construction softened clay/silt is present at the pipe subgrade. The need for this is best determined during construction.

#### 6.3.3 TRENCH BACKFILL

The granular bedding material should be carried up as backfill for at least 300 mm above the pipe obvert and compacted to at least 95% Standard Proctor maximum dry density.

If significant post construction settlement of the ground surface at the top of the trench cannot be tolerated, it will be necessary to ensure that the remainder of the trench backfill material comprises approved material placed in uniform 200 mm thick lifts within 2 to 4% of the optimum moisture content and compacted to at least 95% Standard Proctor maximum dry density.

The natural moisture content of the clay/clay till soils indigenous to this site typically ranges from 20 to 25%, locally 35%. We expect that a water content of about 20% is the upper limit at which this type of soil can be placed and compacted expeditiously. Above optimum moisture contents were also measured within the sands and silts. Therefore, we consider that the excavated soils will be suitable for reuse in the trench only if post construction ground surface settlements in excess of the normal 100 mm are tolerable and/or the work is carried out during the dry summer months. The construction schedule should be suited to provide for air drying to reduce the moisture content closer to the optimum value for efficient compaction. Additionally, these materials must remain free of organics and other deleterious materials.

If insufficient quantities of backfill are available on site or if the construction schedule does not provide adequate time for air drying wet materials, it will be necessary to use select imported material as trench backfill. The imported fill ideally should comprise silty clay/clayey silt fills in order to match the excavated soils as close as possible, and thus reduce the impact of construction on the local groundwater table by minimizing changes to the subsurface stratigraphy.

The pavement granulars may be used for reconstruction of the roadways provided they meet the OPSS Granular "A" gradation requirements and are appropriately stockpiled.

All asphalt, topsoil and other deleterious material should be selectively excavated and disposed of off-site.

Where the sewer trench crosses the existing roadways, attention must be given to the construction operations, particularly when native materials are used as trench backfill in order to reduce and render more uniform post construction settlements and to minimize any detrimental effects to the roadway pavement.

Should construction extend to the winter season, particular attention should be given to ensure that frozen material is not used as trench backfill.

Frequent inspection should be carried out by Peto MacCallum Ltd. geotechnical personnel to examine and approve backfill materials and to carefully inspect placement and verify the compaction by in situ density testing using nuclear gauges.

#### 6.3.4 pH AND SULPHATE CONCENTRATIONS

The results of sulphate content testing on two (2) soil samples one (1) from borehole 6 and one (1) from borehole 8, indicate concentrations of 0.029 and 0.046%, respectively. pH levels were measured to be 8.4 for both samples. These results indicate a negligible degree of attack on concrete. For recommendations regarding protective measures, reference is made to CSA Standard A.23.

#### 6.4 GROUNDWATER

##### 6.4.1 GROUNDWATER LEVELS

As previously mentioned, site specific water levels are not available for boreholes 1 to 4 due to the need to use drilling water to core the rock. Evidence of groundwater seepage on the face of the escarpment above Hwy. 403 is often noted during the

winter/spring season. In addition significant flow of groundwater through discontinuities in the rock is possible.

Our review of MOE water well records compiled from 1949 to 1979 for the area at the top of the escarpment, near borehole 1, concluded that most wells in the area are supplied by an artesian aquifer located within the upper dolomitic/limestone bedrock at the site. The water table was contacted at depths between 7.0 to 21.0 m and static levels were about 1.0 to 11.5 m below the ground surface at the well locations.

Within the open cut section observed groundwater levels ranged from 1.7 to 8.2 m depths with a distinct change evident near Leland Avenue. The water table was contacted about 3.0 to 3.5 m above the proposed invert elevation at test holes 9, 10 and 11 and 0.5 to 2.5 m below the invert level at test holes 6 and 7. Free water was not encountered in test holes 5 and 8.

#### 6.4.2 ANTICIPATED IMPACT OF CONSTRUCTION ON THE GROUNDWATER TABLE

Construction of the drop shaft could cause downward flow of water from the upper aquifer in the permeable dolomitic/limestone bedrock to lower zones. Left unchecked during the construction period or if allowed to develop following construction, this could effectively lower the local groundwater table.

Tunnelling operations through the relatively impermeable Queenston shale should have no significant impact on the local groundwater table. Anticipated minor seepage should be adequately controlled by conventional sump pumping techniques.

Construction of the portion of the driving shaft through overburden soils and the open cut portion of the sanitary trunk sewer may require temporary construction dewatering by conventional sump pumping, particularly along the east section of the underground service where invert elevations are approximately 3.0 to 3.5 m below the local groundwater level. The actual impact of construction dewatering on groundwater levels will depend on the quantity of water pumped and the hydrogeologic characteristics of the soil. Cognizant of the anticipated short construction period, it is anticipated that only a minor lowering of the groundwater table will occur.

The installation of the sanitary trunk sewer should have little impact on the stabilized (long term) groundwater levels provided native soils are reused as backfill. It is expected that impact of the installation of the trunk sewer on groundwater levels in the area will be minimized due to the presence of the existing combined box sewer which parallels the proposed sanitary sewer at an invert elevation about 1.5 m higher than the current trunk service design.

#### 6.4.3 MITIGATION OF IMPACT

The use of a grout curtain or steel liner may be required to control groundwater ingress during construction of the drop shaft. An inventory of operating wells should be completed and special care should be exercised to ensure adjacent wells in the vicinity of the shaft are not adversely affected by pumping operations.

To prevent continuous leakage from the upper zone, it is recommended that the void between the drop shaft liner and the bedrock be fully grouted from the bedrock surface (elevation 187.4) to the upper boundary of the first major shale unit (elevation 173.4).

The trench backfill for the sanitary sewer should comprise native clay/clay till or similar imported material in order to minimize the change to local subsurface conditions and thus mitigate the impact of the sewer construction on the local groundwater table. Flow along the base of the sewer trench can be reduced through the use of a concrete cradle in place of granular bedding material.

If these measures are implemented, installation of the sewer is expected to have minimal impact on the groundwater table.

We trust the information presented in this report is sufficient for your present purposes. If you have any questions, please do not hesitate to contact our office.

Yours very truly,

Peto MacCallum Ltd.



Timothy J. Garde, P.Eng.  
Senior Geotechnical Engineer

A handwritten signature of Dennis W. Kerr in black ink.

Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton



TJG:lh



**TABLE I**

**Iona Sanitary Trunk Sewer Extension**

**The Regional Municipality of  
Hamilton-Wentworth**

**Bulk Unit Weights of Soil Samples**

<u>Borehole No.</u>	<u>Depth (m)</u>	<u>Unit Weight (Mg/m<sup>3</sup>)</u>	<u>Soil</u>
5	4.6-5.1	2.08	Clay
5	7.6-8.1	2.13	Clay Till
6	6.1-6.6	2.23	Clay
6	7.6-8.1	2.06	Clay
7	4.6-5.1	2.09	Clay
7	7.6-8.1	2.11	Clay Till
8	6.1-6.6	2.20	Clay Till
9	6.1-6.6	2.09	Clay Till
10	4.6-5.1	2.17	Clay
10	6.1-6.6	2.21	Clay
11	4.6-5.1	2.08	Clay

## LIST OF ABBREVIATIONS

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N', - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 0.3m INTO THE SUBSOIL. DRIVEN BY MEANS OF A 63.5kg HAMMER FALLING FREELY A DISTANCE OF 0.76m.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS. 0.3m INTO THE SUBSOIL. THE DRIVING ENERGY BEING 475 J PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3 m</u>	<u>ckPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3 m</u>
VERY SOFT	0 - 2	0 - 12	VERY LOOSE	0 - 4
SOFT	2 - 4	12 - 25	LOOSE	4 - 10
FIRM	4 - 8	25 - 50	COMPACT	10 - 30
STIFF	8 - 15	50 - 100	DENSE	30 - 50
VERY STIFF	15 - 30	100 - 200	VERY DENSE	> 50
HARD	> 30	> 200		
W.T.P.L.	WETTER THAN PLASTIC LIMIT		D.T.P.L.	DRIER THAN PLASTIC LIMIT
	A.P.L.		ABOUT PLASTIC LIMIT	

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W	THINWALL OPEN
W.S	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H.	SAMPLE ADVANCED HYDRAULICALLY	
	P.M.	SAMPLE ADVANCED MANUALLY	

### SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL		

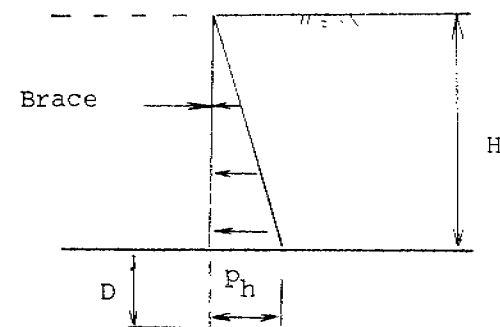
△,△ - Undisturbed and remoulded undrained shear strength determined from insitu vane test

■ - Undrained shear strength determined from pocket penetrometer test

# NOTES:

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system is, in addition to the soil type and temporary/permanent surcharge loads, dependent upon the permissible lateral/vertical movements adjacent to the excavation, the groundwater conditions, drainage provisions, the type of bracing system adopted, weather conditions, quality of workmanship and length of time excavation will be supported. Hence, recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system is established.
2. Earth pressure diagram applicable to multi-braced cuts in stiff clays. Maximum depth of excavation 12 m (40 ft.).
3. Design lateral pressure may be reduced if some surface movement acceptable and design life of bracing system less than 1 month.
4. Stability of base of excavation must be confirmed when bracing system, excavation geometry and surcharge loads established.
5. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system not included in earth pressure diagram.
6. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
7. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
8. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
9. Earth pressure diagram applicable for time frame of relatively short construction periods. If excavation to be open for long periods, monitoring of deformation is essential; earth pressure diagram to be reviewed; remedial works may be required.
10. If settlement sensitive structures located near excavation, special measures to be undertaken to control settlements. Condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
11. Structural components of bracing system to be confirmed adequate for each level of excavation.
12. Bracing system to be regularly examined for signs of distress.
13. All work to be carried out in accordance with conventional construction practice, good quality workmanship and satisfy requirements of local building codes.
14. This sheet to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

## EARTH PRESSURE DIAGRAM



## DESIGN PARAMETERS

$p_h$  = lateral earth pressure  
=  $K\gamma H$

$K$  = lateral earth pressure coefficient

$\gamma$  = unit weight of soil

$H$  = depth of excavation

$D$  = depth of embedment of soldier piles  
(if used)

## RECOMMENDED DESIGN PARAMETERS

$\gamma$  = 20.5 kN/m<sup>3</sup>

$k$  = 0.6 for rigid wall  
= 0.4 for flexible wall

LATERAL EARTH PRESSURE DISTRIBUTION  
Singly Braced Cuts in  
Stiff Clays



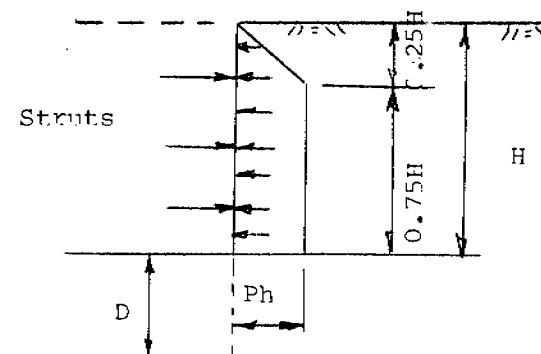
PETO MACCALLUM LTD.  
CONSULTING ENGINEERS

DATE	SCALE	JOB NO.	DRAWING NO.
Aug. '91	NTS	91HF007	3

## NOTES:

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system is, in addition to the soil type and temporary/permanent surcharge loads, dependent upon the permissible lateral/vertical movements adjacent to the excavation, the groundwater conditions, drainage provisions, the type of bracing system adopted, weather conditions, quality of workmanship and length of time excavation will be supported. Hence, recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system is established.
2. Earth pressure diagram applicable to multi-braced cuts in soft to firm saturated normally consolidated clays which extend well below base of excavation. Maximum depth of cut 12 m (40 ft.).
3. Stability of base of excavation must be confirmed when bracing system, excavation geometry and surcharge loads established.
4. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system not included in earth pressure diagram.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. Earth pressure diagram applicable for time frame of relatively short construction periods. If excavation to be open for long periods, monitoring of deformation is essential; earth pressure diagram to be reviewed; remedial works may be required.
9. If settlement sensitive structures located near excavation, special measures to be undertaken to control settlements. Condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
10. Structural components of bracing system to be confirmed adequate for each level of excavation.
11. Bracing system to be regularly examined for signs of distress.
12. All work to be carried out in accordance with conventional construction practice, good quality workmanship and satisfy requirements of local building codes.
13. This sheet to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

## EARTH PRESSURE DIAGRAM



## DESIGN PARAMETERS

$p_h$  = design lateral earth pressure  
 $p_h = \gamma H - 1.6 c_s \geq 0.4 \gamma H$   
 $c_s$  = average undrained shear strength of clay long face of excavation  
 $\gamma$  = unit weight of soil  
 $H$  = depth of excavation  
 $D$  = depth of embedment of soldier piles (if used).

## RECOMMENDED DESIGN PARAMETERS

$\gamma = 20.5 \text{ kN/m}^3$   
 $c_s = 75 \text{ kPa}$

LATERAL EARTH PRESSURE DISTRIBUTION  
 Multi-Braced Cuts in soft to firm normally consolidated saturated clays extending below base of excavation



**PETO MACCALLUM LTD.**  
 CONSULTING ENGINEERS

DATE	SCALE	JOB NO.	DRAWING NO.
Aug. '91	N.T.S.	91HF007	4

**APPENDIX A**

**Rock Core Test Results**

**Provided by**

**Franklin Geotechnical Ltd.**



**IONA SANITARY TRUNK SEWER  
HAMILTON, ONTARIO**

**Results of Testing on Rock Cores**

Prepared for:

**PETO MACCALLUM LTD.,**  
45 Burford Road  
Hamilton, Ontario  
L8E 3C6

Prepared by:

**FRANKLIN GEOTECHNICAL LTD.**  
The Stream  
R.R. #1, Orangeville, Ontario  
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**franklin geotechnical ltd.**

the stream, r.r. #1, orangeville, ontario, canada L9W 2Y8

tel: (519)941-3392

fax: (519)941-9773

Peto MacCallum Ltd.,  
45 Burford Road  
Hamilton, Ontario  
L8E 3C6

27th May, 1991

Attention: Mr. Ty Garde, P.Eng  
Project Engineer

Dear Sirs,

**IONA SANITARY TRUNK SEWER - ROCK TESTING**

**REPORT G678.1**

We have pleasure in submitting the results of rock testing as authorized in your letter of February 21st.

The numerical modelling is still in progress. After some initial difficulties in calibrating the constitutive law for the shale, the results are now giving a realistic simulation of long term tunnel behaviour. A report will follow later this week.

I suggest that when convenient a meeting be arranged between R.V. Anderson Associates, you and us, probably at the University of Waterloo, to discuss the interpretation of results and to decide on the scope of any further calculations.

Sincerely,

**FRANKLIN GEOTECHNICAL LTD.**

John A. Franklin, Ph.D., P.Eng., CPGS,  
President

JAF/amp

## **TABLE OF CONTENTS**

### **1. INTRODUCTION**

### **2. TEST METHODOLOGIES**

- 2.1 Specimen Preparation
- 2.2 Uniaxial Compression Tests
- 2.3 Moisture Contents and Units Weights

### **3. RESULTS OF TESTING**

Table 1:	Sample Information-Iona Sewer Project Rock Core
Table 2:	Test Results
Appendix 1:	Stress-Strain Graphs



## 1. INTRODUCTION

This report presents the results of tests on shale and limestone core from the Iona Sanitary Trunk Sewer Project in Hamilton, Ontario. The tests were conducted in the Rock Mechanics Laboratories at the University of Waterloo, on instructions from Franklin Geotechnical Ltd.

Table 1 identifies the core samples supplied by Peto MacCallum Ltd., Consulting Engineers. Fifteen specimens were tested, prepared from 22 core sticks of Niagara Escarpment Series rock formations.

## 2. TEST METHODOLOGIES

### 2.1 Specimen Preparation

The cores were delivered by Peto MacCallum to the University of Waterloo. They arrived in good condition; no damage had been caused by sampling or transport. The core sticks were 47 mm diameter and from 120 mm to 230 mm long.

ASTM Standards and ISRM Suggested Methods were used in determining Young's modulus, uniaxial compressive strength, moisture content, and wet/dry densities.

### 2.2 Uniaxial Compression Tests

Fifteen specimens were prepared for uniaxial compressive strength tests in accordance with the techniques outlined in ASTM D 4543-85; Standard Practice for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances. Cores were cut and surface ground, and each specimen was measured to ensure a length-to-diameter ratio (L/D) of 2.0 to 2.5 and all elements straight to within 0.50 mm.

Uniaxial compressive strength was determined after one or more unload/reload cycles. During the first unload cycle, the load frame was manually adjusted to prevent total unloading of the specimen, with a minimum of 0.3 kN of load being retained during this and all subsequent unloading cycles. Axial displacement was applied at a constant rate of 1% strain per five minutes.

Graphs were automatically plotted of force versus displacement during the test. The secant and tangent Young's moduli were calculated at stress levels of half the uniaxial compressive strength, using the ISRM Suggested Method for Determining Deformability of Rock Materials in Uniaxial Compression.

### 2.3 Moisture Contents and Unit Weights

The moisture contents of the rock cores were determined using the techniques outlined in ASTM D 2216 - 80; Standard Method for Laboratory Determination of Water (Moisture) Content in Soil, Rock, and Soil-Aggregate Mixtures.

### 3. RESULTS

The results of index tests are included in Table 1 and those for strength and modulus are presented in Table 2. Stress-strain plots are given in Appendix 1.

For the dolostone rocks, the average strength value of 79.2 MPa classifies this as a "very strong" rock according to ISRM and Canadian Foundation Engineering Manual criteria. The average tangent Young's Modulus of 32.8 GPa is in the middle of the range 17 to 100 GPa for crystalline limestones reported in Franklin & Dusseault (1989).

For the five Queenston Shale specimens tested (the rock formation through which the proposed tunnel is to pass), the average strength value of 15.8 MPa classifies this as a "weak" rock. The average tangent Young's Modulus of 11.9 GPa compares with a range of 2 to 30 GPa for low-durability shales reported in Franklin and Dusseault (1981). It is somewhat higher than the typical value of 1.3 GPa given for the Queenston shale in the report "Evaluation of Shales for Construction Projects", Franklin (1983), MTO Report RR229. This is not unexpected, because data on the Queenston Shale have been few and not very reliable. The values reported in the literature probably represent a weathered and softened shale, whereas the shale test results reported here apply to an unweathered shale recovered from a relatively deep drillhole.

Respectfully Submitted  
Franklin Geotechnical Ltd.



John A. Franklin, Ph.D., P.Eng., CPGS  
President



SAMPLE NUMBER	FORMATION	ROCK TYPE*	BH	DEPTH (ft, in)**	NOTES
1	Lockport	Dolostone	1	26'6"-27'1"	Waxed
2	DeCew	Dolostone	1	53'0"-53'7"	Waxed
3	Rochester	Shale	1	55'8"-56'5"	Unwaxed
4	Rochester	"	1	57'6"-58'0"	Unwaxed
5	Rochester	"	1	58'0"-58'7"	Unwaxed
6	Irondequoit	Limestone	1	63'4"-63'9"	Waxed
7	Reynales	Dolostone	1	64'8"-65'3"	Unwaxed
8	Thorold	Sandstone	1	74'10"-75'3"	Unwaxed
9	Grimsby	Shale	1	89'6"-90'0"	Waxed
10	Cabot Head	Shale	1	113'3"-113'9"	Waxed
11	Cabot Head	"	1	133'8"-134'2"	Waxed
12	Cabot Head	"	1	137'8"-138'2"	Waxed
13	Manitoulin	Dolostone	1	162'8"-163'5"	Unwaxed
14	Whirlpool	Sandstone	1	169'10"-170'4"	Waxed
15	Queenston	Shale	1	273'7"-274'3"	Waxed
16	Queenston	"	2	60'4"-60'11"	Waxed
17	Queenston	"	2	70'3"-70'10"	Unwaxed
18	Queenston	"	2	73'2"-73'9"	Waxed
19	Queenston	"	2	76'10"-77'4"	Unwaxed
20	Queenston	"	2	85'3"-85'10"	Unwaxed
21	Queenston	"	3	26'0"-26'6"	Waxed
22	Queenston	"	3	28'5"-28'11"	Unwaxed

\* rock type for the formation, not confirmed by thin section microscopy

\*\* samples and depth information supplied by Peto MacCallum Ltd.

