82-26025-1

MORTON & PARTNERS LIMITED
CONSULTING ENGINEERS & ENGINEERING GEOLOGISTS

215 CARLINGVIEW DRIVE,
REXDALE, ONTARIO, CANADA M9W 5X8
TELEPHONE: (416) 675-1440

01/84.405

REPORT

to

GO-ALRT

on

SUBSURFACE CONDITIONS AND FOUNDATIONS

PROPOSED DORVAL DRIVE SUBWAY

OAKVILLE ONTARIO

by

MORTON & PARTNERS LIMITED
215 CARLINGVIEW DRIVE
REXDALE, ONTARIO, M9W 5X8

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memorandum

Ontario

To: Mr. G. C. E. Burkhardt
Head, Structural Section
Central Region
5000 Yonge Street, Willowdale

Date: 84 04 24

Attn: M. D. Bendayan

From: Foundation Design Section
Room 315, Central Building
Downsview

Re: Foundation Investigation Report
For
W.O. 82-26025
District 4, Hamilton
GO-ALRT, West Extension, Oakville Project
Dorval Drive Subway

This memo is a supplementary to the attached report prepared by consultant Morton & Partners Limited for the above-noted project. This section has reviewed the consultant's report and our comments are as follows:

The factual information (Sections 1 - 6 & Record of Borehole Sheets & Borehole Locations/Soil Strata Drawing) within the consultant's report is considered to be acceptable for the purpose of this project.

The foundation recommendations provided in this memo supersede those in the consultant's report (Section 7 & Table 1).

FOUNDATION RECOMMENDATIONS

Two bridge designs have been proposed to carry the GO-ALRT over Dorval Drive at a grade of 107 ± m.

DESIGN A - two spans at 25 ± m each
DESIGN B - two spans at 19 ± m each

General Recommendations (Applicable to Both Foundation Alternatives)

EARTH PRESSURE CALCULATIONS:

Backfill to structures should consist of granular material in accordance with MTC Standard Special Provision #121 (83 10). Computation of earth pressures should be in accordance with Section 6.6.1.2 of the O.H.B.D.C.

For design purposes, the physical properties of the backfill are as follows:

MATERIAL	ϕ	γ
GRANULAR 'A'	35°	22.0 kN/m^3
GRANULAR 'B'	30°	21.2 kN/m^3

At this site, the foundation is considered to be 'non-yielding' and the at-rest condition applies insofar as lateral earth pressures are concerned.

SETTLEMENT CONSIDERATIONS:

Differential settlements will be negligible.

SLOPE STABILITY:

No stability problems are anticipated for embankments or cuts with slopes of 2:1 or flatter. If steeper slopes are required, please contact this section for recommended slope angles.

FROST PROTECTION:

The minimum cover required for frost protection is 1.2 m.

DE-WATERING:

De-watering is not anticipated to be a major problem. It is expected that groundwater entering excavations can be controlled by conventional pumping techniques.

FOOTING EXCAVATIONS:

Excavations in bedrock may be accomplished without blasting techniques.

Temporary cut slopes in the overburden, and in the upper 1 m of weathered shale bedrock should not be steeper than 1.5 horizontal to 1 vertical. Temporary cut slopes in the weathered shale bedrock below 1 m depth, and in the sound shale bedrock should not be steeper than 1 horizontal to 3 vertical.

All soft or loose material at the proposed footing locations should be removed, and the excavation bottom should be covered, within 12 hours of exposure, with a 15 cm pad of mass concrete. Any existing trenches (e.g. underground utilities) encountered within the footing areas should be excavated to sound bedrock and backfilled within 12 hours of exposure, with mass concrete.

RESISTANCE TO LATERAL FORCES

For design purposes, the following friction coefficients may be assumed to apply between the essentially horizontal bedding planes of the bedrock, and between the base of the footing and the bedrock surface:

weathered shale	-	tan 22°
sound shale	-	tan 25°

To supplement the frictional resistance between the base of the footing and the bedrock surface, a key cut a minimum depth of 0.5 m into the sound bedrock may be assumed to provide a resisting pressure of 1 MPa against lateral forces, provided that the key is placed against the 'undisturbed' rock face.