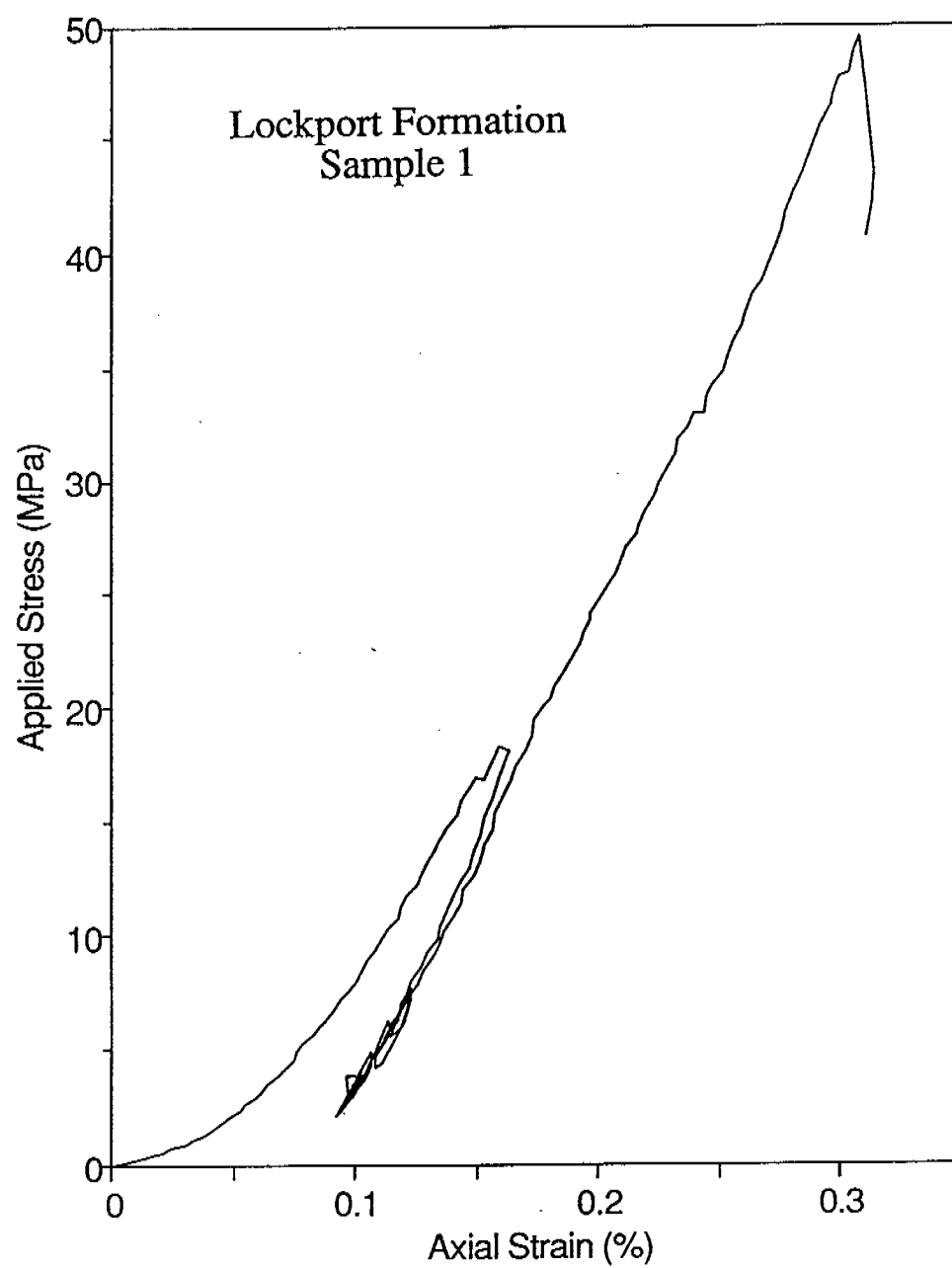
TABLE 1: SAMPLE INFORMATION - IONA SEWER PROJECT ROCK CORE

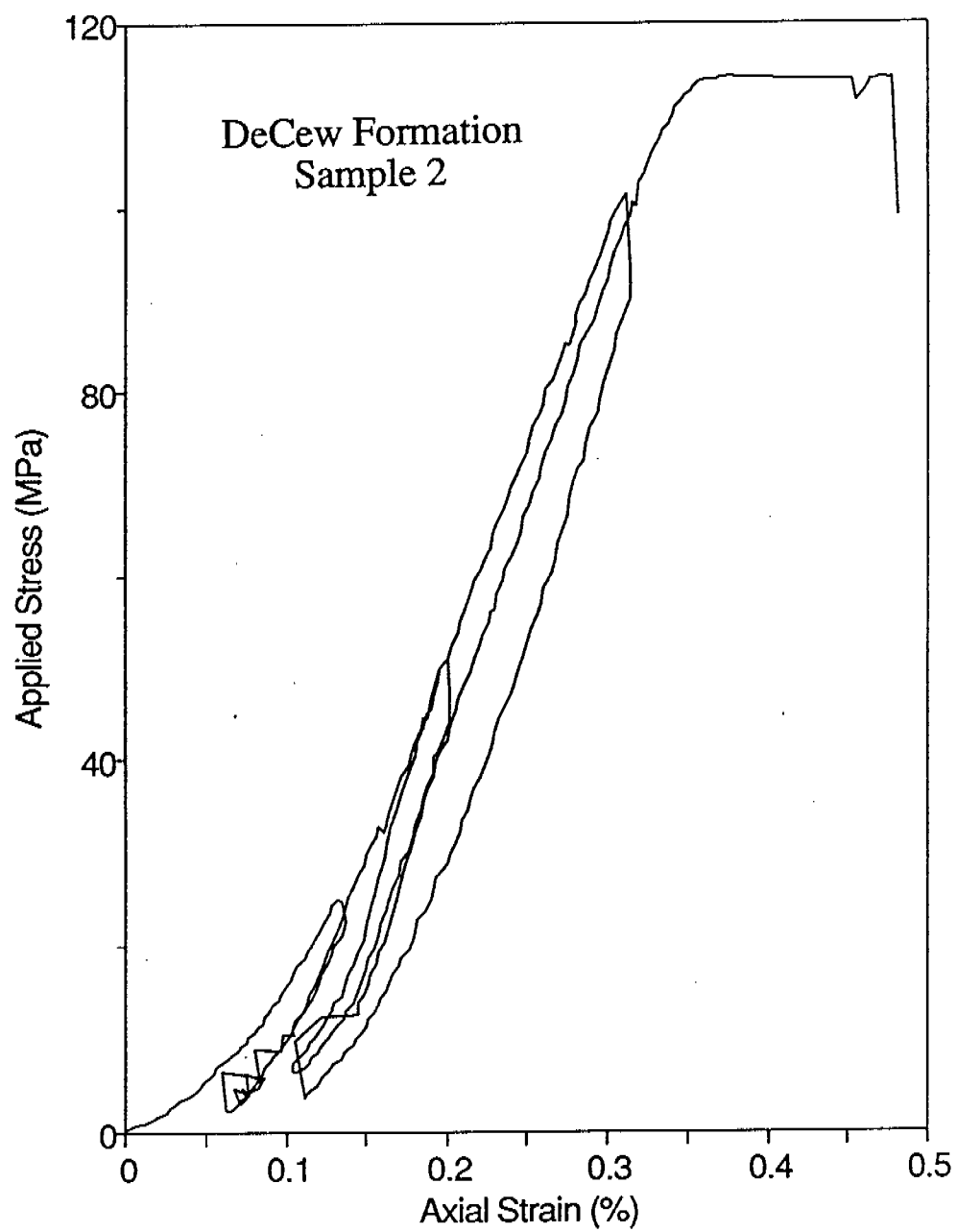
SAMPLE NUMBER	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	SONIC VELOCITY (km/s)	UNIAX. STRENGTH (MPa)	SECANT MODULUS (GPa)	TANGENT MODULUS (GPa)
1	2.84	2.56	4.56	49.44	12.32	18.44
2	1.34	2.60	4.56	113.95	22.25	44.40
3	0.37	2.43	5.18	25.72	13.77	25.40
4	0.22	2.19				
5	0.68	2.59				
6	1.61	2.00		76.83	21.95	33.50
7	0.55	2.96		112.23	31.98	47.07
8	0.61	2.12		86.39	22.79	34.98
9	5.00	2.16		9.55	2.72	3.33
10	3.59	2.56	3.00	7.04	1.81	2.69
11	4.20	2.27				
12	4.00	2.31				
13	0.43	2.83	5.10	41.88	6.58	20.78
14	5.50	2.12	4.26	94.17	27.19	39.62
15	2.45	2.68	3.34	13.30	5.84	8.64
16	1.74	2.56	2.99	8.61	3.28	3.78
17	0.47	2.52	3.65	19.07	11.05	15.88
18	3.51	2.51				
19	0.69	2.32	3.71	29.22	10.82	23.01
20	2.55	2.25				
21	4.66	2.21				
22	0.66	2.48	3.51	10.30	6.04	8.46

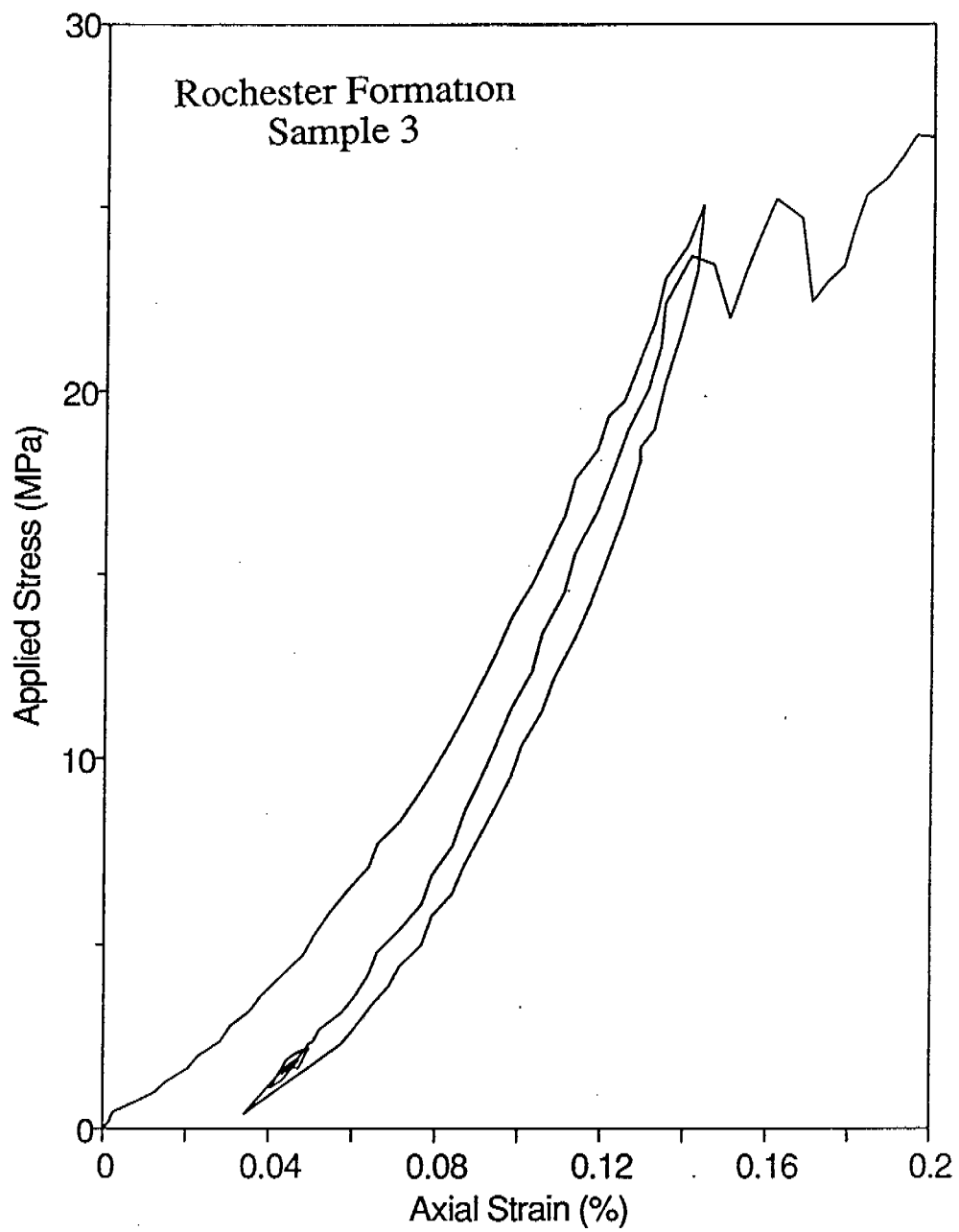
TABLE 2: TEST RESULTS

**APPENDIX 1**

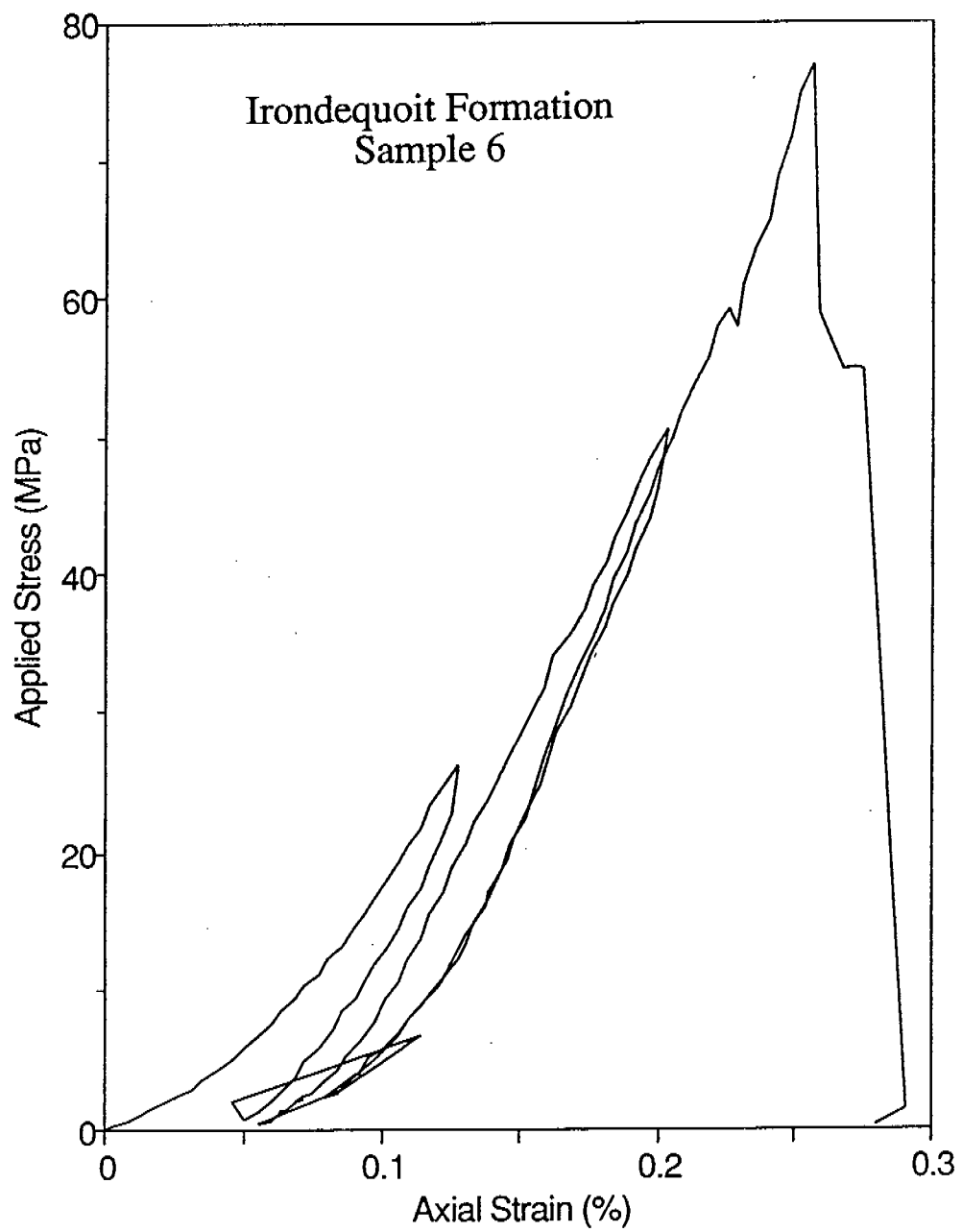
**STRESS-STRAIN GRAPHS**

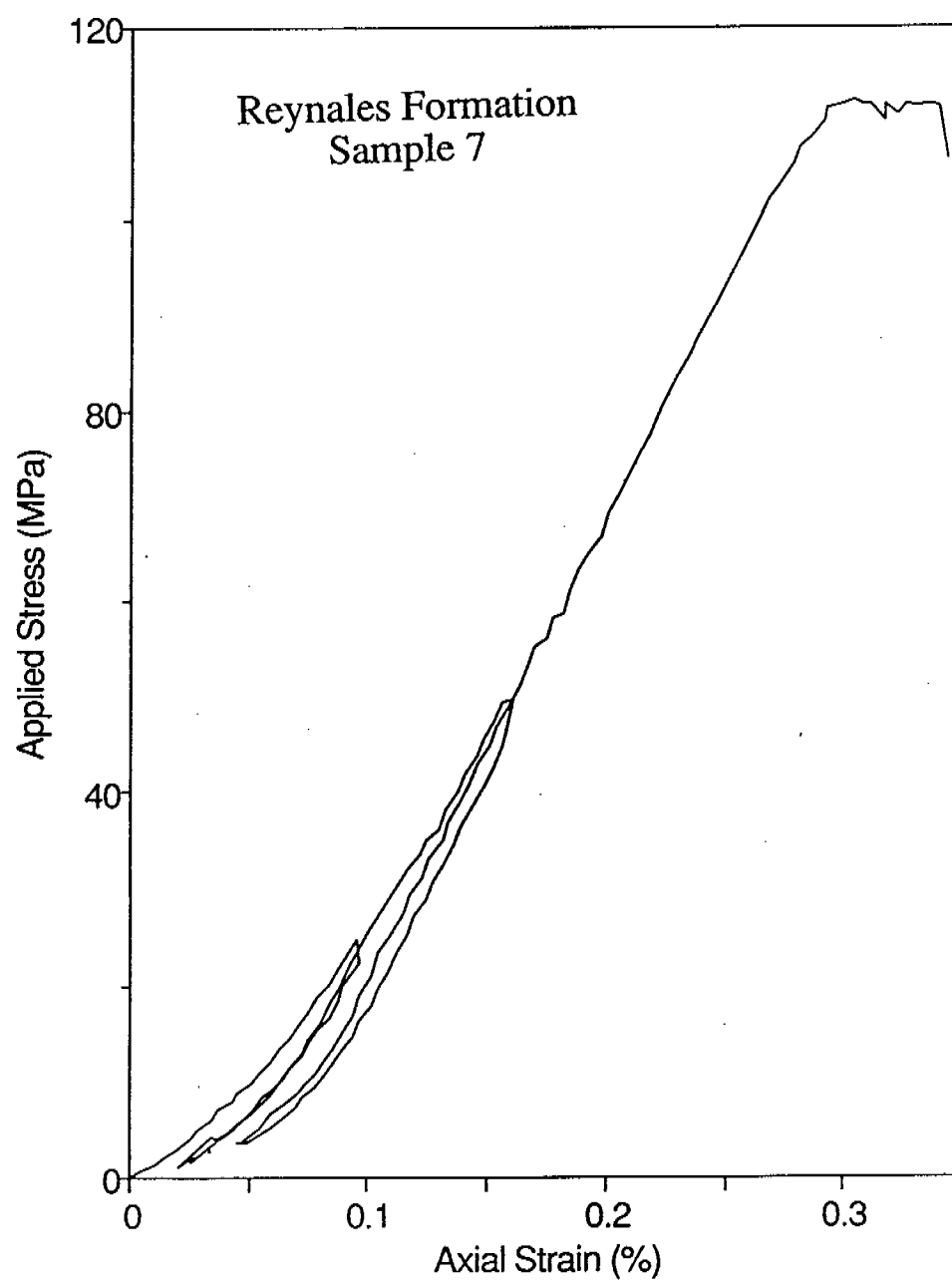


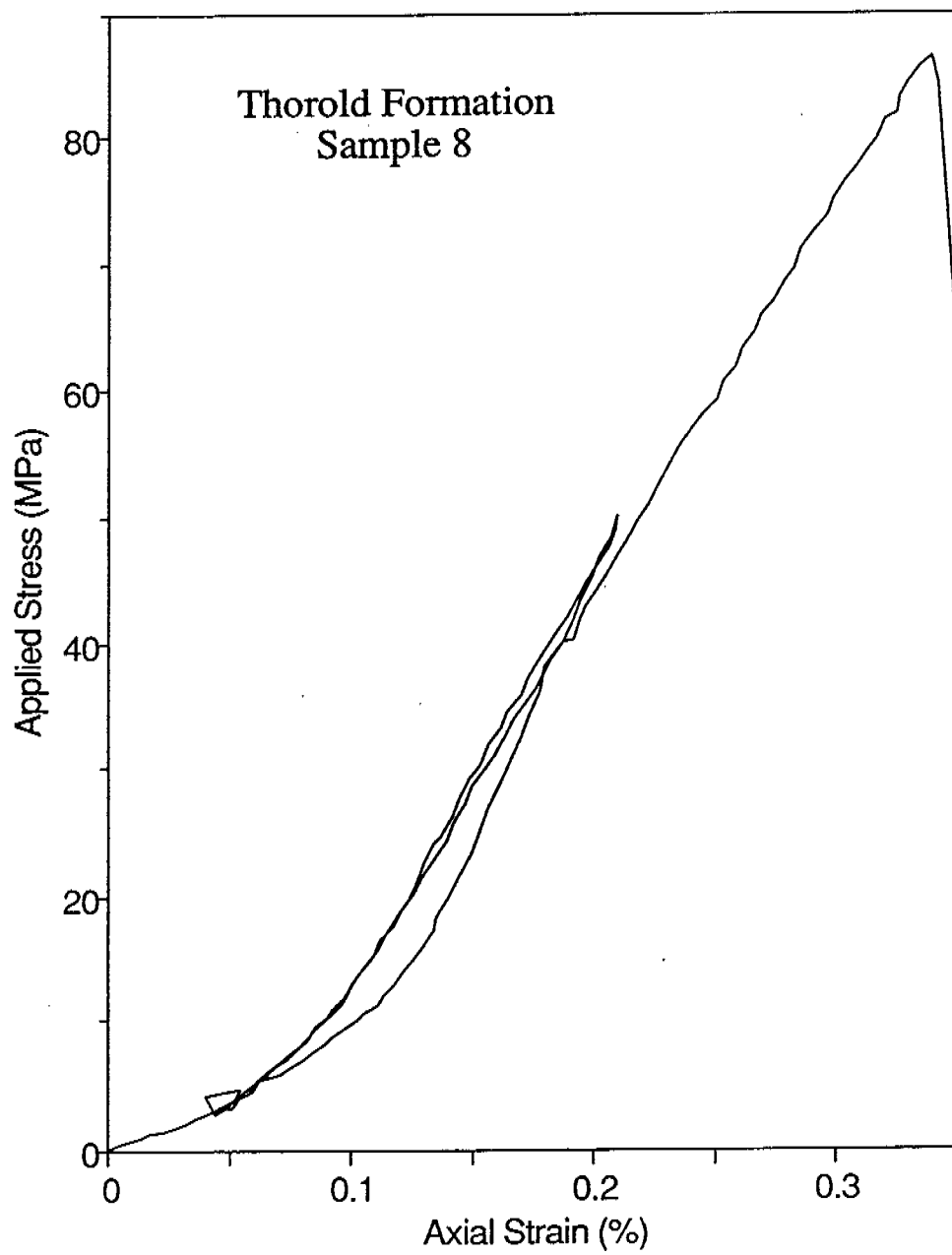


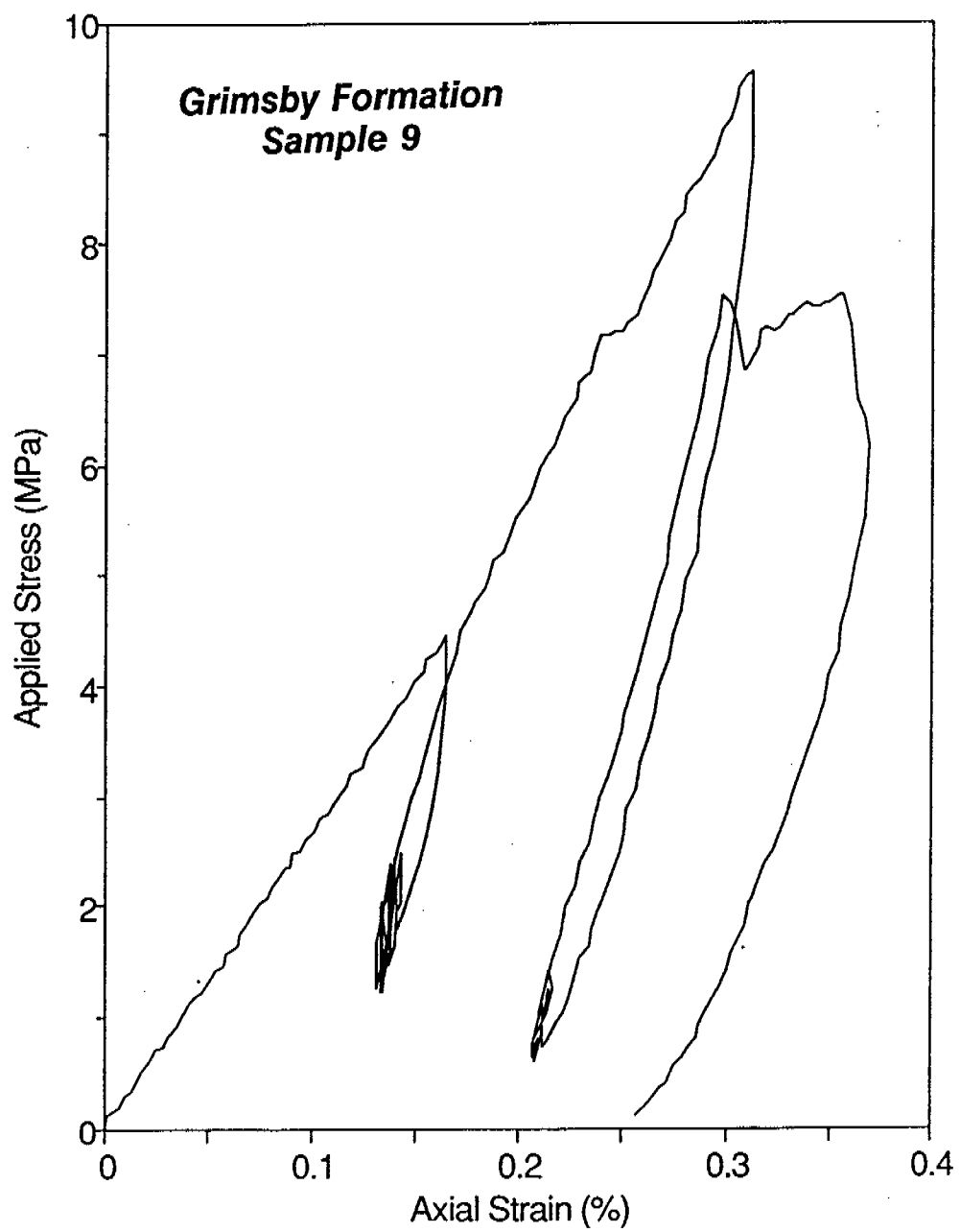


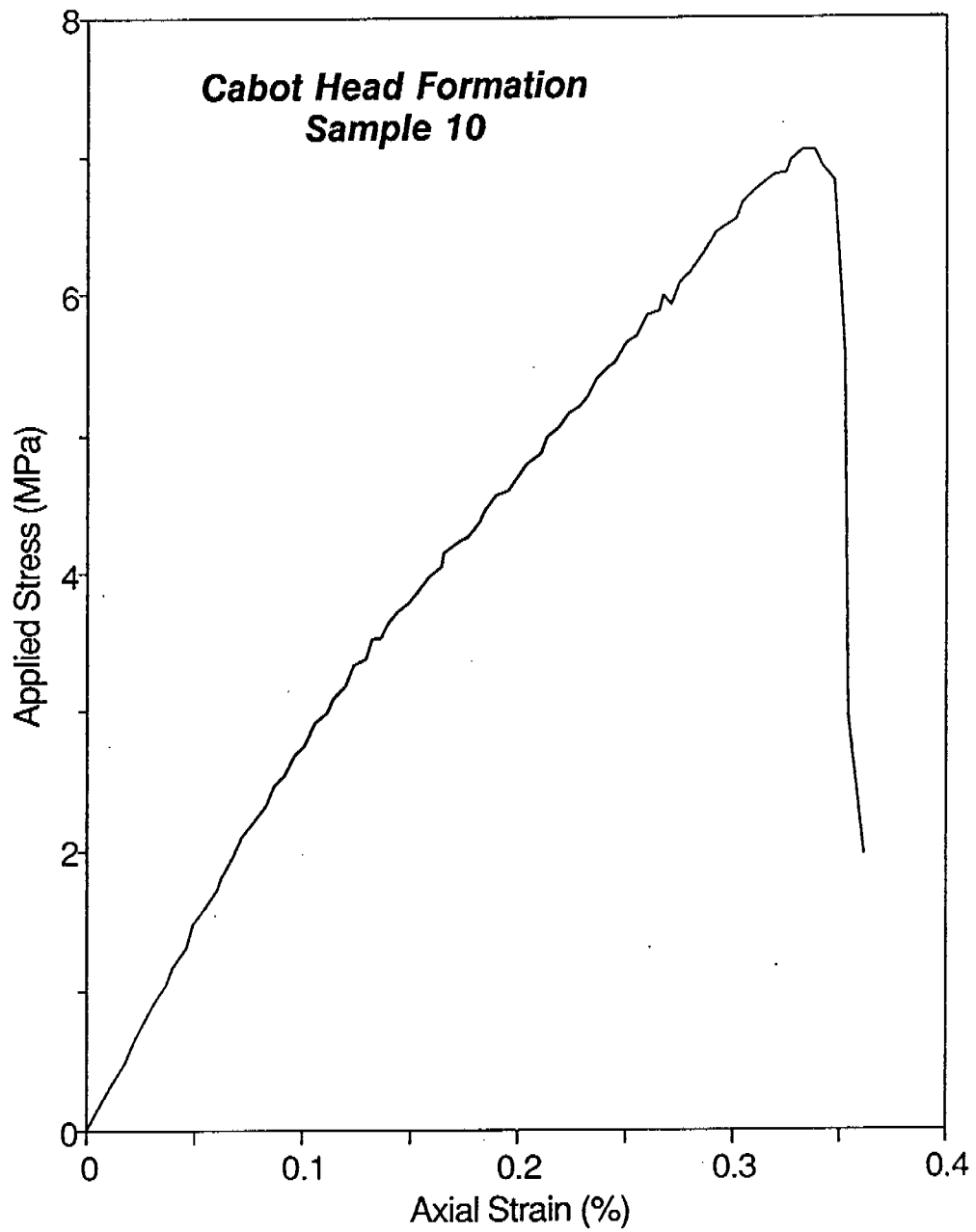


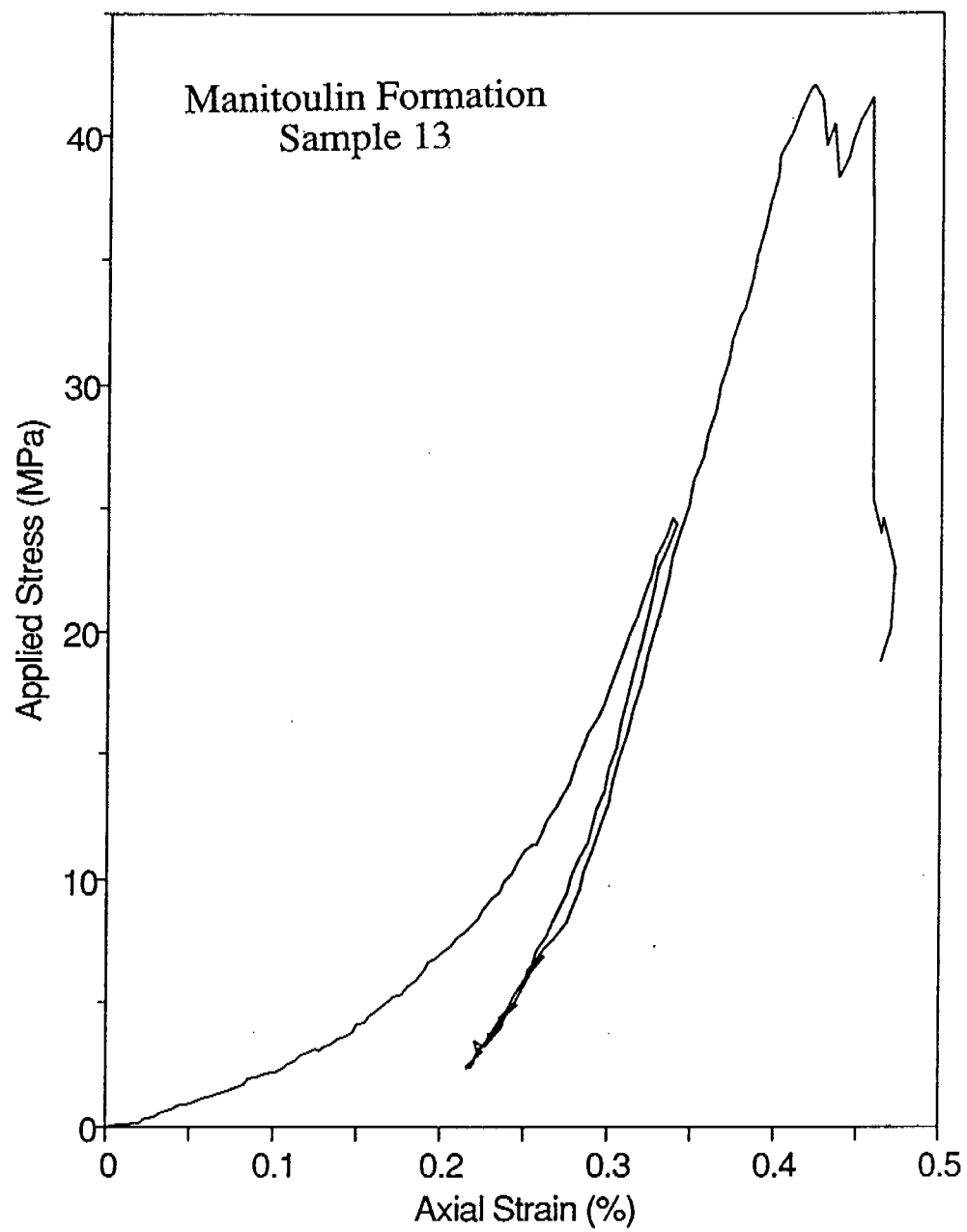


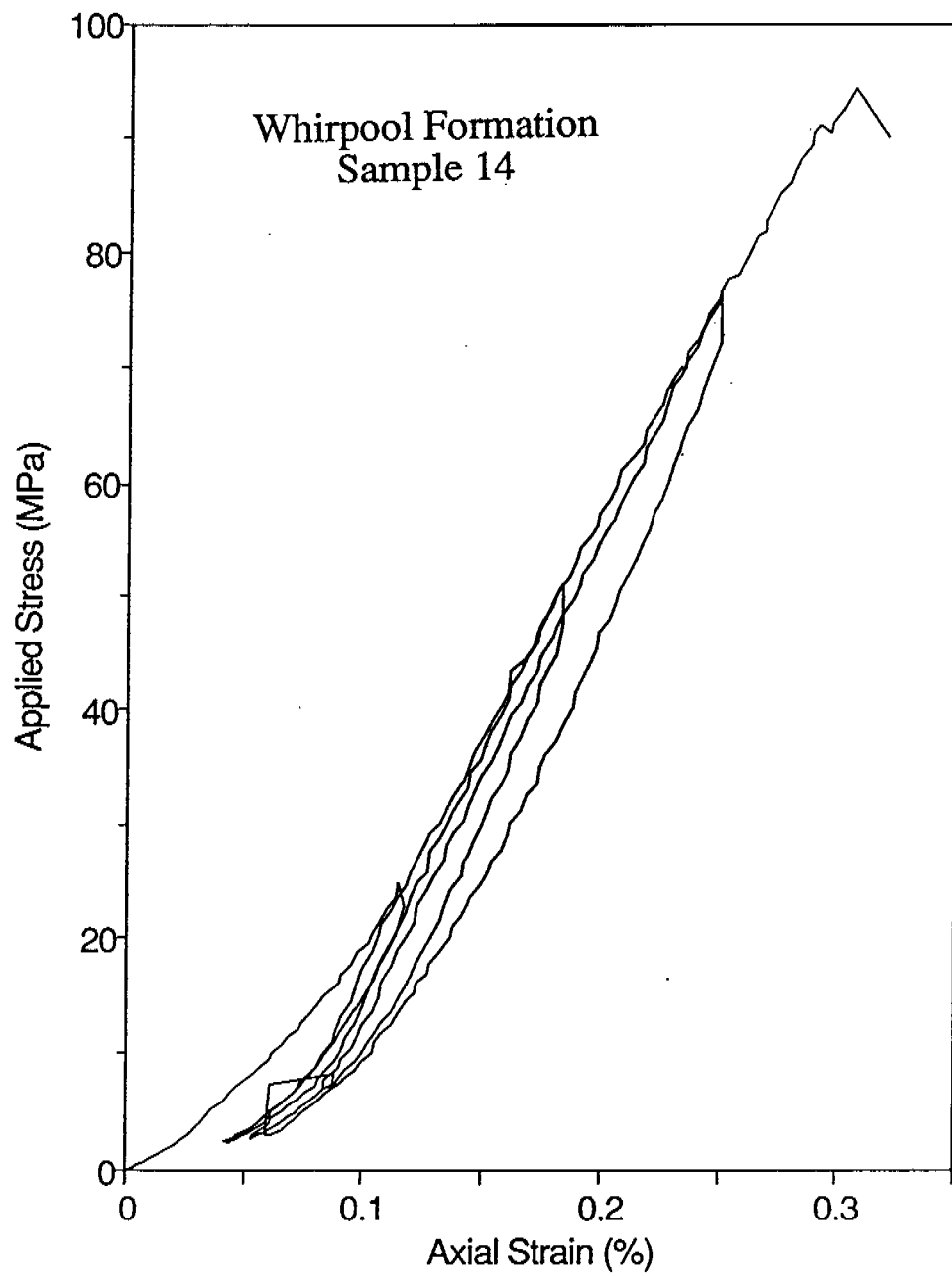


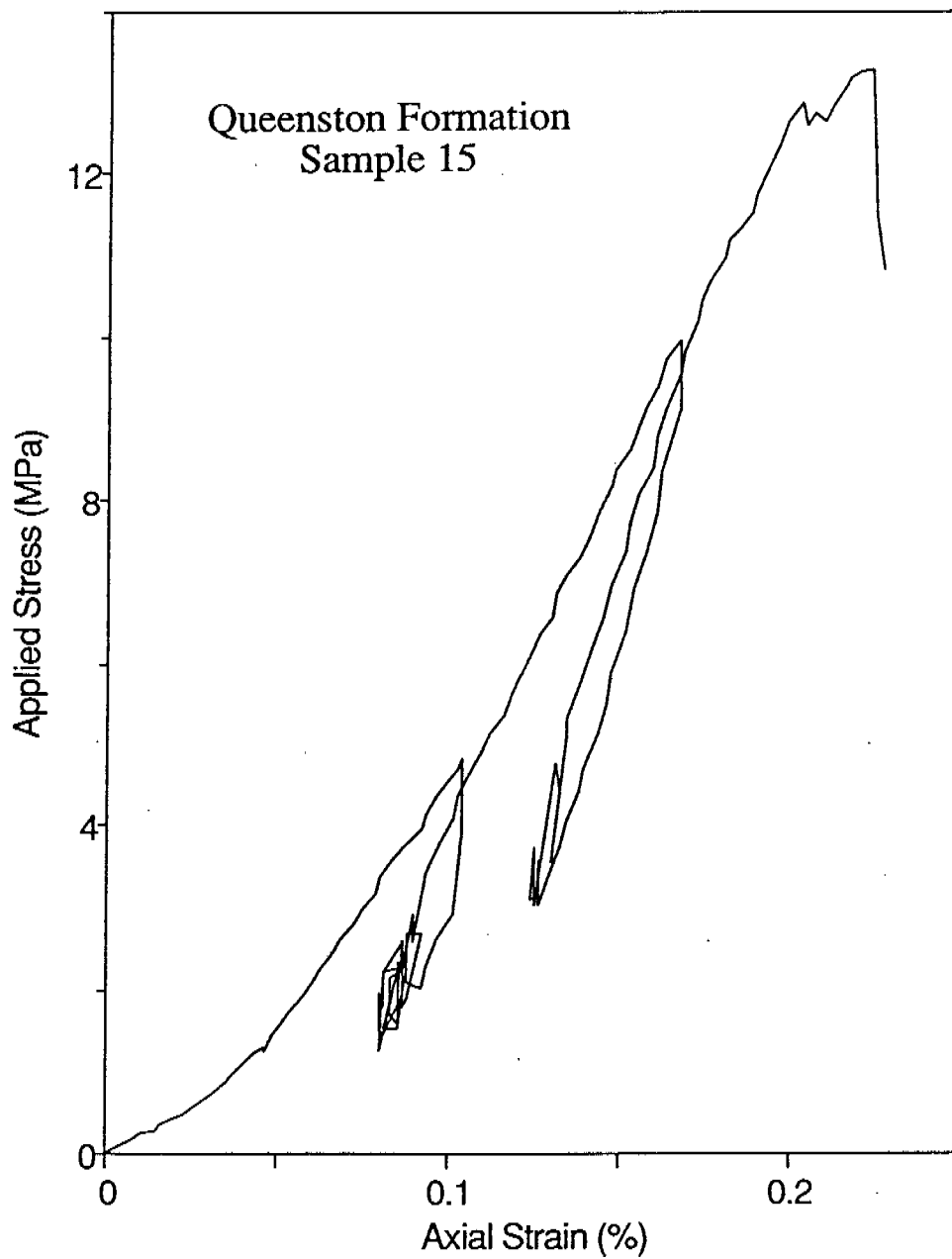




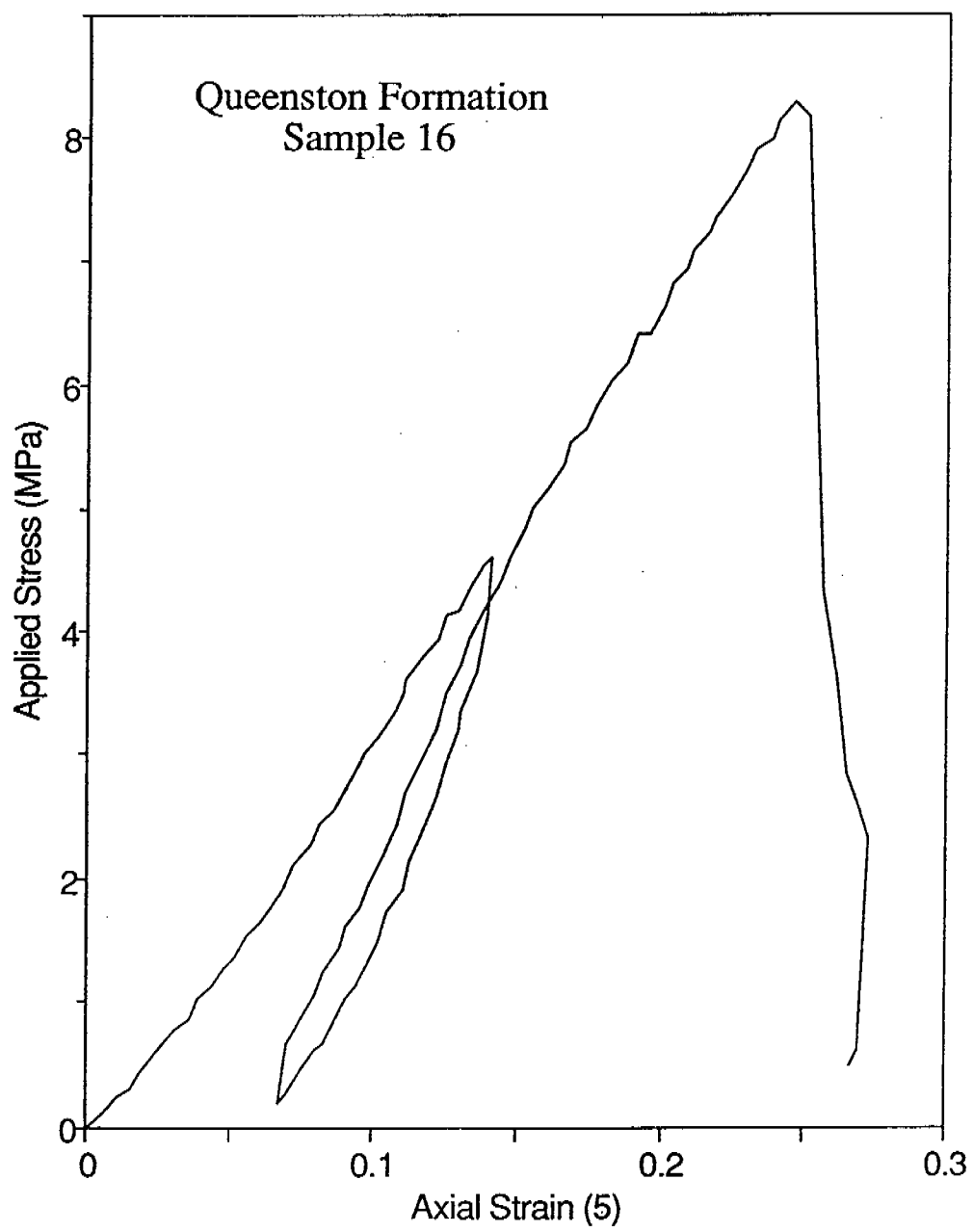


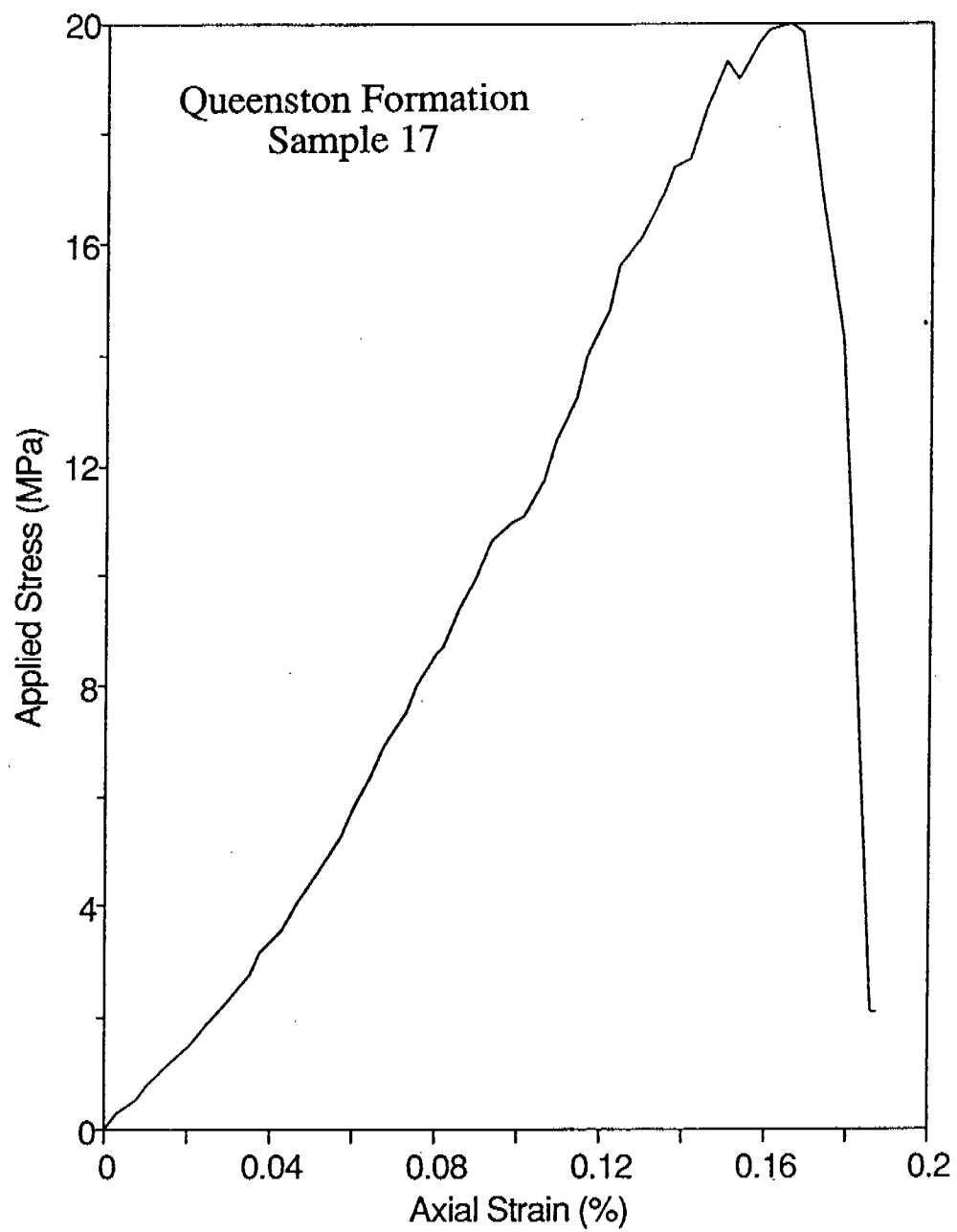


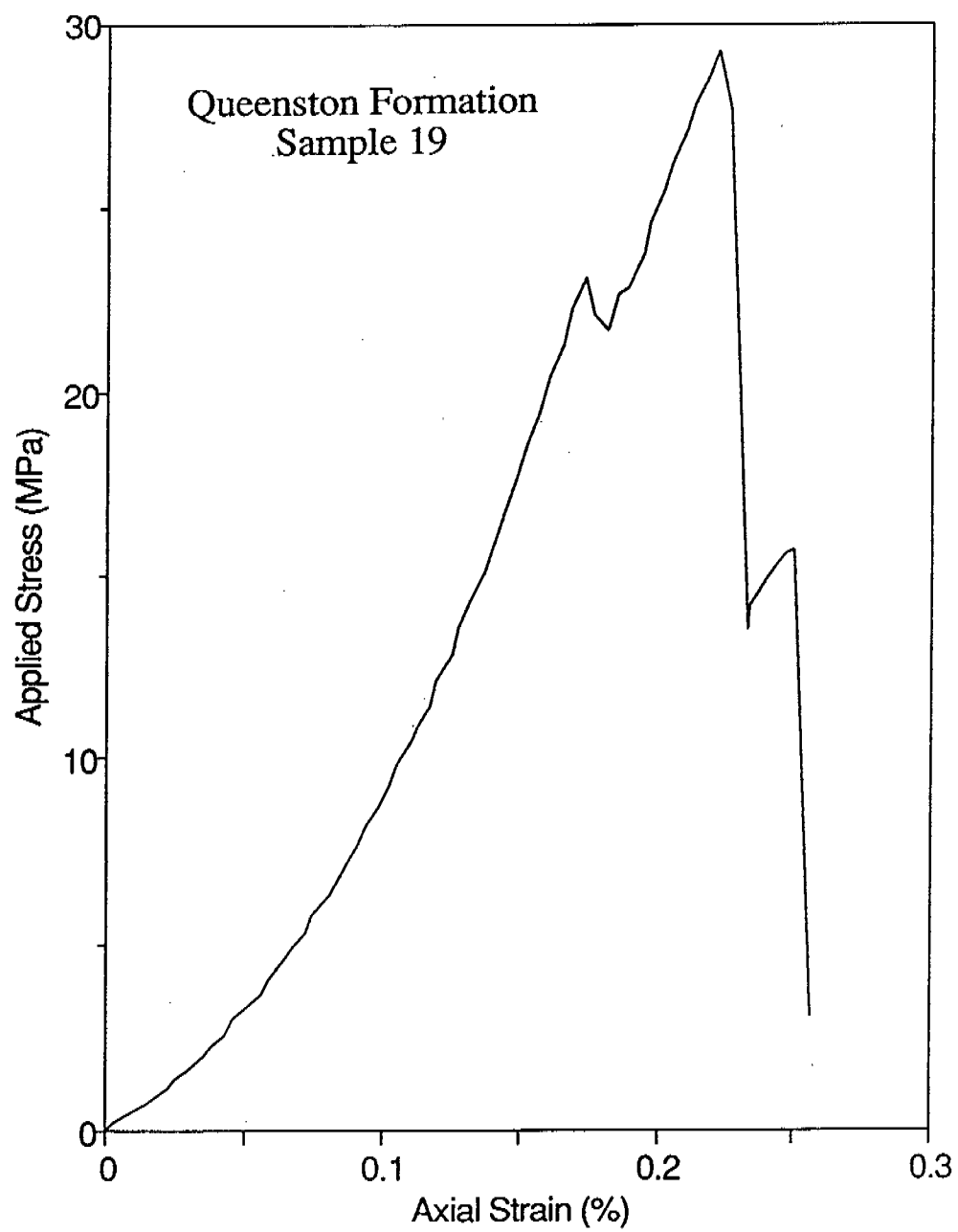


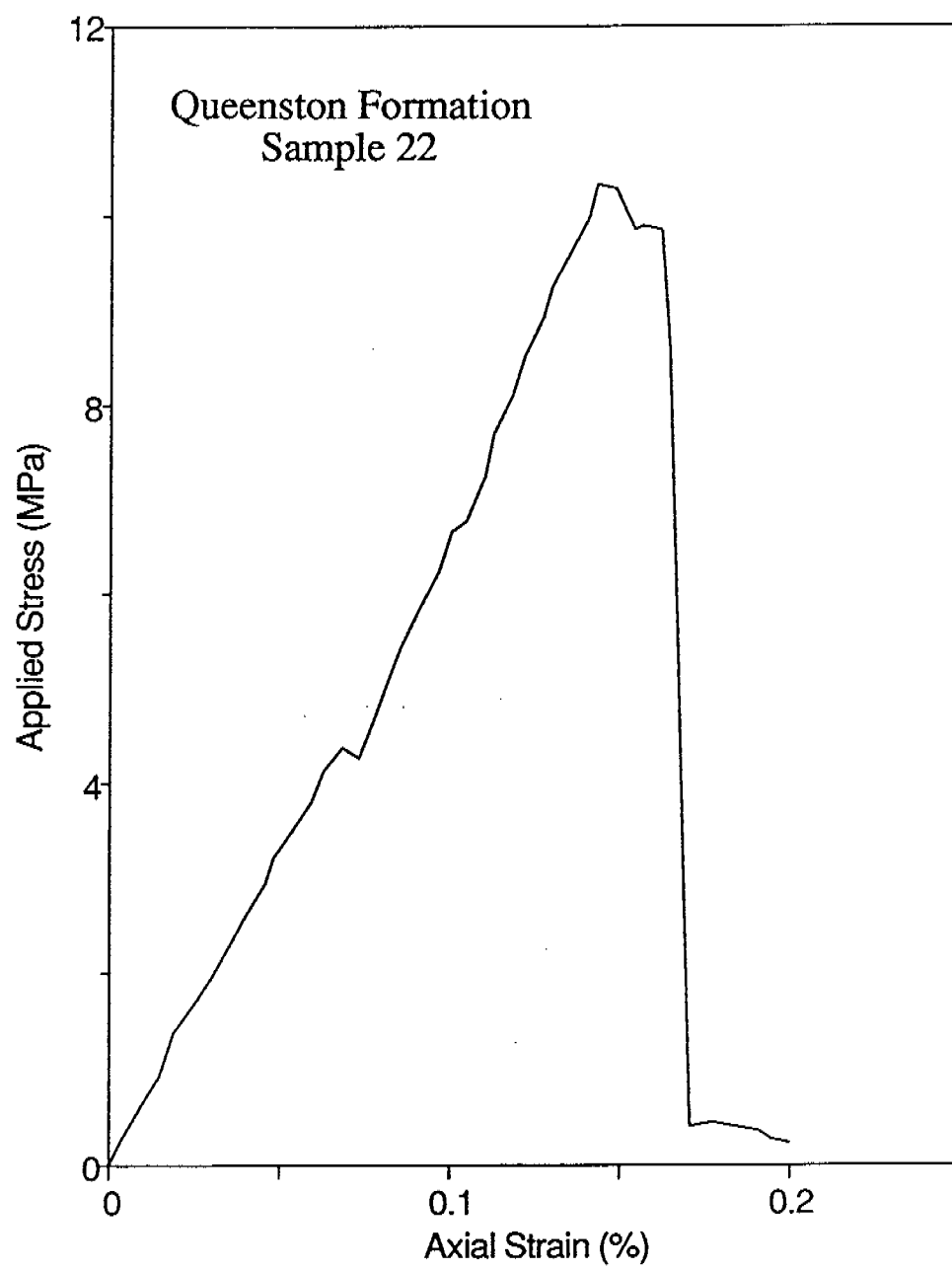












**APPENDIX B**

**Tunnel Liner Design Considerations**

**Provided by**

**Franklin Geotechnical Ltd.**



# **REPORT ON IONA SANITARY TRUNK SEWER HAMILTON, ONTARIO**

## **Input for Tunnel Liner Design**

Prepared for:

**PETO MACCALLUM LTD.**

45 Burford Rd.  
Hamilton, Ontario  
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Prepared by:

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Peto MacCallum Ltd.  
45 Burford Rd.  
Hamilton, Ontario  
L8E 3C6

June 3, 1991

Attention: Mr. Ty Garde, P.Eng.

Dear Ty,

#### INVESTIGATION

Enclosed is our Report G678.2 giving the results of this investigation. We trust this meets with your approval, and will be happy to answer any questions that may arise.

Yours truly,

**FRANKLIN GEOTECHNICAL LTD.**

John A. Franklin  
President

**Distribution:**

PML - 2  
FGL - 2

JAF:amp

/Enclosure

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- 2. VISCOT**
- 3. INPUT DATA**
  - 3.1 Tunnel Data
  - 3.2 Stress Assumptions
  - 3.3 Rock Properties
- 4. RESULTS OF NUMERICAL MODELLING**
- 5. CONCLUSIONS**
- 6. REFERENCES**

### **FIGURES**

- 1 Finite Element Model
- 2 Absolute Displacements vs Time After Excavation
- 3 Distribution of Horizontal Stresses Beneath Tunnel Invert



## 1. INTRODUCTION

Peto MacCallum Ltd. has engaged Franklin Geotechnical Ltd. as subconsultants to assist in rock mechanics aspects of an investigation for the Iona Sanitary Trunk Sewer Extension in Hamilton, Ontario.

Our recent Report G678.1 presented the results of tests on rock cores from the site investigation. This follow-up Report G678.2 contains responses to questions related to the design of the tunnel, as contained in Hamilton Wentworth Region's November 20, 1990 letter, and Addendum #1 of November 30, namely:

- Need for compressible materials around structures in shale (i.e. is the shale subject to convergence due to stress release?);
- Need for protection of shale after excavation;
- Modulus of Deformation and Poisson's Ratio of the rock;
- Ratio of fully mobilized horizontal pressure to vertical pressure acting on the tunnel lining;
- Estimate of convergence versus time for the tunnel and shafts;
- Feasibility of reducing liner stresses by delaying the installation of the liner;
- Estimation of pressure distribution around tunnel and shaft liners for an assumed delay time.

We discussed the rock mechanics requirements with Mr. Harvey Johnson and Mr. Douglas Dunbar of R.V. Anderson & Associates Ltd., and in our letter of 13th February offered to investigate interaction of the rock and liner using a finite element creep model similar to the one used in design of the Beulah Tunnel in North Dakota.

The computations were carried out under the supervision of Prof. Dusseault and by postgraduate students Milutin Petrovic and Pavel Vasak in the Earth Sciences Department at the University of Waterloo.

## 2. VISCOT

The numerical analyses have made use of VISCOT, a two-dimensional, non-linear, transient viscoelastic and viscoplastic finite element code for modelling the time-dependent mechanical behaviour of a rock mass.

The original version of VISCOT was developed by INTERA Environmental Consultants, Inc. for the National Waste Terminal Storage (NWTS) program of the United States Department of Energy, and was an adaptation of subroutines given in Owen and Hinton (1980).

The version of VISCOT that we adapted to study the Iona tunnel in shale had been developed by Professor Leo Rothenburg and employed by Rothenburg and Dusseault to simulate the behaviour of cavities mined into rock salt. The salt version employed a viscoelastic temperature- and stress-dependent constitutive model based on experimental work on salt creep at the University of Waterloo. The original viscoplastic routines were omitted, being inappropriate for the simulation of rock salt behaviour.

To study a tunnel in shale with a liner installed at a specified time after excavation, the following modifications were necessary:

- Reactivation of the subroutines for modelling elasto-viscoplastic behaviour;
- Rewriting part of the code to permit introduction of liner elements after some specified time.

The elasto-viscoplastic behaviour simulation was tested using the one-dimensional bar test cases published by Owen and Hinton (1980) and in the VISCOT manual.

To simulate lining of the tunnel, two compiled versions of VISCOT were used in parallel. The first version simulated creep of the unlined tunnel, and the second used an updated grid with liner elements added, taking as input the stress and strain results from the first run.

### 3. INPUT DATA

#### 3.1 Tunnel Data

The tunnel will be driven through the Queenston Shale Formation, probably by drill-and-blast methods. As presently envisaged, and as modelled using Viscot, it will have a circular, 3.6 m diameter cross-section, and a 300 mm thick concrete liner with compressive strength 35 MPa and Young's Modulus 30 GPa, Poisson's Ratio 0.17 (R.V. Anderson communication of 13th February, 1991).

#### 3.2 Stress Assumptions

Vertical stress was assumed to be gravitational, increasing linearly with depth. Rock cover was assumed as about 40 m, and with a stress gradient of 26 kPa/m for sedimentary rocks, this gives a vertical stress prior to excavation of 1 MPa at the tunnel crown.

In the absence of stress measurements for this project (which are difficult when the rock is a low strength shale), as a first approximation, the horizontal stresses were assumed equal and constant at 7.0 MPa (Figure 1). A constant horizontal stress over a limited range of relatively shallow depths is realistic, and the value of 7 MPa is typical of horizontal stresses that have been measured in Ontario shales. Values of up to 8.2 MPa were reported by Morton et al. (1975) for the Dundas Shale, and of 1.6 MPa by Franklin Trow Associates (1975) for the Collingwood Shale at the Easterly Filtration Plant tunnel in Scarborough. The value of 7 MPa is about one half of the 14 MPa which is a typical maximum for the stronger dolostones of southern Ontario.

#### 3.3 Rock Properties

The Queenston Shale samples tested (see Report G678.1) had an average uniaxial compressive strength of 15.8 MPa, and an average Young's Modulus of 11.9 GPa. Typical values reported by Franklin (1983) for the Queenston Shale are 8.7 (7.2 - 9.6) MPa for uniaxial compressive strength, and 1.3 (0.5 - 2.3) GPa for laboratory measurements of Young's Modulus. Hence the Iona tunnel

samples tested were 1.8 times as strong and 9.1 times as stiff as the previously reported "typical" values.

Time-dependent deformation (creep) consists of an elastic component and a viscoplastic component which are estimated using a constitutive model for the shale. No data are available from triaxial creep tests on Queenston Shale. Therefore, the "fluidity parameter" in VISCOT was estimated by calibrating the model to give a vertical convergence of 100 mm in one month, as measured at the Easterly Filtration Plant Intake Tunnel. Creep could be terminated by adjusting a strain hardening parameter, but for the results reported here, creep was allowed to continue at a decreasing rate indefinitely.