Passive resistance in front of the footings should be discounted except for that portion below the frost penetration zone (1.2 m depth).

Design Details

Two foundation alternatives, applicable to both bridge design proposals, are recommended. The combination of alternatives which leads to the least expensive design should be adopted.

ALTERNATIVE 1 - SPREAD FOOTINGS ON SOUND BEDROCK

The entire structure (abutments and centre pier) may be supported on spread footings founded on sound bedrock at or below the elevations indicated in Table A.

The following design values are recommended:

Working Stress Design Method;
- net safe bearing pressure = 1000 kPa

O.H.B.D.C. Method;
- Factored Bearing Capacity at U.L.S. = 1500 kPa
- Bearing Capacity at S.L.S. Type II will not govern design.

ALTERNATIVE 2 - SPREAD FOOTINGS ON WEATHERED BEDROCK

The entire structure (abutments and pier) may be supported on spread footings founded on weathered bedrock at or below the elevations indicated in Table A.

The following design values are recommended:

Working Stress Design Method;
- net safe bearing pressure = 670 kPa

O.H.B.D.C. Method;
- Factored Bearing Capacity at U.L.S. = 1000 kPa
- Bearing Capacity at S.L.S. Type II will not govern design.

TABLE A

BRIDGE DESIGN PROPOSAL	FOOTING DESCRIPTION	FOOTING LOCATION	FOOTING ELEVATION	
			FOUNDATION ALTERNATIVE 1 (sound bedrock)	FOUNDATION ALTERNATIVE 2 (weathered bedrock)
A	EAST ABUT.	STA. 11 + 510 ±	99.5 m	100.5 m
B	EAST ABUT.	STA. 11 + 516 ±	98.5 m	99.0 m
A & B	CENTRE PIER	STA. 11 + 535 ±	98.0 m	99.0 m
B	WEST ABUT.	STA. 11 + 554 ±	98.5 m	99.0 m
A	WEST ABUT.	STA. 11 + 560 ±	98.5 m	99.5 m

If further information is required please contact this Section.

D. H. Dundas

D. H. Dundas, P.Eng.
Foundations Engineer

for K. G. Selby, P.Eng.
Chief Foundations Engineer
(West)

1.0 INTRODUCTION

Morton & Partners Limited has been retained by the GO-ALRT Programme to carry out an investigation and analysis of foundation conditions at the site of a proposed subway, at Dorval Drive, Oakville, Ontario. The GO-ALRT reference for the site is noted as PD 2-300. The site lies near the west boundary of The Town of Oakville, in the Regional Municipality of Halton.

The purpose of the investigation was to determine by detailed sampled boreholes the soil, bedrock and groundwater conditions beneath the intended foundations. Also, to analyse and advise on possible foundation designs.

The following report summarizes the results of this investigation. Sections 2 to 5 inclusive provide factual information. Section 6 is interpretive and contains recommendations for foundation treatment based on the preliminary design concepts provided in Oakville Project - West Extension Drawing P-004, Revision 0, which was issued November 17th, 1983, expressly for Foundation Investigation purposes.

The planned structure comprises a concrete bridge for the GO-ALRT tracks which is to overpass the presently depressed Dorval Drive roadway section that forms the north side approach to the existing CN Rail Underpass. Two alternative abutment designs have been detailed, with different span lengths. A central column/pier in the roadway median is common to both alternatives.

2.0 GENERAL SITE DESCRIPTION

The site is located within an Ontario Hydro Right-of-Way that parallels and borders on the north side of the CN Rail mainline Right-of-Way between Toronto and Hamilton. Dorval Drive is an already developed (paved) road that passes beneath the CN tracks in a reinforced concrete underpass. The new GO-ALRT subway bridge-overpass lies some 34 m north of the CN structure and will pass over the depressed north approach of the road at a point where

2.0 GENERAL SITE DESCRIPTION (Continued)

it is beginning to climb northwards towards the South Service Road intersection and Queen Elizabeth Highway (QEW). The South Service Road intersection is more or less at original ground level, while the crossing of the Queen Elizabeth Way is by reinforced concrete overpass.