The behaviour monitored in the Easterly Filtration Plant intake tunnel in Scarborough (Franklin Trow Associates Ltd. Report F101, 1975) is considered the closest available comparison for the behaviour of the Iona tunnel in Queenston Shale, although the Collingwood shale at Scarborough is somewhat stronger and stiffer.

Estimation of the fluidity parameter should be checked by monitoring convergences in the Iona tunnel during construction, and if possible also by laboratory creep testing on samples of Queenston Shale. Creep tests on Queenston Shale are in progress at the University of Waterloo as part of an unrelated research project, but the results are unlikely to be ready in time to be useful for the Iona project.

It may be relevant to note that suitable creep test data were absent also for design of the liner for the Beulah Tunnel in North Dakota. Franklin Trow Associates Ltd., working for R.V. Anderson Associates Ltd., then made use of the best available alternative, which at the time was creep data for the London Clay in England (Hanafy, Emery and Franklin, 1976, 1977).

#### 4. RESULTS OF NUMERICAL MODELLING

Figure 2 shows the anticipated time-convergence behaviour of the tunnel, with primary support (shotcrete) to prevent shale loosening and drying-out, but before installation of a concrete liner. Figure 2 demonstrates that Viscot is capable of realistically simulating the parabolic shape of convergence-time graphs such as obtained in shale tunnels in southern Ontario.

The displacements shown are absolute, measured with respect to a fixed datum 18 m below the invert (Fig. 1). Horizontal convergence reaches about 40 mm (2 x 20 mm inward-positive wall displacements) after about 30 days, and continues at a decreasing rate. Vertical convergence includes about 20 mm of floor heave and 28 mm of roof sag for a total of about 48 mm after 30 days.

Following tunnel excavation in shale, a plastic (yielded) zone develops in which shear stress concentrations exceed the strength of the rock. Stresses are transferred into the surrounding unyielded rock, because the yielded material is no longer capable of sustaining shear stresses. Figure 3 shows the tangential stress profile beneath the invert, as the plastic zone develops with time. Because of the 7 MPa far-field horizontal stress, the invert is the most severely stressed location.

The solid line, which represents the initial elastic condition immediately after excavation, shows a stress concentration of about  $\times 2$  (the maximum value is about 14 MPa close to the invert). The stress concentration becomes insignificant about 6 m below the invert.

The various broken lines show stress distributions computed at various times after excavation. Values further than 4 m below the invert show a consistent trend towards stabilization with decreasing amounts of stress buildup as time elapses. The innermost ring of finite elements, however, shows anomalous values of stress that are considered unrealistic.

The anomaly is the result of a flaw found in the viscoplasticity subroutines of VISCOT. The stresses in the ring of elements directly adjacent to the opening continue to increase unrealistically with time. This increase in stress results from the stiffening of these elements. The flaw can be corrected relatively easily, given time. For the present, however, stress buildup on the concrete liner cannot be simulated realistically because it is exactly at this location that the anomaly occurs.

## 5. CONCLUSIONS

The following responses to the questions posed by the terms of reference are therefore based largely on experience in previous tunnels, rather than on the results of numerical modelling. This is not unusual: for example, in the recent guide to cavern engineering published by the Geotechnical Control Office of Hong Kong, the consulting engineers Bernal Strome point out that although large caverns are often analyzed to determine stresses and stability, the results are used mainly to extrapolate experience to cover conditions not previously encountered. An empirical approach governs the design even of these large and expensive rock caverns. They stress that "mathematical analysis is no substitute for experience".

We nevertheless hope to complete the calculations working alongside R.V. Anderson design engineers. The limit of resources available in the site investigation budget has been reached. Further analysis might be financed either from research or from project funding, but this remains to be determined. In the meantime, our responses to the questions posed in the Terms of Reference are as follows:

- The shale will converge as a result of the concentration of stresses around the tunnel. We anticipate a maximum pre-excavation horizontal stress of between 2 MPa and 7 MPa, accompanying a vertical stress of 1 MPa. The stresses will be magnified by a factor of about  $\times 2$  in the vicinity of the tunnel. Hence the levels of stress will approach or exceed locally the uniaxial compressive strength of the shale which is estimated at between 10 MPa and 16 MPa. In practice this presents little or no problem if the shale is shotcreted. A plastic yield zone will develop, although the thickness of this zone is likely to be limited to about 1 m if the shale is confined by appropriate primary support.

The Regional Municipality of Hamilton Wentworth have adopted the routine of installing a compressible layer between concrete liners and rock, as a result of earlier problems in the Red Hill Creek and other cut-and-cover projects. A compressible buffer would help here also, although it is not so easy to implement in the case of a circular fully below-ground tunnel. A sprayed foam liner might be worth investigating.

- The Queenston shale has a relatively low durability and should be prevented from drying out. In the tunnel, the immediate application of a shotcrete (fibercrete) primary liner is

recommended, which will serve the multiple purposes of preventing drying, inhibiting loosening, and permitting controlled displacements and arching to develop. The liner also assists in monitoring ongoing movements.

- In further calculations we recommend assuming a Modulus of Deformation of 10 GPa and a Poisson's Ratio of 0.3 for the rock.
- The ratio of fully mobilized horizontal pressure to vertical pressure acting on the tunnel lining remains to be computed.
- Convergence is likely to develop in the shotcreted tunnel to a maximum of about 40 mm over a period of about 1 month, becoming slower thereafter. The rates of tunnel liner displacement are likely to be similar to those shown in Fig. 2. Much less convergence is expected in the case of the shaft, which because of the dolostone and other hard rock beds, should not experience the same problem of rock squeeze.
- Installation of the primary shotcrete liner should not be delayed. Shotcrete should be applied daily to keep pace with excavation. The final liner can be installed on completion of the tunnel provided that rates of convergence appear to have stabilized to acceptable values. Further delays are unlikely to be beneficial.
- Further calculations might assist in estimating the reductions in liner pressures as a function of delay time between excavating and lining. However, these calculations are seldom reliable, particularly in the absence of creep test data. The best alternative is to use tunnel monitoring to calibrate the creep law, and to re-run the calculations during construction as an aid to deciding whether delays are justified.

## 6. REFERENCES

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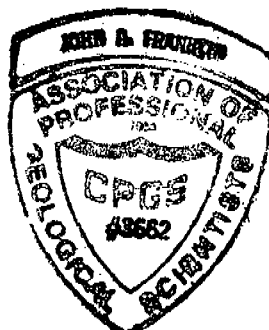
HANAFY E.A., EMERY J.J. and FRANKLIN J.A., 1976. Tunnel lining strategies. Proc. 3rd Symp. Eng. Applications of Solid Mechanics, Toronto, Vol. 2, pp. 179-202.

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Respectfully submitted,  
FRANKLIN GEOTECHNICAL LTD.