The land to the north of the proposed GO-ALRT site, on either side of Dorval Drive has already been developed for commercial purposes.

Some considerable amount of general regrading, levelling and trench or construction backfilling has undoubtedly gone on in the immediate and general vicinity of the site, including levelling and filling for the CN tracks. A major depressional open-cut excavation has been created for the Dorval Drive underpass. Storm sewer tunnels are understood to exist at depth (in the bedrock) to collect storm water and meltwater run-off from the underpass and the several adjacent road intersections and commercial or industrial structures.

The site area was found to be clear of woodland or scrub vegetation and to be covered with grasses, weeds or sod.

3.0 PHYSIOGRAPHY AND GEOLOGY

The general site area is located on an extensive formerly level tract of land that was formed as a shoreline and offshore wet beach feature of glacial Lake Iroquois. The beach line and weathered cliffs of this glacial lake shoreline are clearly visible in the fields located immediately north of the Queen Elizabeth Way. To the south of the CN Right-of-Way, the land begins to slope gently downwards towards the flat coastal plain that borders present-day Lake Ontario. The area is defined as lying within the Iroquois Plain physiographic region.

Overburden at the site has been found in all the geotechnical surveys to comprise a heterogenous mix of silt, sand, gravel and clay sizes that

3.0 PHYSIOGRAPHY AND GEOLOGY (Continued)

has been variously identified as Lake Iroquois beach formation deposits, fill or glacial till. Despite the clay content, the material is essentially granular in character and ranges from compact to very dense. Some zones of clayey cementation appear to exist in the zone of seasonal groundwater fluctuation. The base of the overburden appears to vary from place to place but comprises either a stoney glacial till or a series of slabby boulders and cobbles set into a clay or highly weathered shale matrix (ie erosional lakeshore *boulder pavement*).

Bedrock in the area comprises a repeatedly interbedded series of dark red or greenish-grey shales, mudstones, siltstones and limestones in which the shales and mudstones predominate. At the site the rock condition varies repeatedly from deeply weathered to sound for some depth below natural rock contact. A more dominantly sound sequence ultimately develops at a depth of 1 to 3 m into the rock, by transition (ie reduction of intensity, frequency and thickness of the weathered, more shaley rock types). Several zones or seams of clay were noted in the rock sequence.

The rock is tentatively identified as comprising the basal sequence of the Queenston Shale Formation. This formation is of Upper Ordovician (Paleozoic) age and the base of the formation is known to pass transitionally into the underlying Meaford Unit of the Middle Ordovician Meaford-Dundas Formation (Georgian Bay Formation). The interbedded dark red and greenish-grey colouration is characteristic of both the basal Queenston Shales and the Meaford Unit, while presence of hard, thin limestone or siltstone beds is noted as being characteristic of the Meaford Unit sequence. The type section of the transition zone is located along the valley wall of Oakville Creek, nearby.

All bedrock units in the Region are known to be characterized by high residual horizontal stresses. Due to the existence of these unbalanced residual stresses, the upper layers of rock tend to become fractured or buckled at intervals. This results in the occurrence of elongated, restricted zones of broken (rubbled) rock that tend to occur in an approxi-

3.0 PHYSIOGRAPHY AND GEOLOGY (Continued)

mately WNW to ESE trend at intervals of 200 to 600 m. Such upward buckling, rubbling and bedding plane separation tends to accelerate the weathering process and resultant localized areas of intensely altered clayey mylonite or gouge are known to exist.

The natural groundwater surface is known to lie within the surficial Lake Iroquois beach formation gravels and sands. This condition has been modified by urban development and the water level is now likely to be located anywhere between the original level and some depth into the bedrock. Control of groundwater is currently being provided by various ditches, open-cuts and backfilled service trenches along Dorval Drive. New excavation may possibly modify the present system.

Where the water level is now in bedrock well below the original natural position, the rock above the newly lowered phreatic surface will be subjected to accelerated weathering.

A stable groundwater pattern is unlikely to have yet developed at the site due to the relatively recent nature of various construction changes at or in close proximity. Several more years after the planned construction will almost certainly be required for the ultimate stable condition to develop.

Seismic risk along the north fringe of the Lake Ontario - Lake Erie Rift Trench is noted in Federal E, M & R publications as Level 1. At this level and with no seismically sensitive soils present at the site, no seismic risk is assessed for the intended structures.

4.0 FIELD WORK: PROCEDURES AND EQUIPMENT

The field work for this investigation was carried out in two phases, related to the availability/non-availability of hydrant water for core drilling. A diamond drill equipped to use solid stem augers for soil boring

4.0 FIELD WORK: PROCEDURES AND EQUIPMENT (Continued)

and sampling as well as for diamond core drilling in N size was mobilized to the site on February 3rd, 1984, after several days or part-days of technician involvement in obtaining site entrance permits, site access and stakeouts for gas lines, hydro, sewers, phone, etcetera. Overburden drilling of one hole was accomplished before the field crew were advised that

a. no drilling of any kind would be allowed unless the gas company fieldman was present at the site (ie intended weekend work had to be cancelled) and

b. authority from the Town of Oakville to use fire hydrants for drill water supply was over-ruled by the Municipal Region of Halton.

The latter ruling was not confirmed until Tuesday, February 7th, 1984, at which time no alternative water source from neighbouring industrial or commercial premises could be arranged. The rig and crew were thereupon demobilized while alternative water source, water trucking and drill equipment arrangements were organized.

Appropriate equipment, crew and water supply vehicles were scheduled back to the site on February 26th, 1984, which was the earliest that alternative (new) crews and equipment could be made available. The onset of deep cold and heavy snowfall thereafter slowed progress down due to freezing up of trucked or tanked water, water lines and pumps. Waterline heaters were used as appropriate. Gas company and other stake-out personnel required to be on site full-time failed to show on several occasions (explained as being due to severe weather).

On March 2nd, 1984, use of a nearby hydrant for water supply was negotiated on an emergency basis and the core drilling was thereafter advanced more rapidly, to be concluded at nightfall on Sunday, March 4th, 1984.

Throughout both phases of the field work, the boreholes were advanced through penetrable overburden using solid stem or hollow stem augers. Samples were taken during augering using the Standard Penetration Test method. Sto-

4.0 FIELD WORK; PROCEDURES AND EQUIPMENT (Continued)

ney overburden and bedrock were drilled using an NXL core barrel, which represented the largest core size available using N series equipment. N series drill rods were used throughout to minimize drill vibration, since soft or broken, weathered bedrock layers or zones were anticipated. N casings were needed in each borehole, to case off the caving, granular overburden. Such casings were drilled through the stoney overburden and weathered bedrock in stages, after each coring run.. An approximately 3.0 m long core barrel was used for the final runs in sounder bedrock, so as to obtain safe penetration to maximum 3.0 m depth below each casing shoe.

Boreholes were left standing open until March 7th in order to determine groundwater level. The holes were thereafter backfilled.

All samples of soil and rock were identified, classified and logged in the field and were thereafter returned to the Toronto area laboratory of Morton & Partners Limited for further detailed examination, testing and storage. Such samples are to be retained for a period of three months following submission of this report, but will thereafter be discarded unless instructions to the contrary are received prior to the scheduled discard date (June 4th, 1984).

Precise locations and elevations of each borehole, as completed, were established using locational and elevation markers that were noted on the set of drawings and documents provided by GO-ALRT at commencement of study. Actual locations were finalized in accordance with the various stake-out clearances. As such, they are somewhat different from those intended that were marked (by agreement) on the initial drawing set. Geographic grid locations and Geodetic elevations (in metric) are listed on the Site Layout Plan, see attached.

5.0 SOIL CONDITIONS

The soil strata and range of elevations between which they occur are shown clearly on the attached individual borehole logs. For ease of reference this information has been summarized in the composite profiles on the Site Plan. The various dominant soil units are briefly noted in the following paragraphs.

5.1 Topsoil

No fully developed natural humic topsoil layer was noted in any of the seven boreholes, except for about 60 to 75 mm of sod in the boulevard areas adjacent to Dorval Drive (between the sidewalks and the curbs). Elsewhere a grass or weedy grass cover was found to be growing out of the surficial soil, such that no clearly organic-rich layer exists. A zone with root fibres extends down some 1.0 m depth, which at the time of the 2 phases of site investigation was partly frozen.