John A. Franklin, Ph.D., P.Eng., CPGS,  
President



## FIGURES



## CASE 1: 7 MPa UNIFORM

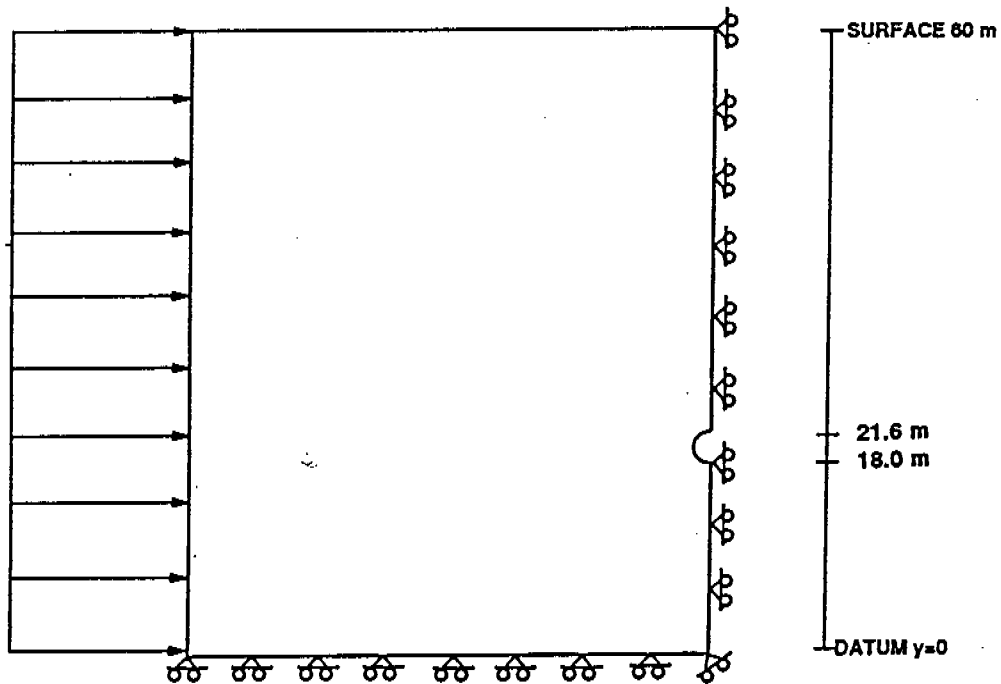

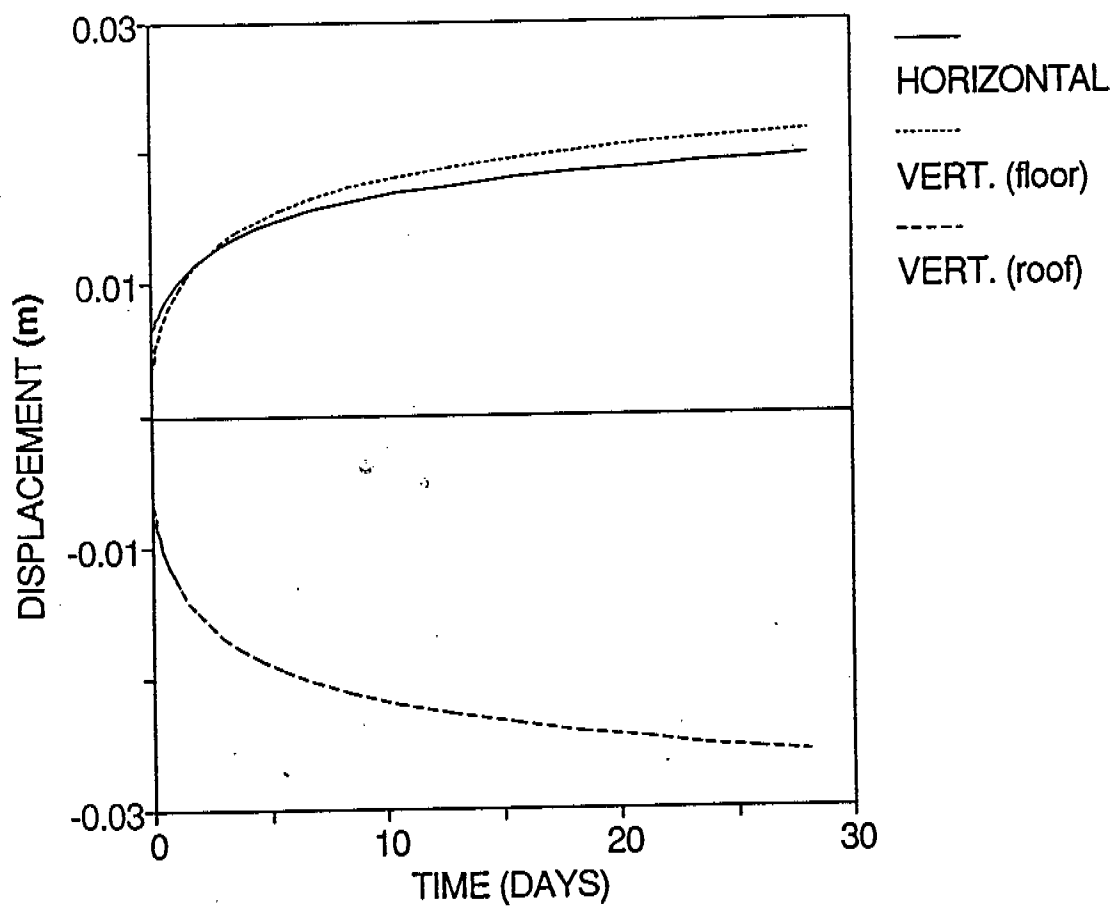



FIG 1: FINITE ELEMENT MODEL

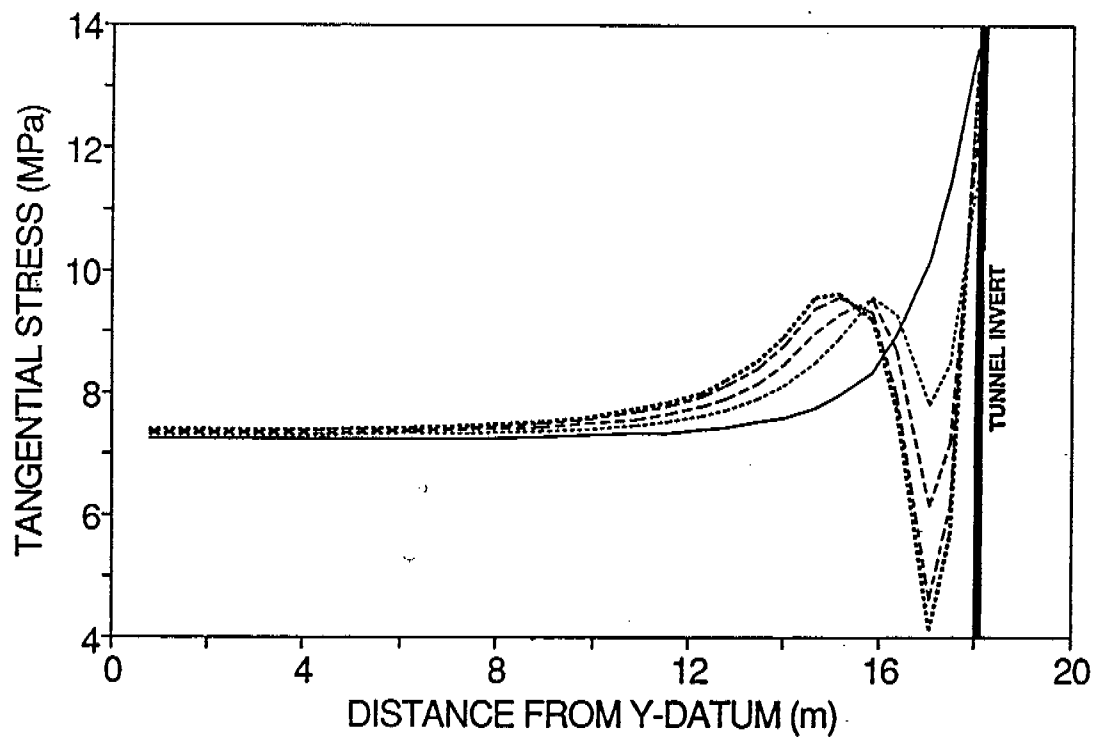
Drawn by:	NAME	DATE		franklin geotechnical ltd.	
	PV	MAY91		PROJECT:	FIGURE:
	Checked:			G678.2	1
	Revisions:				



**FIG 2: ABSOLUTE DISPLACEMENTS VS TIME AFTER EXCAVATION**


Drawn by:	NAME	DATE		franklin geotechnical ltd.	
	PV	MAY91		PROJECT: G678.2	FIGURE: 2
	Checked:				
	Revisions:				





— time = 0      ..... time = .6 days      - - - - time = 3 days  
 - . - . time = 13 days      ..... time = 28 days

**FIG 3: DISTRIBUTION OF HORIZONTAL STRESSES BENEATH TUNNEL INVERT**

Drawn by:	NAME	DATE		franklin geotechnical ltd.	
	PV	MAY91		PROJECT: G678.2	FIGURE: 3
	Checked:				
	Revisions:				

### LOG OF BOREHOLE NO. 1

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 & 27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE			SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	NUMBER	TYPE								
	GROUND ELEVATION 190.32											
0.15	TOPSOIL: Dark brown, silty clay, low organic		190									
	CLAY: Very stiff, khaki brown, silty clay with trace of sand, medium plastic, W.T.P.L.		189	1 SS	19*							
1.5			188									
2.80	commence rotary coring		2	SS	225 mm & bouncing							
3.0	DOLOSTONE: Tan to light grey, aphanitic to fine-grained, massive to thick bedded, numerous white chert nodules from 12 to 75 mm diameter, random vugs lined with chert/calcite in upper section, fair becoming excellent quality		187	RC								
4.5			186	3 NQ	1750	95	54	100				
6.0			185	RC								
			184	4 NQ	3050	100	66	100				
7.5	Lockport Formation - Goat Island Member		183									
9.0			182									
			181	5 RC NQ	3050	100	67	100	2.8	2.56	49.4	
10.5			180									
12.0			179	RC								
			178	6 NQ	3050	100	93	100				
13.10	DOLOSTONE: Grey, medium-grained massive to thickly bedded, excellent quality, Lockport Formation - Gasport member		177									
14.45	DOLOSTONE: Decew Formation		176									
15.0	BOREHOLE CONTINUED ON NEXT SHEET		175									
16.5												

NOTES: \* 1) Within overburden, value denotes 'N' values in blows/0.3 m.  
2) R.Q.D. (Rock Quality Designation) is the total length of NQ core segments longer than 100 mm divided by the drill run length, expressed as a percent.

CHECKED BY: *TJ*

# Peto MacCallum Ltd.

CONSULTING ENGINEERS

2 of 7

## LOG OF BOREHOLE NO. 1

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 &  
27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN(mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
	GROUND ELEVATION 190.32												
15.0	DOLOSTONE: Medium grey, aphanitic, massive to medium bedded, excellent quality		175		RC								
			174	7	NQ	3050	100	88	100	1.3	2.60	114.0	
16.5	Decew Formation												
	SHALE AND LIMESTONE: Dark grey interbedded shale and limestone fine-grained - highly fractured good to excellent quality, Rochester Formation		173		RC					0.4	2.43	25.72	
17.95										0.2	2.19		
18.0										0.7	2.59		
	LIMESTONE: Light grey, medium-grained, crystalline, porous, stylolitic, excellent, quality, Irondequoit Formation		172	8	NQ	3050	100	92	100				
			171							1.6	2.00	76.8	
19.45													
19.5	DOLOSTONE: Tan to light grey, fine-grained to aphanitic, massive, shale partings throughout, good to excellent quality, Reynales Formation		170		RC								
			169	9	NQ	3050	100	90	100	0.6	2.96	112.2	
21.0													
22.00													
	SANDSTONE: White to light grey medium to fine-grained sandstone, small scale cross bedding, layers of grey to green shale in lower section, good quality Thorold Formation		168										
22.5			167										
24.0			166		RC					0.6	2.12	86.4	
24.60													
	SHALE AND SANDSTONE: Interbedded red and green shale and sandstone, massive to thinly bedded, abundant shell fragments, shale deteriorates rapidly upon exposure, low strength		165	10	NQ	3050	100	88	100				
25.5			164										
27.0			163	11	NQ	3050	100	88	100	5.00	2.16	9.6	
			162		RC								
28.5	Grimsby Formation												
			161										
30.0	BOREHOLE CONTINUED ON NEXT SHEET		160										
31.5													

NOTES:

CHECKED BY: TA

### LOG OF BOREHOLE NO. 1

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 &  
27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
	GROUND ELEVATION 190.32												
30.70	Grimsby Formation Cont'		160		RC								
	SHALE: Grey, occ. sandstone/ limestone layers, low strength, fair quality		159	12	NQ	3050	100	80	100				
			158										
			157		RC								
			156	13	NQ	3050	100	67	100	3.6	2.56	7.0	
			155										
			154		RC								
			153	14	NQ	3050	100	73	100				
			152										
			151		RC								
			150	15	NQ	3050	100	45	100	4.2	2.27		
	becoming poor quality, clayey seams		149										
			148		RC								
			147	16	NQ	3050	100	61	100	4.0	2.31		
			146										
			145										
	becoming fair quality												
	BOREHOLE CONTINUED ON NEXT SHEET												

NOTES:

CHECKED BY: T.J.

### LOG OF BOREHOLE NO. 1

PROJECT IONA SANTIARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario



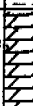


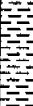
BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 & 27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN(mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
	GROUND ELEVATION 190.32												
45	Cabot Head Formation		145		RC								
			144	17	NQ	3050	100	53	100				
			143										
40			142		RC								
			141	18	NQ	3050	100	81	100				
49.15			140										
	DOLOSTONE: Buff to light grey, dolostone with interbedded shale good quality Manitoulin Formation		141							0.4	2.83	41.9	
50.35	SANDSTONE: White, massive to finely bedded, quartz rich, separation of core along thin shale fragments parallel to bedding, fair quality Whirlpool Formation		139		RC								
			138	19	NQ	3050	100	64	100	5.5	2.12	94.2	
			137										
53.75	SHALE: Red and green mottled, massive, occ. calcite infilling, poor quality Queenston Formation		136		RC								
			135	20	NQ	3050	100	30	100				
			134										
57	becoming fair quality		133		RC								
			132	21	NQ	3050	100	71	100				
			131										
60	BOREHOLE CONTINUED ON NEXT SHEET		130										

NOTES:

CHECKED BY: *TJ*

**LOG OF BOREHOLE NO. 1**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

OUR PROJECT NO. 91HF007

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE March 25, 26 &  
27, 1991

ENGINEER T.J. Garde

BORING METHOD Rotary Diamond Coring, NQ Core Size

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
60	GROUND ELEVATION 190.32												
	QUEENSTON FORMATION		130		RC								
	becoming good quality		129	22	NQ	3050	100	85	100				
			128										
63			127		RC								
			126	23	NQ	3050	100	88	100				
			125										
66	become fair quality		124										
			123	24	NQ	3050	100	70	100				
			122										
69			121										
			120	25	NQ	3050	100	73	100				
			119										
72			118		RC								
			117	26	NQ	3050	100	54	100				
			116										
75	BOREHOLE CONTINUED ON NEXT SHEET		115										

NOTES:

CHECKED BY *TL*

## LOG OF BOREHOLE NO. 1

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE March 25, 26 &  
27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

BORING METHOD Rotary Diamond Coring, NQ Core Size

[illegible]

NOTES:

CHECKED BY: Tg

**LOG OF BOREHOLE NO. 1**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 25, 26 &  
27, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAXIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
	GROUND ELEVATION 190.32												
90	Queenston Shale		100										Tunnel Obvert 100.2
			99	RC									
			32	NQ	3050	100	95	100					
			98										
			97										
93	becoming fair quality		96	RC									
			33	NQ	3050	100	55	100					
			95										Tunnel Invert 95.8
			94										
			93	RC									
			34	NQ	3050	100	85	100					
	becoming good quality		92										
99	99.15		91										
	BOREHOLE TERMINATED AT 99.15 m DEPTH												
102													
105													

NOTES:

CHECKED BY: *TJG*



### LOG OF BOREHOLE NO. 2

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 17, 18 &  
19, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE								
	GROUND ELEVATION 120.00												
0.15	TOPSOIL: Dark brown silty clay, low organic												
	CLAY TILL: Brown, silty clay, some sand, medium plastic		119	1	SS	19*							
1.35	SHALE: Red and grey mottled shale, poor rock quality, low strength, strength increases with depth		118		RC								
	Queenston Formation		117	2	NQ	3050	100	33	100				
3			116										
			115		RC								
6	becoming fair rock quality		114	3	NQ	3050	83	66	100				
			113										
			112		RC								
			111	4	NQ	1500	100	59	100				
9			110		RC								
			109	5	NQ	2315	86	55	100				
12			108		RC								
			107	6	NQ	3050	98	85	100				
	becoming good rock quality		106										
			105										
15	BOREHOLE CONTINUED ON NEXT SHEET												

NOTES: \* 1) Within overburden, value denotes 'N' values in blows/0.3 m.  
2) R.Q.D. (Rock Quality Designation) is the total length of NQ core segments longer than 100 mm divided by the drill run length, expressed as a percent.

CHECKED BY: T.J.

## LOG OF BOREHOLE NO. 2

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Rotary Diamond Coring, NQ Core Size

BORING DATE March 17, 18 &  
19, 1991

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

[illegible]

NOTES:

CHECKED BY. T2

**LOG OF BOREHOLE NO. 3**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

OUR PROJECT NO. 91HF007

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE April 2 & 3, 1991 ENGINEER T.J. Garde

BORING METHOD Rotary Diamond Coring, NQ Core Size

TECHNICIAN M. Rapsey

SOIL PROFILE			SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	UNIAxIAL STRENGTH (MPa)	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER								
	GROUND ELEVATION 106.02											
	Borehole advanced without sampling to bedrock surface		105									
	From cuttings - Clay Till		104									
1.5												
3.0	Commence Rotary Drilling		103									
3.35	SHALE: Red, massive, excellent rock quality becoming good		102	1	RC							
					NQ	1220	92	100	100			
4.5			101		RC							
	Queenston Formation		100	2	NQ	3050	100	90	100			
6.0			99									
7.5			98		RC							
9.0			97	3	NQ	3050	100	76	100	4.7	2.21	
9.30										0.66	2.48	98.6 Tunnel Obvert
	very poor rock quality, very weak shale		96					0				
10.5			95		RC							
	becoming good rock quality		94	4	NQ	3050	100	88	100			
12.0			93									94.2 Tunnel Invert
13.5	becoming excellent rock quality		92		RC							
			91	5	NQ	1525	100	93	100			
15.0												
15.40	BOREHOLE TERMINATED AT 15.40 m DEPTH		90									
16.5												

NOTES: 1) R.Q.D. (Rock Quality Designation) is the total length of NQ core segments longer than 100 mm divided by the drill run length, expressed as a percent.

CHECKED BY: *TL*

## LOG OF BOREHOLE NO. 4

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

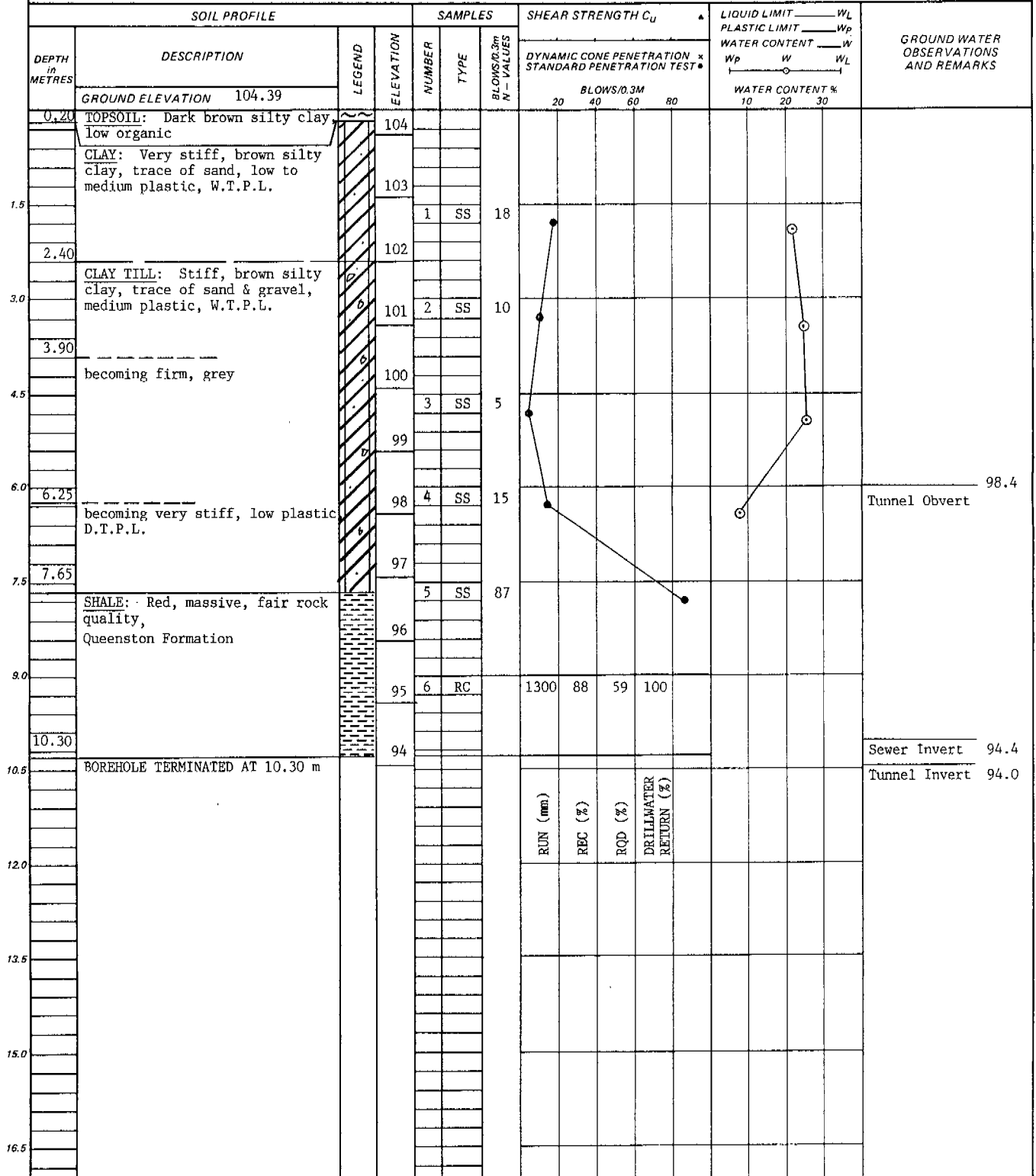
BORING METHOD Continuous Flight Solid Stem Augers

OUR PROJECT NO. 91HF007

BORING DATE March 21, 1991

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: T.J.G.

**LOG OF BOREHOLE NO. 5**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

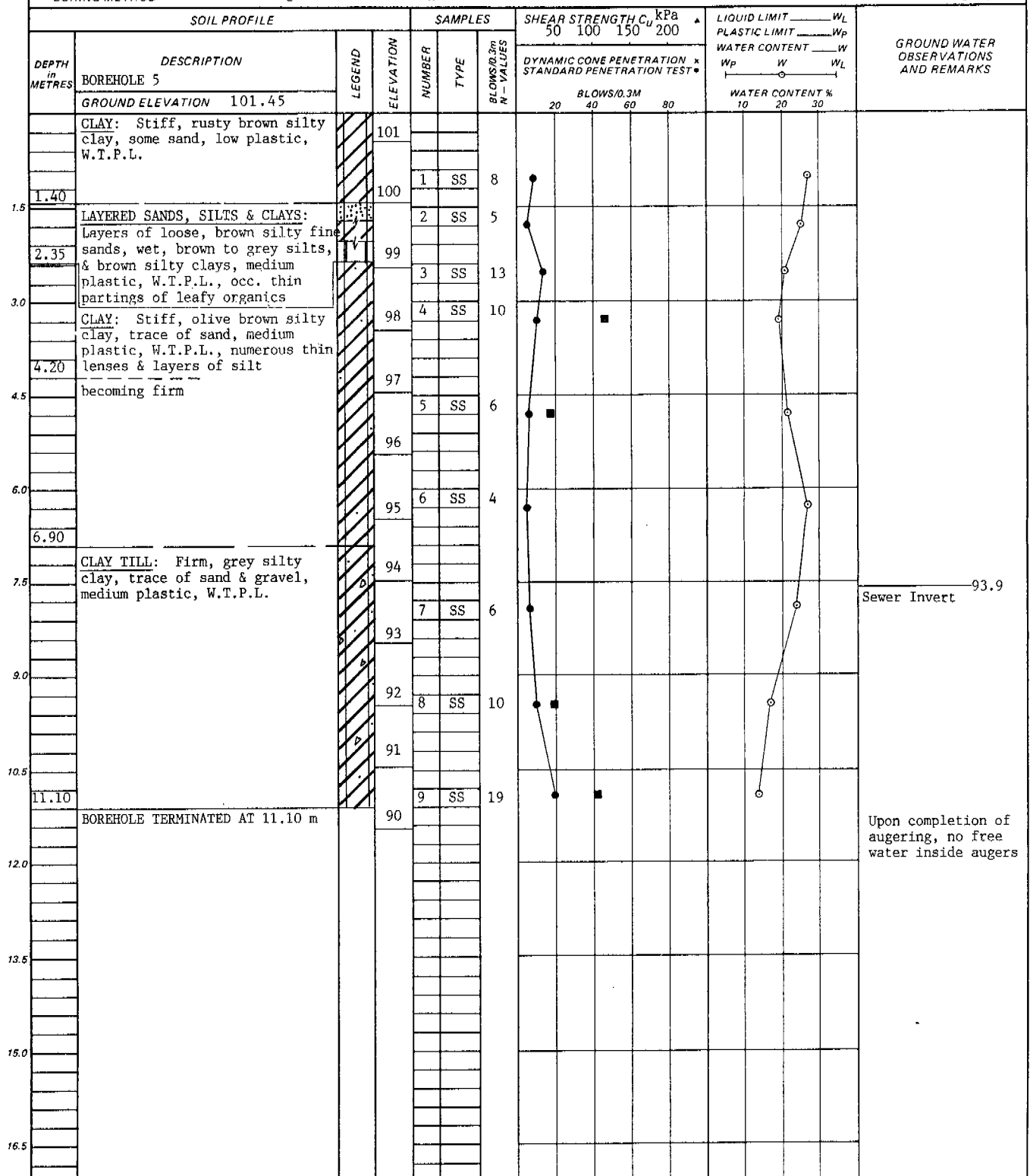
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE January 31/91

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: T.J.

**LOG OF BOREHOLE NO. 6**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

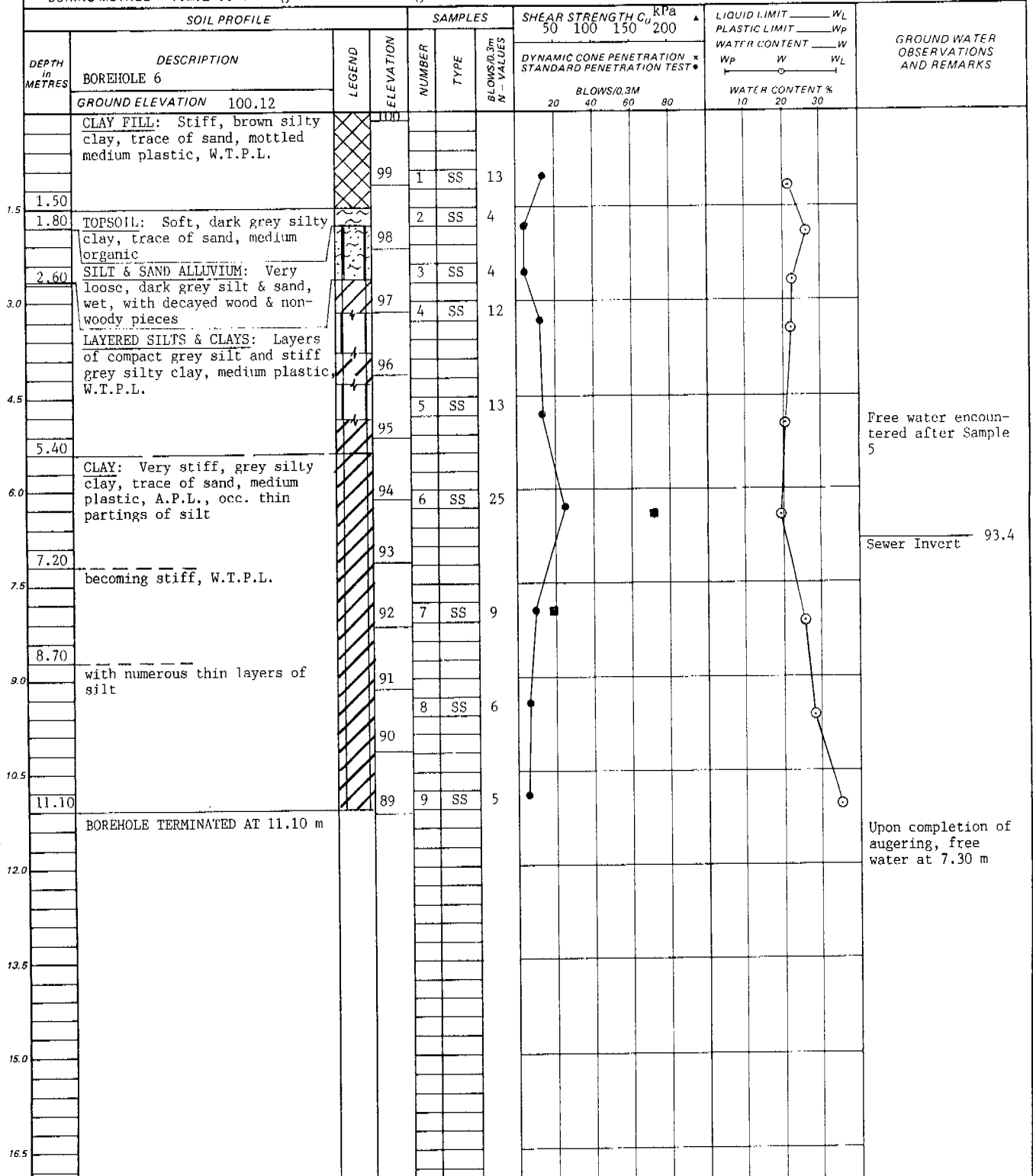
BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE January 30/91

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsy



NOTES:

CHECKED BY: *[Signature]*

## LOG OF BOREHOLE NO. 7

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

**LOCATION** Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE January 30/91

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

BORING METHOD Continuous Flight Solid Stem Augers

TECHNICIAN M. Rapsey

[illegible]

**NOTES:**

CHECKED BY: T.A.

**LOG OF BOREHOLE NO. 8**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

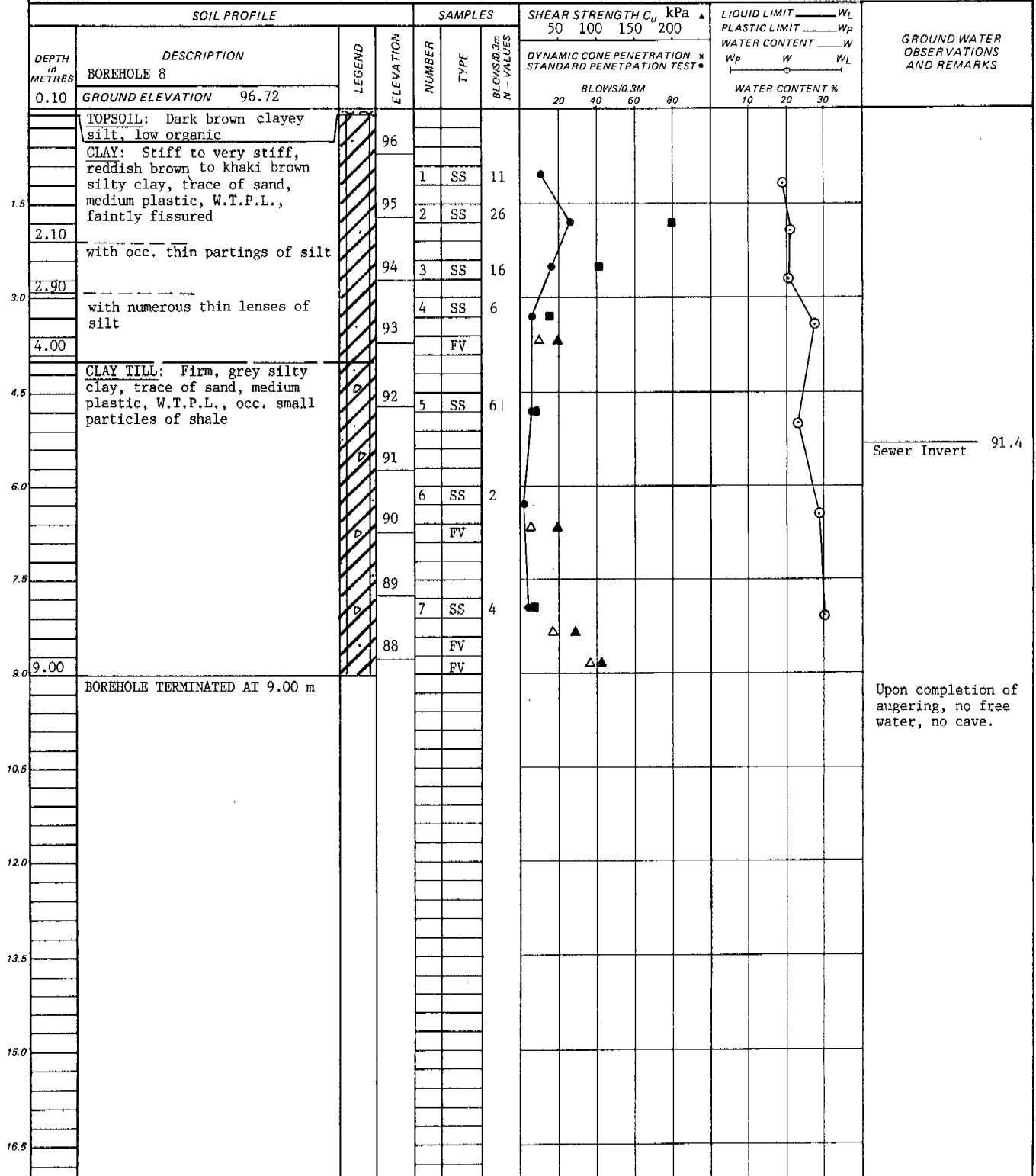
BORING METHOD Continuous Flight Solid Stem Augers

OUR PROJECT NO. 91HF007

BORING DATE January 30/91

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: *TJG*



# PetoMacCallum Ltd.

CONSULTING ENGINEERS

## LOG OF BOREHOLE NO. 9

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE Jan. 30/91

OUR PROJECT NO. 91HF007

ENGINEER T.J. Garde

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		SHEAR STRENGTH $C_u$		LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_p$ WATER CONTENT $W$ $W_p$ — $W$ — $W_L$ WATER CONTENT % 10 20 30			GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N-VALUES	DYNAMIC CONE PENETRATION * STANDARD PENETRATION TEST *				
							BLOWS/0.3M 20 40 60 80				
	BOREHOLE 9										
	GROUND ELEVATION 96.50										
	CLAY FILL: Stiff, brown silty clay, trace of sand, mottled dark brown, medium plastic, W.T.P.L.		96								
1.40			95	1	SS	14					
1.80	SILT ALLUVIUM: Very loose, dark brown sandy clayey silt, small shell fragments		94	2	SS	4					
	LAYERED CLAYS & SILTS: Layers of soft, grey silty clays, medium plastic, W.T.P.L. & brown silts, wet, dilatant		93	3	SS	5					
3.90			92	4	SS	6					
4.5	CLAY TILL: Hard, olive brown silty clay, trace of sand, medium plastic, D.T.P.L.		91	5	SS	35					
5.40			90	6	SS	15					
6.0	becoming very stiff, W.T.P.L., with thin partings of silt		89	7	SS	6					
6.90			88								
7.5	becoming firm		87	8	SS	7					
9.60	BOREHOLE TERMINATED AT 9.60 m										
10.5											
12.0											
13.5											
15.0											
16.5											

Free water encountered after Sample 2

Sewer Invert 90.5

Upon completion of augering, cave at 2.44 m

NOTES:

CHECKED BY:

## LOG OF BOREHOLE NO. 10

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

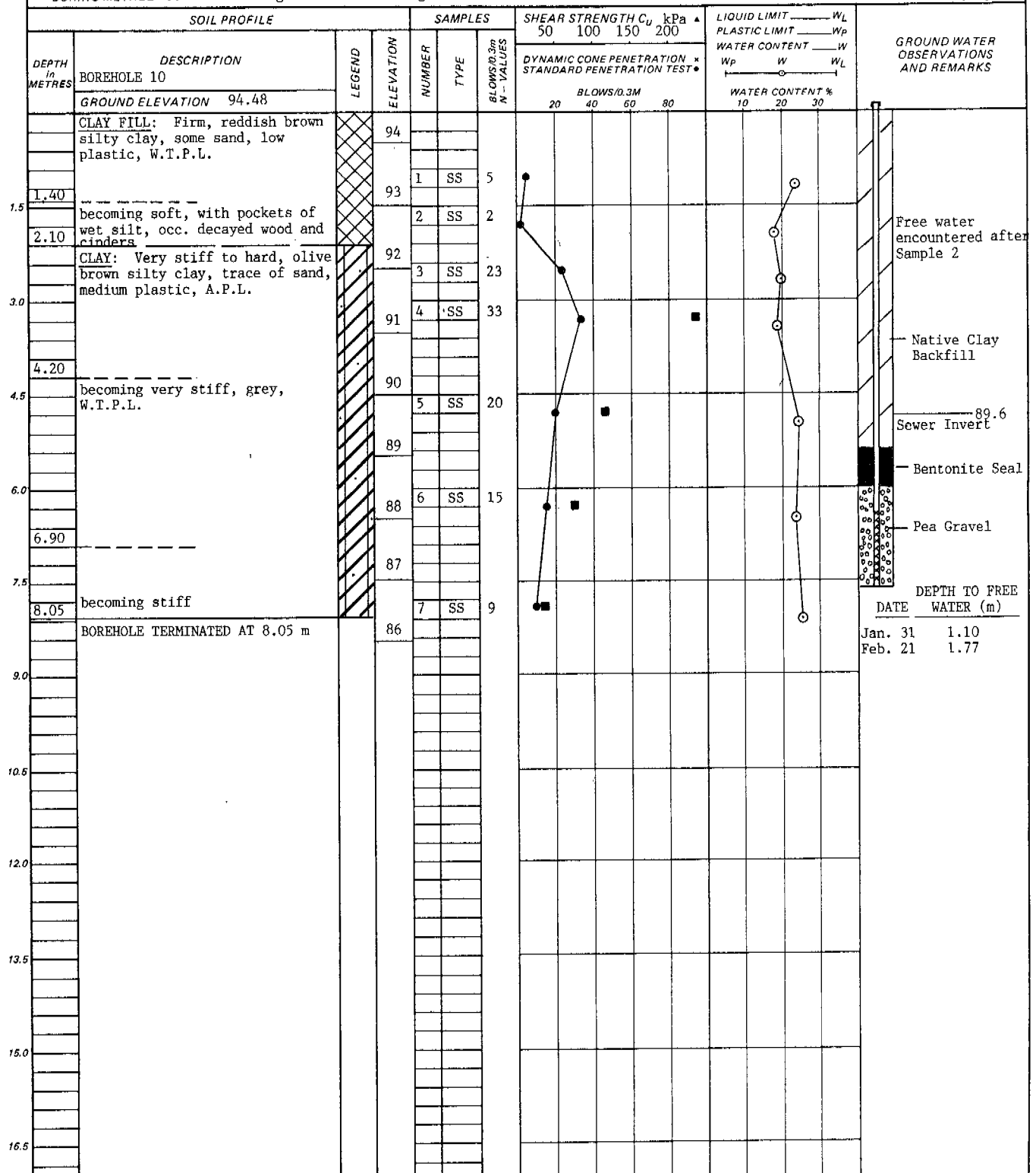
BORING METHOD Continuous Flight Hollow Stem Augers

BORING DATE January 31/91

OUR PROJECT NO. 91HF007

ENGINEER T.G. Garde

TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: *TG*

# PetoMacCallum Ltd.

CONSULTING ENGINEERS

## LOG OF BOREHOLE NO. 11

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

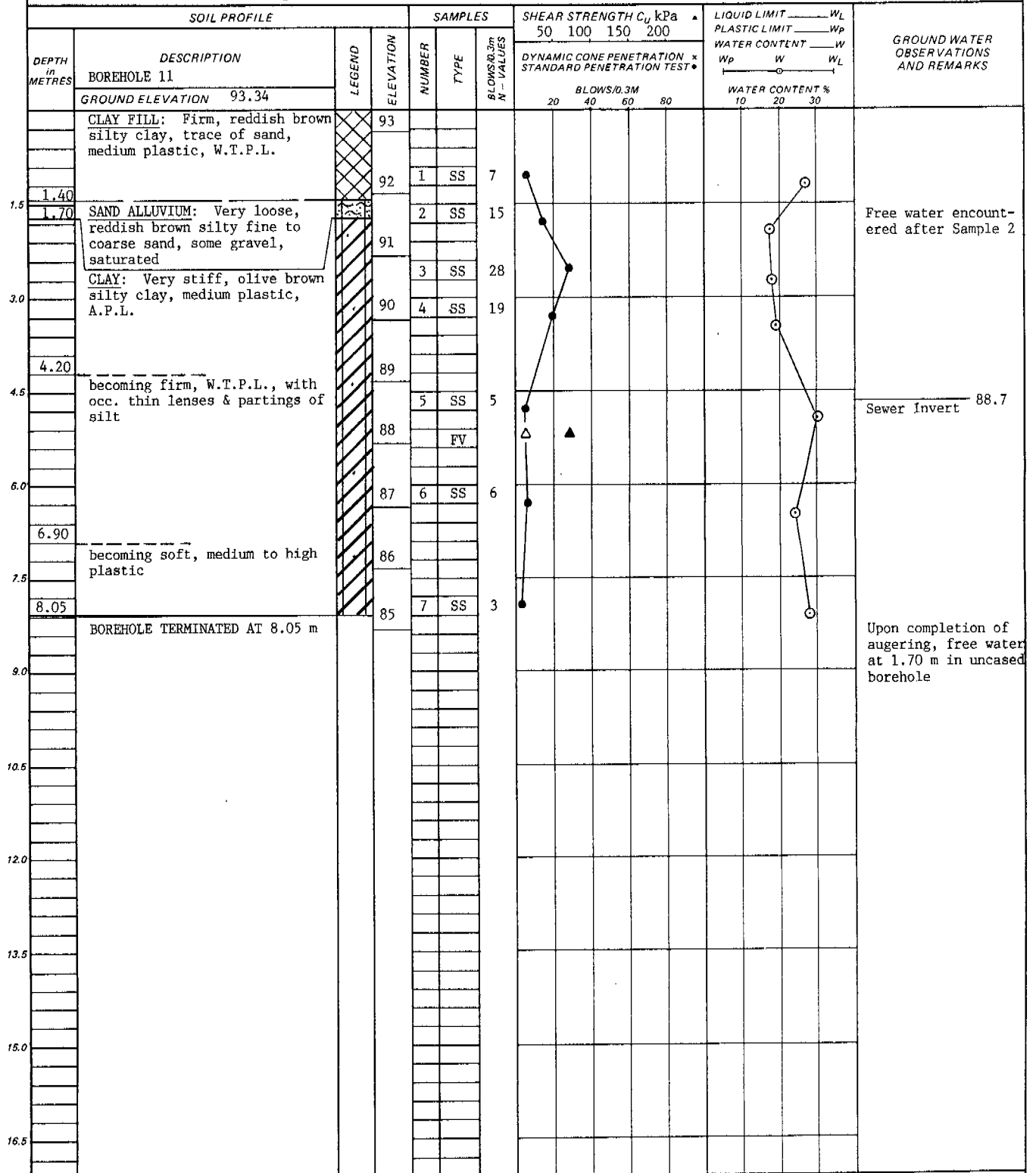
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT NO. 91HF007

BORING DATE January 31/91

ENGINEER T.G. Garde

TECHNICIAN M. Rapsey

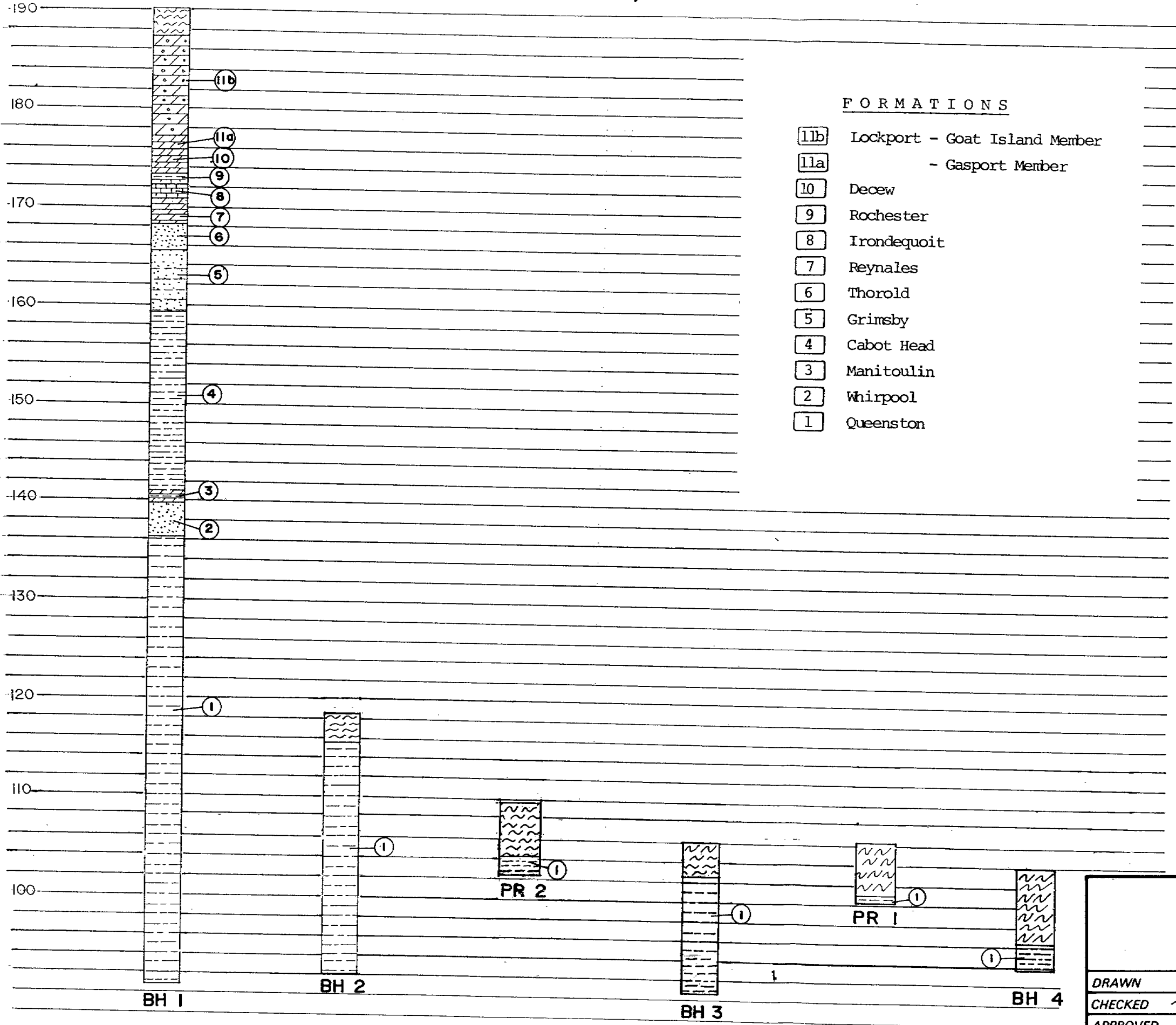


NOTES:

CHECKED BY: T.G.

SOUTH —→ NORTH


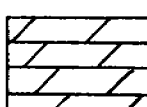
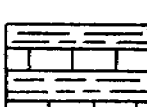
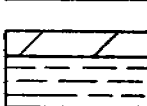
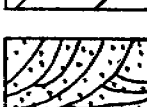
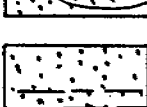
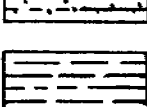
ELEVATION (m.)