5.2 SILT, SAND and GRAVEL with trace CLAY

This layer is the surficial layer of soil on the high ground that is located to either side of the Dorval Drive roadway cut. In part, the material may comprise fill or include similar soil that has been used to backfill service trenches or to form the railway embankment or to "level up" undulations in the natural ground surface. The layer extends to natural depths of 4.55 m in Borehole 2 and 6.78 m in Borehole 7. Refusal to auger or sampler advance within the stratum was noted at a depth of 1.98 m in Boreholes 1 and 7 and at 2.90 m and 1.07 m in Boreholes 2 and 6 respectively. Cause of refusal was identified by core drilling in Borehole 2 and 7 as being cobbles or boulders.

The base of the stratum may include a glacial till remnant in Borehole 7, but directly overlies weathered shale bedrock at Borehole 2.

Samples of the stratum below point of auger refusal comprise gravel or stone fragments only that were recovered in the coring process, plus

5.2 SILT, SAND and GRAVEL with trace CLAY (Continued)

rare instances of collected wash water return*. In both Boreholes 2 and 7, the basal fragments of cobble or stone were identified as slabby pieces of greenish-grey siltstone or limestone that is typical of certain hard beds within the bedrock. Their presence is interpreted herein as representing an erosional "lag" deposit or boulder pavement (the sole formation when active glacial ice movement or thrust occurs over bedrock of varied texture or hardness).

5.3 SAND, GRAVEL and SILT (FILL)

In Boreholes 3, 4 and 5 a layer of granular fill exists at ground surface that extends down to 1.12 to 1.60 m depth. This fill includes the road base and backfill of service trenches and sidewalks. It is present in a compacted, dense state. The lower contact rests on an excavated surface of bedrock that appears to be now undergoing progressive weathering.

5.4 TILL

A thin clayey stoney glacial till remnant was possibly intersected at the base of the Iroquois beach formation sediments in Borehole 7. Positive identification was not possible from the quality of core sample recovered. Such a glacial till formation, however, was identified by Geocon during their investigation in 1974.

5.5 BEDROCK SHALE (Weathered)

Boreholes 2 and 7 were advanced by core drilling into the natural sequence of overburden and bedrock that lies beneath the general surface of the Lake Iroquois strand flat. A zone of deeply weathered shale, mudstone and claystone was noted in Borehole 2 only, however, where the Lake Iroquois beach deposits appear to directly overlie bedrock.

(*) In most cases no water return to surface was noted when core drilling in tough overburden or weathered, fractured bedrock conditions.

5.5 BEDROCK SHALE (Weathered) (Continued)

In Borehole 7, by contrast, the lowest portion of the overburden appears to comprise a clayey till and the bedrock beneath is present in a relatively continuously sound basis. The weathered shale comprises a sequence of deteriorated reddish mudstone or shale in which the harder, thin siltstone or limestone layers are still relatively sound. The shales have weathered to a clay or clayey gouge in some instances. In those cases where such clay was recovered during core drilling it appeared to be of stiff consistency. Elsewhere the material was eroded during drilling leaving a gap in the core recovery.

5.6 BEDROCK SHALE (Sound)

Bedrock at the site was intersected in all 5 of the core drill holes, beneath a cover of granular fill at Holes 3, 4 and 5 and a natural sequence of weathered rock or overburden at Borehole 2 and 7. The sound rock was found to comprise a varied sequence of thinly bedded to fissile shales, mudstones and, more rarely, siltstones and limestones. Colour is sharply variable between dark red and greenish-grey. Some fractured zones (or layers) of weathered rock occur in the sequence and continuously fresh, sound rock units of more than 0.5 m thickness were noted only in Boreholes 4 and 7. The rock is tentatively identified as belonging to the transition beds between the Meaford Unit of the Georgian Bay Formation (previously referred to as the Meaford-Dundas Formation) and the Queenston Formation.