FORMATIONS

- 11b Lockport - Goat Island Member
- 11a                      - Gasport Member
- 10 Decew
- 9 Rochester
- 8 Irondequoit
- 7 Reynales
- 6 Thorold
- 5 Grimsby
- 4 Cabot Head
- 3 Manitoulin
- 2 Whirpool
- 1 Queenston

LEGEND:

-  DOLOSTONE WITH CHERT NODULES
-  DOLOSTONE
-  SHALE AND LIMESTONE
-  DOLOSTONE AND SHALE
-  SANDSTONE WITH CROSS BEDDING
-  SANDSTONE AND SHALE
-  SHALE

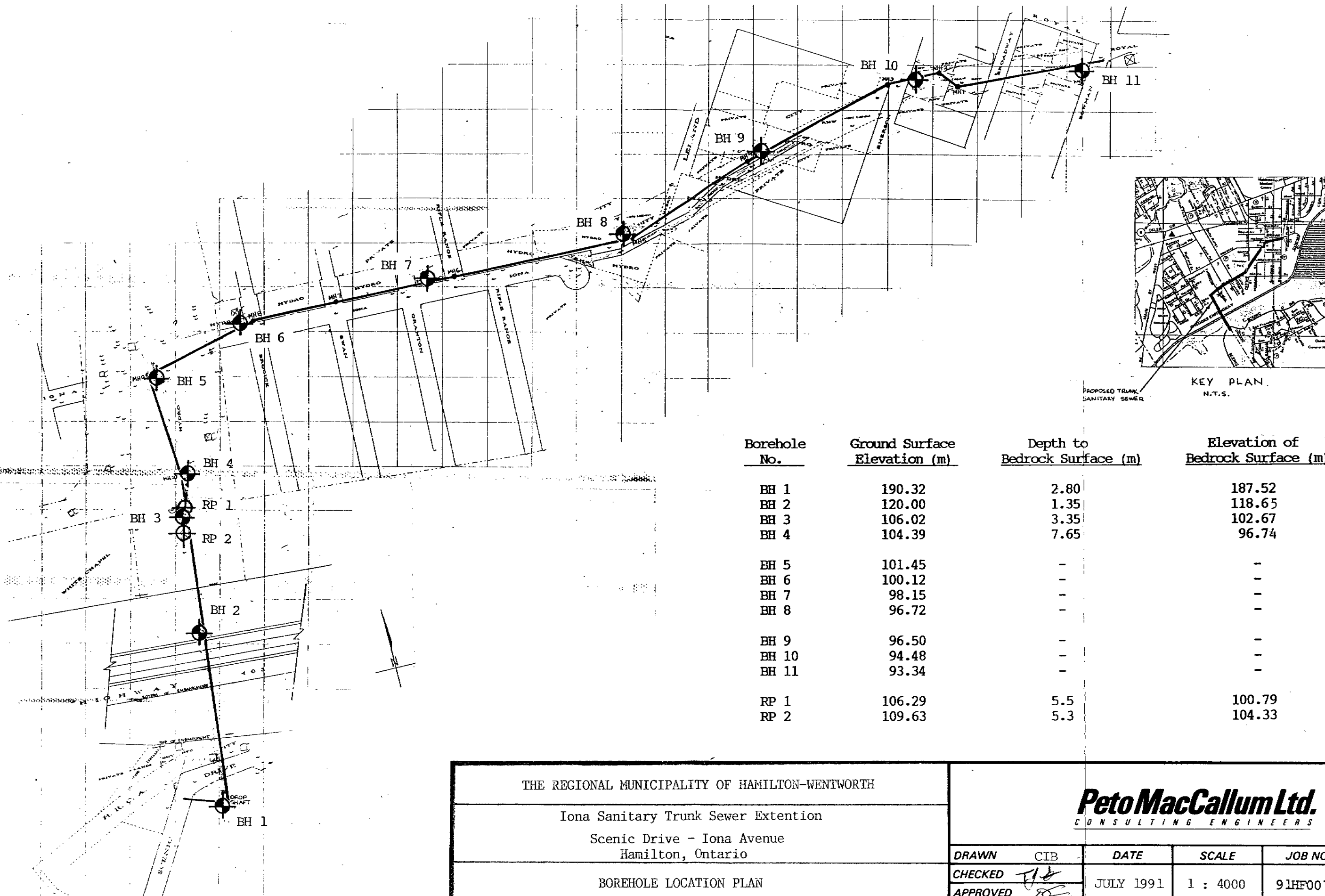
THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

Geotechnical Investigation for Iona Sanitary  
Trunk Sewer Extension  
Hamilton, Ontario

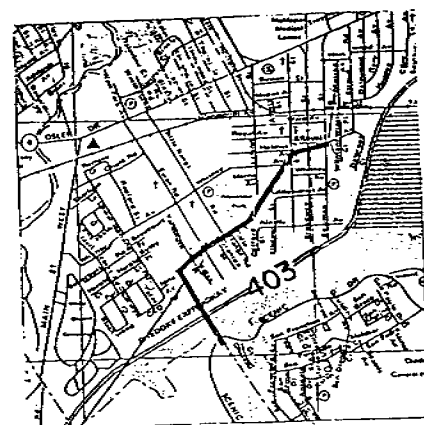
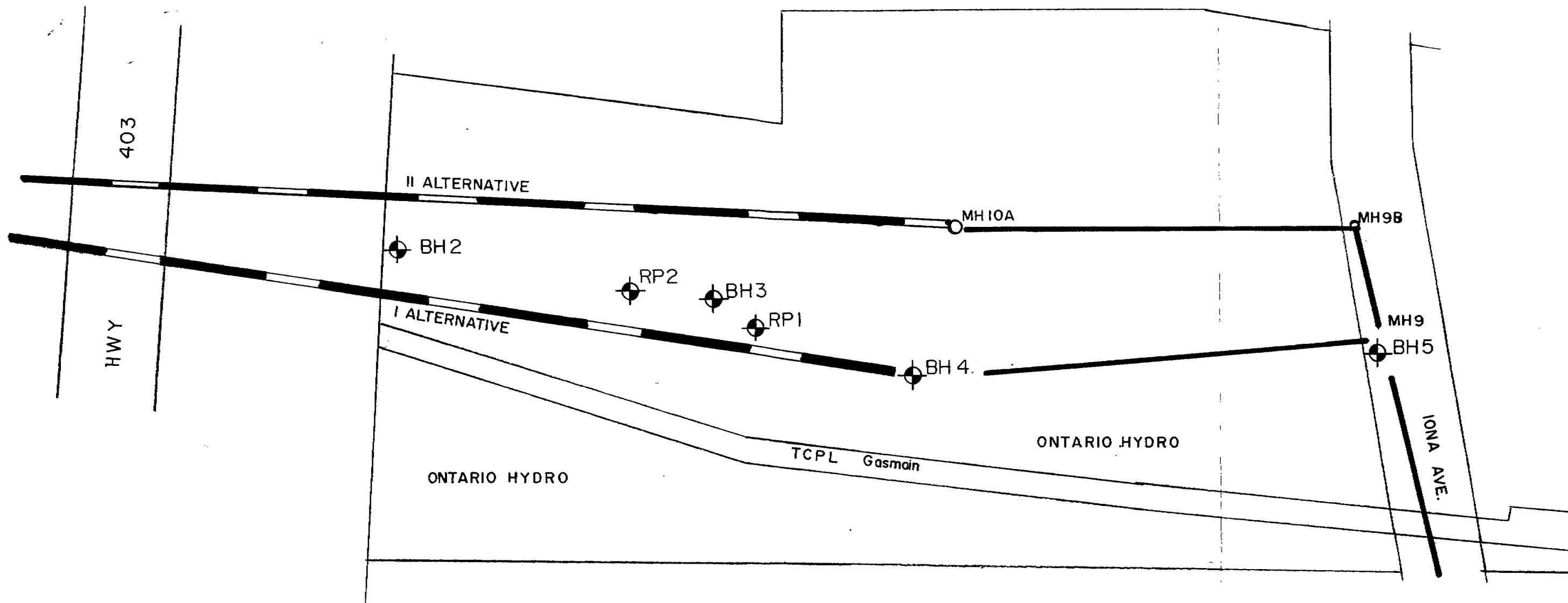
SOIL AND ROCK PROFILE

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS

DRAWN	CIB	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED	TJB	JULY 1991	N.T.S.	91HF007	2
APPROVED	RE				



THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH		<b>Peto MacCallum Ltd.</b> CONSULTING ENGINEERS					
Iona Sanitary Trunk Sewer Extention							
Scenic Drive - Iona Avenue Hamilton, Ontario		DRAWN	CIB	DATE	SCALE	JOB NO.	DRAWING NO.
BOREHOLE LOCATION PLAN		CHECKED	<i>[Signature]</i>	JULY 1991	1 : 4000	91HF007	1
		APPROVED	<i>[Signature]</i>				



KEY PLAN  
N.T.S.

Borehole No.	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Elevation of Bedrock Surface (m)
BH 2	120.00	1.35	118.65
BH 3	106.02	3.35	102.67
BH 4	104.39	7.65	96.74
BH 5	101.45	-	-
RP 1	106.29	5.5	100.79
RP 2	109.63	5.3	104.33

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH				
IONA SANITARY TRUNK SEWER EXTENSION HWY 403-IONA AVE. HAMILTON, ONTARIO				
BOREHOLE LOCATION PLAN				
<b>Peto MacCallum Ltd.</b> CONSULTING ENGINEERS				
DRAWN <i>TK</i>	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED <i>TG</i>	NOV '91	1:1000	91HF007	2
APPROVED <i>TG</i>				

SUPPLEMENTARY ROCK CORING  
IONA SANITARY TRUNK SEWER  
HAMILTON, ONTARIO  
FOR  
THE REGIONAL MUNICIPALITY OF  
HAMILTON-WENTWORTH

Letters dated:

June 23, 1992  
July 9, 1992  
August 20, 1992  
September 2, 1992  
September 16, 1992  
November 24, 1992

Distribution:

4 cc: Client  
1 cc: PML Hamilton  
1 cc: PML Toronto

Job No. 91HF007A

# *Peto MacCallum Ltd.*

C O N S U L T I N G   E N G I N E E R S

June 23, 1992

Our Ref: 91HF007A

Mr. H. Johnson, P.Eng.  
R.V. Anderson Associates Limited  
Suite 401, 1210 Sheppard Avenue East  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Supplementary Rock Coring  
Iona Sanitary Trunk Sewer  
Hamilton, Ontario

Further to your facsimile transmission dated June 17, 1992, we are pleased to provide our cost estimate for additional coring at the above noted project site.

We understand that the additional boreholes are required for further evaluation of the shale quality. It is noted that the shale was of very poor quality between elevations 91.6 to 93.2 at rock probe RP 2 and elevation 95.2 to 96.7 in borehole BH 2.

The estimated costs to locate and drill boreholes at the locations indicated in your fax, including engineering supervision, piezometer installation and reporting are as follows:

At RP 1 - approximately 5.5 m of overburden  
          augering and 10 m of rock coring.....\$ 4 350.00

Between BH 2 and RP 2 - approximately 6 m of  
          overburden augering and 20 m of  
          rock coring.....\$ 5 900.00

Monitoring and reporting of water levels can be carried out in conjunction with measurements in the existing piezometers.

...2



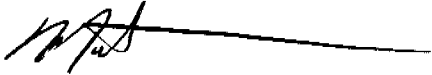
June 23, 1992, P2

91HF007A

Should you require any additional information, please do not hesitate to contact this office.

Sincerely

Peto MacCallum Ltd.



*12* Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

MRA:hb



# *Peto MacCallum Ltd.*

CONSULTING ENGINEERS

July 9, 1992

Our Ref: 91HF007A

The Regional Municipality of  
Hamilton-Wentworth  
c/o Mr. H. Johnson, P.Eng.  
1210 Sheppard Avenue East  
Suite 401  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Iona Sanitary Trunk Sewer  
Hamilton, Ontario

We have reviewed your April 6, 1992 letter concerning this project and are pleased to make the following comments.

Franklin Geotechnical Ltd. was retained by Peto MacCallum Ltd. to provide specialist advice concerning the rock mechanics aspects of the tunnel design. The February 13, 1992 letter from Franklin Geotechnical Ltd. (FGL) to R.V. Anderson Associates Limited (RVA) described the procedures that would be adopted during this study.

The magnitude of the rock pressure exerted on the secondary tunnel liner during convergence of the rock following excavation of the tunnel is, in general, dictated by:

- i) The in situ stresses in the rock mass.
- ii) The mechanical properties of the rock mass.
- iii) The type and properties of the tunnel support.
- iv) The method of excavation and construction of the tunnel.

The in situ stress in the rock mass and the mechanical properties of the rock were described in our report dated August, 1991.

...2

H. Johnson, July 9, 1992, P2

91HF007A

In order to assess the fully mobilized horizontal and vertical pressures acting on the secondary tunnel liner, it is necessary that the type of liner and excavation technique be known. Use of traditional liner design concepts are, in our opinion, not applicable to the highly stressed rocks that exist in Southern Ontario.

Assessment of the pressures acting on the liner at the design stage is an interactive process that must consider the four (4) factors noted above, particularly the interaction between the liner and the rock. Computer modelling techniques are available to evaluate the time dependent stresses that will be imposed on the secondary tunnel liner by the rock. However, the creep characteristics of Southern Ontario rocks are extremely variable; monitoring of the movements following excavation of a tunnel is an essential input for meaningful results to be provided by the computer model.

The following comments are intended to be used during preliminary design of the tunnel, selection of a suitable construction procedure and liner type. When this is complete, a more refined estimate of the rock pressures acting on the secondary tunnel liner can be provided by using computer modelling techniques.

The vertical component of stress in the rock mass before tunnelling increases with depth below ground surface. The vertical stress ( $\sigma_v$ ) can be computed from the following equation:

$$\sigma_v = \gamma_t h$$

where  $h$  = depth below ground surface

$$\begin{aligned}\gamma_t &= 20.5 \text{ kN/m}^3 \text{ in overburden} \\ &= 26.0 \text{ kN/m}^3 \text{ in rock}\end{aligned}$$

The actual vertical component of rock stress acting on the secondary tunnel liner will be much less than the in situ ground stress, due to arching in the rock mass above the tunnel, inward movements that take place due to the horizontal stresses that exist in the rock, as well as the primary liner (rock bolts, shotcrete). The transfer of stresses from the rock mass to the tunnel is a time dependent process. The mobilized vertical component of stress will depend on the effectiveness of the primary liner as well as the time lapse between excavation of the tunnel and installation of the liner

...3

H. Johnson, July 9, 1992, P3

91HF007A

For preliminary design, a value of zero should be used for the vertical component of stress in the rock adjacent to the tunnel before placement of the secondary liner.

The horizontal component of stress could be as high as the strength of the rock. Therefore, a value of 15 MPa is recommended during preliminary design of the liner.

Deformation of the bedrock and convergence of the tunnel is a time dependent process governed by the creep characteristics of the rock, the time delays between excavation and installation of the liner as well as the rigidity of the liner.

Based on our experience with previous tunnels in Southern Ontario, we expect the horizontal convergence to be in the order of 20 to 60 mm during the first 30 days after excavation. Ultimate closure over the next year or two could be twice this value.

Since the properties of the rock, particularly the time dependent movements and consequent stresses imposed on the secondary liner are quite variable, the secondary liner is normally designed to withstand the water pressures and the thrust of the TBM if this construction technique is employed. The recommended construction procedure is to install the primary liner immediately after excavation and monitor convergence in the tunnel as excavation proceeds. The final liner should be installed when excavation is complete and monitoring indicates the rate of convergence has reached an acceptable level. Convergence monitoring can also be used to evaluate the displacements and stresses that will be imposed on the secondary liner.

In regards to the two shafts, a horizontal stress value of up to 10 MPa is recommended in the Queenston shale and up to 15 MPa in the upper dolostone sequences. Closure of these shafts is expected to be much less than the tunnel, less than 20 mm in the Queenston Shale and 10 mm in the dolostone sequences. Fifty percent of the closure should occur within 30 days following excavation.

Deformation measurements should be carried out during excavation of the tunnel and shaft to confirm the magnitude of predicted convergences, as well as the creep characteristics and stress levels in the rock.

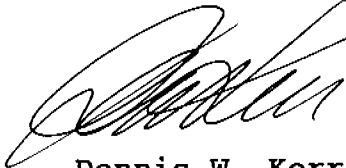
H. Johnson, July 9, 1992, P4

91HF007A

We trust the contents of this letter are sufficient to enable you to complete preliminary design. If further information is required, however, we strongly recommend that a meeting be called to deal with any further questions you may have.

Sincerely

Peto MacCallum Ltd.



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton



DWK:rp

- 1 cc: Region of Hamilton-Wentworth; J. Koshurba
- 1 cc: R.V. Anderson Associates; H. Johnson
- 1 cc: Franklin Geotechnical Ltd.

# ***PetoMacCallumLtd.***

*C O N S U L T I N G   E N G I N E E R S*

September 2, 1992

Our Ref: 91HF007A

The Regional Municipality of  
Hamilton-Wentworth  
c/o Mr. H. Johnson, P.Eng.  
R.V. Anderson Associates  
1210 Sheppard Avenue East  
Suite 401  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Groundwater Impact Summary  
Iona Sanitary Trunk Sewer  
Hamilton, Ontario

This letter summarizes our comments regarding the anticipated impact of the Iona Sewer project on groundwater conditions in the Niagara Escarpment.

A detailed discussion of the geology and groundwater conditions at the site, our assessment of the impact of the project on groundwater, as well as recommendations to minimize the impact are provided in our engineering report dated August 6, 1991. A nested piezometer was installed at the top of the escarpment on July 16, 1992, and supplementary testholes were drilled along the tunnel portion of the sewer north of Hwy 403 during the period June 9 to July 21, 1992. This information was summarized in our letter dated August 20, 1992.

A site plan showing the sewer alignment, borehole locations and piezometer/water level information is provided on Drawing 2.

We conclude that the proposed sewer drop shaft and tunnel will have a negligible long term impact on existing groundwater levels. Construction period impacts are expected to be minor/localized. A summary of our assessment is provided below.

## Proposed Sewer Layout

The proposed sewer installation in the escarpment vicinity will consist of a drop shaft extending some 94 m from the top of the escarpment and a tunnel section proceeding northerly under the escarpment face, the Highway 403 corridor and the slope below the highway.

...2

H. Johnson, September 2, 1992, P2

91HF007A

The drop shaft will extend through some 3 m of overburden clay till and then bedrock consisting of approximately 28 m of dolostone, limestone and sandstone, overlying two major shale formations. The lateral tunnel will be installed entirely within the lower shale unit, the Queenston formation.

#### Existing Groundwater Conditions

Groundwater levels measured in the two lower piezometers of the piezometer nest installed at the drop shaft location were about 23 m below the ground surface (elevation 167). The water level in the upper piezometer (zone of measurement 5 to 30 m) was 26.5 m below grade, near elevation 164. These measurements indicate a small upward hydraulic gradient in the upper rock formation and hydrostatic conditions below a depth of 30 m.

The Hwy 403 corridor is near elevation 142 at the proposed sewer crossing. It is probable that some seepage from the groundwater table occurs from the escarpment face above the highway. Seepage at higher levels on the escarpment face often noted during the winter/spring season probably represents surface water infiltrating through discontinuities and fractures in the upper dolomite/limestone formations.

Groundwater levels measured in three piezometers installed below Highway 403 ranged between 4 to 9 m below the ground surface. Seepage was not observed along this section of the slope during the geotechnical investigations although some groundwater discharge may occur into the small creek that exists at the base of the slope north of the proposed tunnel. The clay till overburden and Queenston shale bedrock in this section are considered to be relatively impermeable.

#### Anticipated Impact of Construction on the Groundwater Table

The rock units below the measured water table consist primarily of relatively impermeable shales. Minor sandstone units and a thin dolostone formation also exist. Although rock core samples showed no significant water bearing fractures or joints, some seepage into the drop shaft should be expected, with a potential for temporary lowering of the local water table. Some seepage from the bedding planes and fractures in the dolostone/limestone sequence above the measured water level may also occur. Water entering the shaft above Hwy 403 could result in a minor reduction in the normal discharge on to the escarpment face.

H. Johnson, September 2, 1992, P3

91HF007A

The possibility of encountering local water bearing fractures/joints in the rock and the potential for seepage from the upper dolostone/limestone sequences were identified in our geotechnical report. Recommendations were made to locally grout/seal off water bearing zones during construction and to permanently grout the annular space between the vertical drop shaft and surrounding rock.

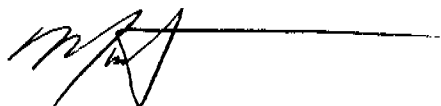
The lateral tunnel section will extend through relatively impermeable Queenston shale and hence any seepage during construction is not expected to affect groundwater levels. An impermeable barrier placed around the tunnel section would control long term seepage.

Provided these measures are implemented, installation of the trunk sewer is expected to have minimal impact on the groundwater table both in the short term during construction as well as long term following completion of construction.

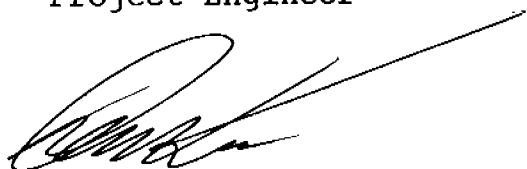
Should you require any additional information, please do not hesitate to contact this office.

Sincerely

Peto MacCallum Ltd.



Murray R. Anderson, P.Eng.  
Project Engineer



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton



MRA:rz

4 cc: R.V. Anderson Associates; H. Johnson  
1 cc: Region of Hamilton-Wentworth; J. Koshurba



# **PetoMacCallumLtd.**

CONSULTING ENGINEERS

September 16, 1992

Our Ref: 91HF007A

The Regional Municipality of  
Hamilton-Wentworth  
c/o Mr. H. Johnson, P.Eng.  
R.V. Anderson Associates  
1210 Sheppard Avenue East  
Suite 401  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Supplementary Comments  
Groundwater Impact Mitigation  
Iona Sanitary Trunk Sewer  
Hamilton, Ontario

This letter provides our supplementary recommendations to minimize the long-term impact of this project on existing groundwater conditions.

A detailed discussion of the geology and groundwater conditions at the site, our assessment of the impact of the project on groundwater, as well as recommendations to minimize the impact were provided in our engineering report dated August 6, 1991.

A nested piezometer was installed at the proposed drop shaft location on July 16, 1992, and supplementary testholes were drilled/piezometers installed along the tunnel portion of the sewer north of Highway 403 during the period June 9 to July 21, 1992. Factual data from this work was provided in a letter dated August 20, 1992 and included in a summary letter dated September 2, 1992.

In view of the subsequent groundwater level measurements and the site geology, we believe that measures to minimize the long-term impact of the shaft/tunnel should include the following:

- i) The annular space between the vertical drop shaft and surrounding rock should be permanently grouted along the entire length of the shaft in rock.
- ii) An impermeable barrier should be placed around the tunnel section in the Queenston shale to prevent long-term seepage between the shale and tunnel wall.

...2

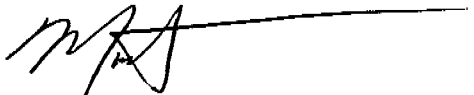
H. Johnson, September 16, 1992, P2

91HF007A

Should you require any additional information, please do not hesitate to contact this office.

Sincerely

Peto MacCallum Ltd.



Murray R. Anderson, P.Eng.  
Project Manager



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

MRA:rz

4 cc: Region of Hamilton-Wentworth; J. Koshurba  
1 cc: R.V. Anderson Associates; H. Johnson



# ***PetoMacCallumLtd.***

C O N S U L T I N G   E N G I N E E R S

November 24, 1992

Our Ref: 91HF007A

Mr. Stephen Szigeti, P.Eng.  
The Regional Municipality of  
Hamilton-Wentworth  
Roads Department  
71 Main Street West  
Hamilton, Ontario  
L8N 3T4

Dear Mr. Szigeti

Iona Trunk Sanitary Sewer  
Hamilton, Ontario

We have reviewed the November 4, 1992 letter from R.V. Anderson Associates as requested.

We note that design of the liner on an empirical basis is planned. The design/construction procedure calls for:

- . Excavation of the tunnel section and application of shotcrete to the surface of the rock.
- . Monitoring of time dependent movements.
- . Application of a 50 to 100 mm thick layer of compressible polyurethane spray foam material about 30 days after excavation. The time delay and foam thickness will be subject to the results of the closure monitoring.
- . Construction of a secondary concrete liner designed to withstand the groundwater pressures.