Shale and mudstone appear to make up well over 50 per cent of the sequence, with the balance comprising harder (and more jointed) siltstone, sandstone or limestone interbeds. The latter, harder beds are usually of greenish-grey colouration, while the shales are invariably a deep purplish to brownish red. The mudstones are either dark red (predominantly) or greenish-grey (along or adjacent to joints or bedding planes). The shale is locally reduced by weathering to a soft state bordering on claystone or clay gouge which exist as thin interbeds in the normal sequence of near-horizontal bedding. In addition, all rock

5.6 BEDROCK SHALE (Sound) (Continued)

types whether weathered or not are occasionally extensively to intensively fractured (ie *"rubbled"*). Such latter zones exist within otherwise completely sound rock sequence and are believed to result from bedding plane slippage during geologically recent isostatic crustal movement.

In situ stress conditions of the rock have not been examined or quantified, but several indicators that high lateral stresses exist in the sounder unweathered rock were noted. The principle such indicator was the tendency for the core to fracture vertically in certain (more brittle) beds, while splitting laterally along planes of fabric or textural change. In one case, the lateral fracture was followed immediately by annular spalling of complete *"wedge rings"* on one side of the first fracture (or both).

6.0 GROUNDWATER

During augering and SPT sampling of overburden, no free groundwater was noted. After several hours or days, however, a free water surface was noted to develop immediately above the (often clayey) cobble or boulder layer on which refusal was recorded (e.g. EL 103.9 in Borehole 1 and EL 104.55 in Borehole 7). This is regarded as representing a perched water condition. In Borehole 7, a later reading taken after core drilling had penetrated through the clayey boulder layer yielded a stable water level at approximately bedrock contact, ie at EL 99.65.

In Boreholes 3, 4 and 5 located in the bottom of the existing road cut, the groundwater level was recorded at or close to excavated bedrock contact, at the base of the granular fill. This is believed to coincide with the invert level of the weeping tile and the roadway subbase elevation, EL 99.44 to EL 99.76.

7.0 DISCUSSION AND RECOMMENDATION

7.1 General

The project forms part of the intended regional rapid transit rail system that has been proposed by the Government of Ontario for the Hamilton - Toronto - Oshawa urban corridor. Two bridge design alternatives have been proposed comprising

a. a closed-type abutment, founded well down the existing roadway cut slope, or

b. a nominal bridge-seat abutment founded more or less at the top of slope but with minimum frost-protective cover as determined normal to the slope as well as vertically downward.

In both cases a central pier is to be located in the roadway median.

Projected top of rail for the subway (bridge) structure is EL 107.0 approximately.

7.2 Structural Foundations; Bearing Depth and Allowable Bearing Capacity

It is recommended that simple spread footings be considered for all the proposed pier and abutment structures. Such footings must be provided with adequate frost protective cover, which is nominally recommended in M.T.C. guidelines at 1.2 m. Since the structure is to be unheated and will be subjected to deep shade on the exposed northward-facing abutment walls however, additonal coverage to 1.8 m depth is suggested*.

Allowable bearing pressures computed for the various strata are in general agreement with those suggested in the Feasibility Foundation

(*) Several cases are now on record in the Hamilton - Toronto Region of winter frost penetration and ice lens formation to depths as great as 2.0 to 2.1 m beneath footings that are separated from any heated enclosure, are located in permanent winter shadow and have snow removed from ground surface.

7.2 Structural Foundations;
Bearing Depth and Allowable Bearing Capacity (Continued)

Investigation, 1983, except that placement of footings on weathered bedrock containing a significant number of clay seams is not recommended, since the consistency and continuity of such clay seams is difficult to establish with any degree of certainty. If footings must be considered in the weathered shale zone, then a plate load test is recommended.

Recommendations based on the results of the field investigation are outlined in the following Table, see Table 1. Differential settlement of spread footings is not expected to exceed 25 mm and will, in most instances involving sound construction procedures, be much less. All footings will require minimum frost cover.

Sliding restraint of foundation bases may be computed using an ultimate friction coefficient of 0.45 where sound shale or mudstone bedrock is protected against long-term deterioration or softening (ie remains below the water table. This value should be reduced to 0.4 where progressive deterioration of the rock is anticipated after completion of construction. These recommended friction coefficients are somewhat lower than might be routinely recommended for rock, but are based on test data recently obtained by MTC for the particular type of bedrock involved at the Dorval Drive site and on the anticipation of clay partings along bedding planes.