Adoption of an empirical approach to design of the tunnel liner system is consistent with that recommended in our August 1991 report and subsequent correspondence. The computer modelling referred to in our July 9, 1992 letter was to be based on closure measurements in the tunnel after excavation. Computer modelling based on actual closure measurements was considered to be the only practical means of providing the design data requested by R.V. Anderson Associates Limited.

...2

S. Szigeti, November 24, 1992, P2

91HF007A

In regards to the empirical design concepts outlined in the November 4, 1992 letter referred to previously, we make the following comments:

- i) The most practical means of designing tunnels in Southern Ontario rock is to use an empirical approach.
- ii) Placement of a compressible material between the rock and concrete has been successfully employed on several tunnels excavated in Southern Ontario rocks.

Problems with excessive movement have been experienced with this technique however if the design calls for a change from tunnelling to a cut and cover procedure near the end of the tunnel.

Greater movements near the north limit of this tunnel should be anticipated if a cut and cover technique is adopted. The construction methodology specified by the Region should be employed if sections of the tunnel in rock are constructed using cut and cover techniques.

- iii) Convergence of the rock is unlikely to be complete 30 days after completion of excavation. Closure of the tunnel should be monitored to permit assessment of the rate and magnitude of movements likely to occur after construction of the liner, the thickness of compressible material required to accommodate these movements, the consequent stress imposed on the liner and the optimum time to construct the secondary liner.

Concreting should be delayed until excavation of the tunnel is complete and start at the location where excavation commenced to permit the maximum time lapse between excavation and tunnelling.

S. Szigeti, November 24, 1992, P3

91HF007A

- iv) Preliminary design of the secondary liner should be based on the groundwater pressures. Final design should be based on the water pressures as well as the stresses likely to be imposed on the liner deduced from the convergence measurements coupled with visual examination of the primary liner.
- v) Closure of the tunnel could continue to be monitored after concreting to confirm that the design assumptions were appropriate.
- vi) If grouting between the liner and rock is planned, it should be limited to filling major voids that exist between the liner and the rock that may result in local stress concentrations; a low water cement ratio should be specified.

Reference is made to Section 6.2.1 in our August 1991 report for further comments to be considered during construction.

Should you require any additional information, please do not hesitate to contact this office.

Sincerely

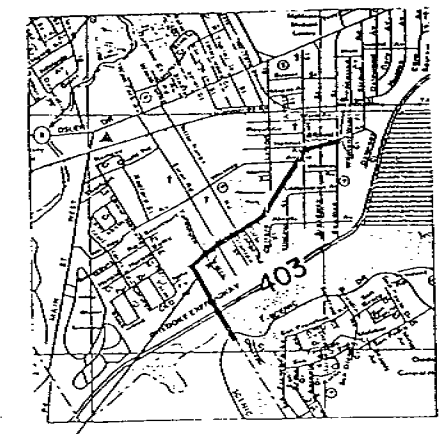
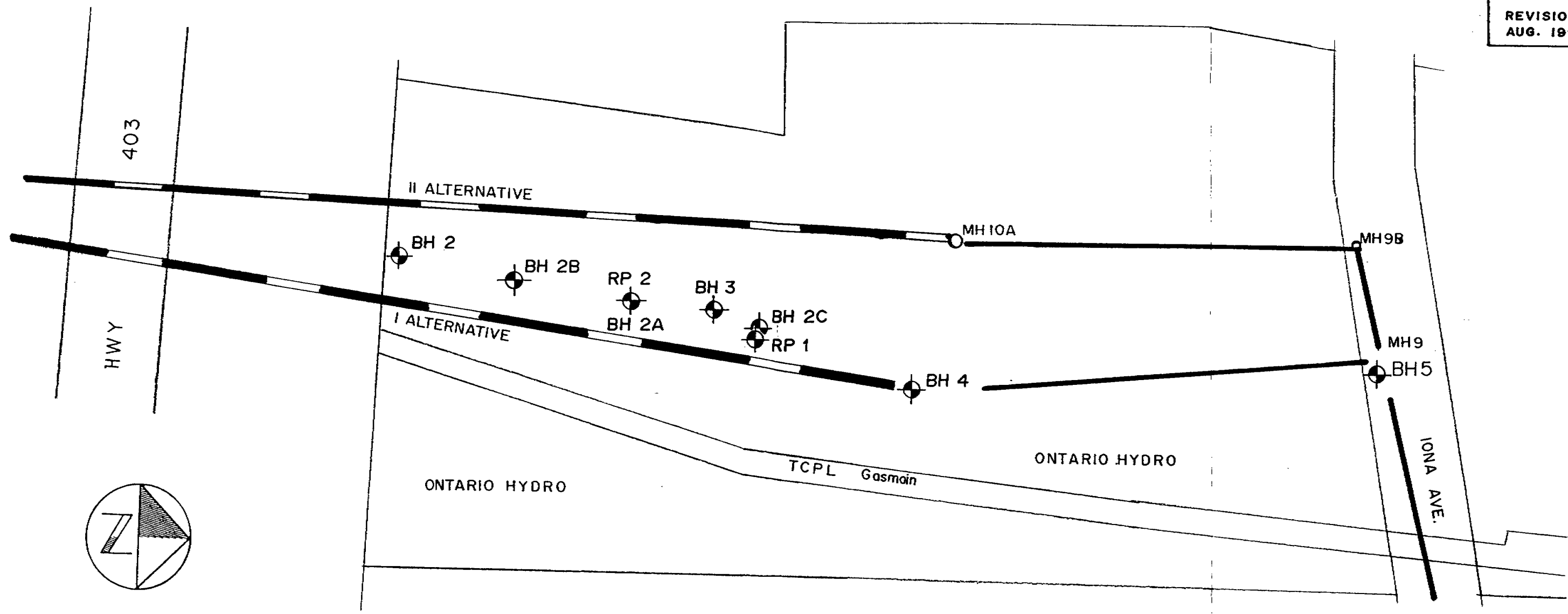
Peto MacCallum Ltd.



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

DWK:rz

4 cc: Region of Hamilton-Wentworth  
1 cc: R.V. Anderson Associates; H. Johnson



PROPOSED TRUNK  
SANITARY SEWER  
KEY PLAN  
N.T.S.

Borehole No.	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
2	120.00	1.35	118.65
2A (RP2)	113.34	7.30	106.04
2B	115.21	2.15	113.06
2C (RP1)	105.82 (106.29)	6.90	98.92
3	106.02	3.35	102.67

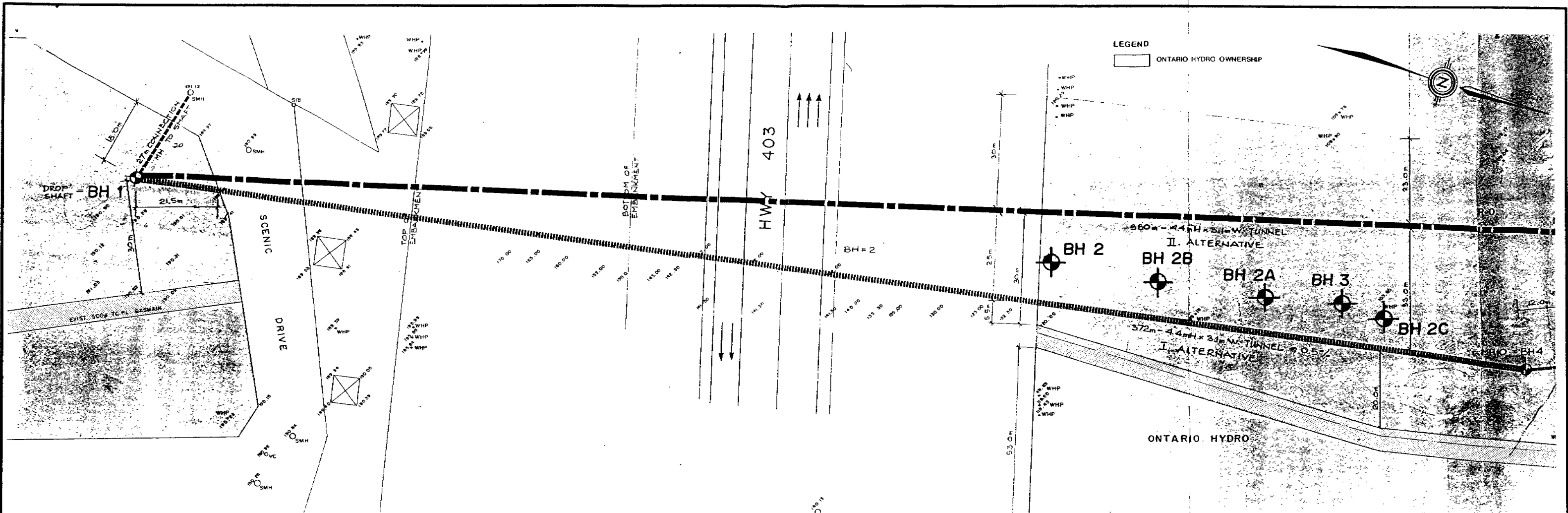
THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

IONA SANITARY TRUNK SEWER EXTENSION  
HWY 403-IONA AVE.  
HAMILTON , ONTARIO

BOREHOLE LOCATION PLAN

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS

DRAWN <i>TK</i>	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED <i>TG</i>	NOV '91	1:1000	91HF007A	1
APPROVED <i>TG</i>				



Borehole No.	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
1	190.32	2.80	187.52
2	120.00	1.35	118.65
2A	113.34	7.30	106.04
2B	115.21	2.15	113.06
2C	105.82	6.90	98.92
3	106.02	3.35	102.67
4	104.39	7.65	96.74

Piezometer No.	Piezometer Screen Depth (m)	Piezometer Details Bentonite Seal Position (m)	Measured Water Level August 7, 1992	
			Depth (m)	Elevation
1a	97.5 - 100.5	64.0 - 67.0	23.3	167.0
1b	58.0 - 61.0	34.6 - 40.7	23.2	167.1
1c	27.5 - 30.5	0.0 - 4.5	26.5	163.8
2A	18.0 - 19.5	6.7 - 7.3	9.1	104.2
2B	19.7 - 22.7	12.3 - 13.6	6.2	109.0
2C	14.3 - 15.8	7.3 - 8.5	4.1	101.7

THE REGIONAL MUNICIPALITY OF HAMILTON-WENTWORTH

IONA SANITARY TRUNK SEWER EXTENSION  
HIGHWAY 403 - IONA AVENUE  
HAMILTON, ONTARIO

BOREHOLE LOCATION PLAN

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS

DRAWN	CIB	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED	<i>[Signature]</i>	AUG. 1992	1:1000	91HF007A	2
APPROVED	<i>[Signature]</i>				

# Peto MacCallum Ltd.

CONSULTING ENGINEERS

## LOG OF BOREHOLE NO. 2B (cont'd)

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

OUR PROJECT NO 91HF007A

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE July 13/92

ENGINEER M.R. Anderson

BORING METHOD Continuous Flight Solid Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE		SAMPLES		RUN(mm)	RECOVERY(%)	R.Q.D.(%)	DRILL WATER RETURN(%)	MOISTURE CONTENT(%)	DRY UNIT WEIGHT(g/cm <sup>3</sup> )		GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION								
	GROUND ELEVATION 115.21										
			100								
			99								
16.5			98	10	RC						
			97	11	RC NQ	760	87	80	100		
18.0				12	RC NQ	450	56	33	100		
18.40	becoming poor quality, 'muddy' zone between 18.40 to 18.60 m		96								
			95	13	RC NQ	1525	100	30	100		
19.5											
20.40	becoming good quality, occasional gypsum seams		94	14	RC NQ	1525	100	80	100		
21.0			93	15	RC NQ	760	100	80	100		
22.5											
22.70	Borehole terminated at 22.70 m		92								
23.0											
24.5											
26.0											
27.5											
29.0											
30.5											

NOTES:

CHECKED BY *[Signature]*



**LOG OF BOREHOLE NO. 2C**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario






BORING DATE July 17/92

OUR PROJECT NO 91HF007A

ENGINEER M.R. Anderson

BORING METHOD Continuous Flight Solid Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE							
	GROUND ELEVATION 105.82											
1.5	Borehole advanced without sampling to bedrock surface		105									Native backfill
			104									
			103									
3.0			102									
			101									
4.5	met refusal and attempted to core; shale fragment in clay till		100									
			99									
6.0												
6.90	SHALE: Successive zones of excellent to poor quality, red shale, occasional grey layering											Bentonite Seal Tunnel Obvert Poor quality zone Tunnel Invert Filter sand
7.5			98	1	RC NQ	1525	100	100	100			
			97									
9.0			96	2	RC NQ	1465	100	23	100			
			95									
10.5			94	3	RC NQ	1525	100	48	100			
			93									
12.0			92	4	RC NQ	1525	100	73	100			
			91									
13.5			90	5	RC NQ	1525	100	33	100			
14.00	75 mm thick 'muddy' zone											
15.0	Borehole terminated at 15.80 m											
15.80												
16.5												

NOTES:

CHECKED BY *[Signature]*

## LIST OF ABBREVIATIONS

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N', - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 0.3m INTO THE SUBSOIL, DRIVEN BY MEANS OF A 63.5kg HAMMER FALLING FREELY A DISTANCE OF 0.76m.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 0.3m INTO THE SUBSOIL, THE DRIVING ENERGY BEING 475 J PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3 m</u>	<u>ckPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3 m</u>
VERY SOFT	0 - 2	0 - 12	VERY LOOSE	0 - 4
SOFT	2 - 4	12 - 25	LOOSE	4 - 10
FIRM	4 - 8	25 - 50	COMPACT	10 - 30
STIFF	8 - 15	50 - 100	DENSE	30 - 50
VERY STIFF	15 - 30	100 - 200	VERY DENSE	> 50
HARD	> 30	> 200		
W.T.P.L. WETTER THAN PLASTIC LIMIT		D.T.P.L. DRIER THAN PLASTIC LIMIT		
A.P.L. ABOUT PLASTIC LIMIT				

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W	THINWALL OPEN
W.S	WASHED SAMPLE	T.P	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

### SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL		

Δ, Δ - Undisturbed and remoulded undrained shear strength determined from insitu vane test.

■ - Undrained shear strength determined from pocket penetrometer test.

**LOG OF BOREHOLE NO. 2A**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

OUR PROJECT NO 91HF007A

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE June 8 & 9/92

ENGINEER M.R. Anderson

BORING METHOD Continuous Flight Solid Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE			SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )		GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE							
	GROUND ELEVATION 113.34											
	Borehole advanced without sampling to bedrock surface											
7.30	Commenced Rotary Drilling		106									Native Backfill
7.5	SHALE: Poor quality red shale, low strength, with grey layering		105	1	RC NQ	1525	100	28	100			Bentonite Seal
8.60	Queenston Formation becoming fair quality		104	2	RC NQ	1525	100	53	100			
9.0			103									
10.40	becoming good quality		102	3	RC NQ	1525	95	90	100			Filter Sand
10.5			101	4	RC NQ	1525	98	75	100			
12.0			100									
13.5			99	5	RC NQ	1525	100	93	100			
15.0			98									
16.50	becoming very poor quality		97	6	RC NQ	1525	100	80	100			Tunnel Obvert 97.7
16.5			96	7	RC NQ	1525	93	13	100			Very poor quality zone
18.0	becoming poor to fair quality		95									
18.0			94	8	RC NQ	1525	97	50	100			
19.50	Borehole terminated at 19.50 m		93									Tunnel Invert 94.4
21.0												
22.5												

NOTES:

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**LOG OF BOREHOLE NO. 2B**

PROJECT IONA SANITARY TRUNK SEWER EXTENSION

LOCATION Scenic Drive to Royal Avenue, Hamilton, Ontario

BORING DATE July 13/92

OUR PROJECT NO. 91HF007A

ENGINEER M.R. Anderson

BORING METHOD Continuous Flight Solid Stem Augers & NQ Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		RUN (mm)	RECOVERY (%)	R.Q.D. (%)	DRILL WATER RETURN (%)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (g/cm <sup>3</sup> )	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE							
	GROUND ELEVATION 115.21		115									
	Borehole advanced without sampling to bedrock surface		114									
1.5			113									
2.15	becoming shale		112									
3.00	Commenced Rotary Drilling		111	1	RC NQ	1525	100	60	100			
	SHALE: Fair quality red shale, low strength, occasional grey layers		110	2	RC NQ	1525	100	47	100			
4.50	Queenston Formation		109	3	RC NQ	1525	100	45	100			
	becoming poor quality, occasional vertical fractures		108	4	RC NQ	1525	100	38	100			
6.0			107	5	RC NQ	1525	100	80	100			
7.5			106	6	RC NQ	1525	95	82	100			
9.0	becoming good quality		105	7	RC NQ	1220	100	58	100			
10.5			104	8	RC NQ	1370	100	59	100			
12.0			103	9	RC NQ	1370	100	93	100			
12.90	becoming fair quality		102	10	RC NQ	1525	100	96	100			
14.80	becoming excellent quality		101									
16.0			100									
16.5			99									

Native Backfill

Bentonite Seal

Filter Sand

NOTES:

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# *PetoMacCallum Ltd.*

C O N S U L T I N G   E N G I N E E R S

August 20, 1992

Our Ref: 91HF007A

The Regional Municipality of  
Hamilton-Wentworth  
c/o Mr. H. Johnson, P.Eng.  
R.V. Anderson Associates  
1210 Sheppard Avenue East  
Suite 401  
Willowdale, Ontario  
M2K 1E3

Gentlemen

Supplementary Geotechnical Investigation  
Iona Sanitary Trunk Sewer  
Hamilton, Ontario

We are pleased to present the results of the supplementary geotechnical investigation completed in connection with the above noted project.

The supplementary work was carried out to further evaluate the nature/extent of the zone of very poor quality shale identified in borehole 3 during the original investigation, and to obtain groundwater level measurements in the tunnel/shaft area.

## Supplementary Coring

On June 8 and 9, 1992, a supplementary borehole designated borehole 2A was put down at the location of rock probe 2. This borehole was advanced unsampled to the bedrock surface and thin coring of the shale bedrock was carried out to total depth of 19.5 m.

After review of the core information from borehole 2A, authorization was received to drill two additional boreholes. Borehole 2B was put down approximately halfway between boreholes 2 and 2A, and borehole 2C was drilled near rock probe 1. Core samples of the shale were recovered between 3.0 to 22.7 m depth in borehole 2B, and between 6.9 to 15.8 m depth in borehole 2C.

The borehole locations are shown on Drawing 1, Revision 1 dated August 1992.

...2

H. Johnson, August 20, 1992, P2

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The ground elevations at boreholes 2A, 2B and 2C were established by Peto MacCallum Ltd. relative to the ground elevation at boreholes 2 and 3 previously defined by The Regional Municipality of Hamilton-Wentworth surveyors. The elevations were not determined relative to a fixed benchmark and are therefore considered to be approximate.

The elevation at borehole 2A (rock probe 2) previously established by The Regional Municipality of Hamilton-Wentworth was inconsistent with the elevations at boreholes 2 and 3. The elevation at borehole 2A has been adjusted accordingly.

The results of the coring operations including rock classifications and rock quality designation (RQD) are provided on the attached borehole logs.

The adjusted ground and bedrock surface elevations at the boreholes are summarized on Drawing 1.

We note that the bedrock depths previously defined by rock probes 1 and 2 may not be reliable. It is possible that these probes were terminated on shale fragments within the clay till overburden. A shale fragment was contacted at 4.6 m depth in borehole 2C; this was believed to be bedrock but coring proved otherwise.

The zone of poor to very poor quality shale encountered in borehole 3 was also encountered in boreholes 2A, 2B and 2C. The elevation of this zone was as follows:

<u>Borehole</u> <u>No.</u>	<u>Zone of Poor Quality Shale</u>	
	<u>Depth (m)</u>	<u>Elevation</u>
2	----	----
2A	16.5 - 18.0	96.8 - 95.3
2B	18.4 - 20.4	96.8 - 94.8
2C	8.7 - 10.6	97.1 - 95.2
3	9.3 - 10.8	96.7 - 95.2

H. Johnson, August 20, 1992, P3

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Groundwater Levels

Piezometers were installed in boreholes 2A, 2B and 2C for measurement of groundwater levels. A nested piezometer installation was constructed adjacent to borehole 1 at the drop shaft location. Details of the piezometer installations were as follows:

<u>Borehole No.</u>	<u>Date of Installation</u>	<u>Screen Depth (m)</u>	<u>Bentonite Seal Position (m)</u>
1a	July 16	97.5 - 100.5	64.0 - 67.0
1b	July 16	58.0 - 61.0	34.6 - 40.7
1c	July 16	27.5 - 30.5	0.0 - 4.5
2A	June 9	18.0 - 19.5	6.7 - 7.3
2B	July 15	19.7 - 22.7	12.3 - 13.6
2C	July 21	14.3 - 15.8	7.3 - 8.5

The water levels measured in the piezometers on August 7, 1992 are summarized below:

<u>Borehole No.</u>	<u>Measured Water Level</u>	
	<u>Depth (m)</u>	<u>Elevation</u>
1a	23.3	167.0
1b	23.2	167.1
1c	26.5	163.8
2A	9.1	104.2
2B	6.2	109.0
2C	4.1	101.7

H. Johnson, August 20, 1992, P4

91HF007A

Should you require any additional information, please do not hesitate to contact this office.

Sincerely

Peto MacCallum Ltd.



Murray R. Anderson, P.Eng.  
Project Manager



Dennis W. Kerr, P.Eng.  
Manager Geotechnical Services  
Hamilton

MRA:rz

4 cc: Region of Hamilton-Wentworth; J. Koshurba  
1 cc: R.V. Anderson Associated; H. Johnson



# memorandum



To: D. Billings  
Head  
Geotechnical Section  
Central Region

Date: 1994 03 07

Atten: R. Kohlberger

From: Foundation Design Office  
Room 315, Central Bldg.

Re: Proposed Tunnel Work under Hwy. #403  
Iona Sanitary Trunk Sewer  
City of Hamilton  
Region of Hamilton-Wentworth  
W.O. 94-11001

A trunk sewer is proposed by the Regional Municipality of Hamilton-Wentworth between Scenic Drive and Royal Avenue in the City of Hamilton. From Scenic Drive to Iona Avenue the construction of a tunnel will be required. At some point the tunnel will cross Hwy. #403 some 40 m below the profile grade. It is our understanding that the tunnel will be constructed using drill and blast method. In order to construct within Hwy. #403 right of way an encroachment permit application is submitted to M.T.O. by the Regional Municipality.

Concerning the blasting operations, the following conditions should be assured by the Regional Municipality.

- 1) Only controlled blasting is permitted
- 2) The maximum peak particle velocity measured in any of three mutually perpendicular direction should not exceed 50 mm/sec.
- 3) The contractors should be advised in the specifications that they will be required to retain the services of a professional blasting engineer to undertake the blasting design and vibration monitoring for the duration of the operations.
- 4) The contractor shall undertake trial blasting on his proposed blasting pattern to demonstrate that the proposed blast design satisfies the vibration limit.
- 5) Prior to any blasting operation, the contractor shall engage an engineering company experienced in pre-blast survey to carry out an inspection of all the 'works within the right-of-way.

- 6) The contractor should not store explosives at the site.
- (7) The contractor should display warning signs, advising the public of blasting operations.
- 8) The contractor should be responsible for safe operations.

We believe that the blasting operations carried out in accordance with the above described procedures will not jeopardize the integrity of Hwy. #403.

Therefore, from the Foundation Design Section's point of view the encroachment permit can be granted to the applicant.

To resolve other issues such as Environmental, Labour Act etc., the Regional Municipality should contact the proper jurisdiction.



P. Payer, P. Eng.  
Sr. Foundation Engineer

PP/mmj

c.c. - S. Szigeti  
P. Kuyntijes