Additional resistance, if required, should be provided by constructing a foundation "key", such that passive restraint by foundation soil or rock is provided. Such passive restraint (P) at a point about 1/3 the height of the key above the base of the key may be approximated for cohesive soil or bedrock by the combined Rankine and Bell Formulae,

$$P = \frac{K_p}{2} \gamma h^2 + \frac{2c}{\sqrt{K_a}} h$$

where

K_a and K_p = active and passive earth pressure coefficients

7.2 Structural Foundations;
Bearing Depth and Allowable Bearing Capacity (Continued)

γ = unit weight of foundation rock or soil (assume 19.5 kN/m^3 for soil and 24.0 kN/m^3 for shale bedrock),

h = a variable between the full depth of the key and about 2/3 depth of key,

c = 20 to 850 kPa depending on the quality of rock and the presence or absence of clay partings.

7.3 Foundation Excavation

The overburden soils and weathered shale formation are all capable of mechanical excavation using routine construction equipment. Some of the overburden soil will be stoney and may contain cobbles or boulders. Such material cannot be used for backfill around conduits or against concrete or steel structures. Weathered shale dug from excavations is likely to become subject to rapid further weathering or frost destruction and must not be used for backfill.

Hard, sound bedrock should be capable of being dug out using more powerful than normal, special excavation equipment; otherwise explosives for light line blasting of footing trenches may be needed. Shale rock should not be used for backfill, since its condition will deteriorate with exposure and with time.

Dewatering of anticipated excavations is not expected to be a problem. Weeping tile systems should be installed behind the abutment structures for safe drainage control, plus free draining backfill.

Side slopes in earth or weathered rock must be cut back to a safe slope of not less than 1 vertical to 1 horizontal, to comply with Provincial safety standards. In sound bedrock, the safe cut slope can be adjusted to 4 vertical to 1 horizontal. Alternatively, the cut margins can be supported using braced shoring designed to an earth pressure coefficient of K_a = 0.25.

7.4 Abutment Face and Side Walls, Earth Pressures

Each abutment wall must be designed to safely support the contained approach fill. Such fill is recommended to comprise free-draining granular borrow confirming to M.T.C. Class "B" gradational requirements. The fill will be placed in individually compacted lifts not exceeding 250 to 300 mm thickness. An average in situ unit weight, $\gamma = 19.5\ kN/m^3$ is suggested for earth pressure computation. An earth pressure coefficient of $K_a = 0.5$ is recommended to allow for full compactive effort and possible vibratory roller weight positioned close to the retaining wall. Such construction condition during placement of the last lift of fill is likely to represent the governing (worst) condition.

Calculations should be in accordance with the following general equation and must include for all anticipated long-term and cyclical short-term (transient) surcharges:

$$p = K \gamma H + q$$

where K = earth pressure coefficient

γ = unit weight of backfill

H = height of fill and equivalent uniform surcharge above point under consideration

p = lateral earth pressure

q = point load surcharge.

This Rankine Theory equation assumes a smooth vertical rear face for the retaining wall and hence tends to overestimate the actual lateral earth pressures that are exerted.

7.5 Slope and Abutment Footing Stability

The existing slopes of the subway road cut are approximately 6 m overall height and are stable.

Any footing to be built on the slope must be positioned to provide adequate frost cover as measured perpendicular to the plane of the slope. In addition, the front margin of the footing should be set such that it

7.5 Slope and Abutment Footing Stability (Continued)

is located one full width of footing back in from the plane of the slope as measured at founding level. Given these criteria and the general granular nature of the overburden, it is unlikely that the stability of the underpass slopes will be critically diminished. It would be prudent, however, to review the overall stability of each slope once the footing locations are finalized and the applied foundation loadings are defined.

Respectfully submitted.

MORTON & PARTNERS LIMITED

John D. MORTON, M.Eng., P.Eng., P.Geol.
Principal

JDM/lh

T A B L E 1

A L T E R N A T I V E S	FACTORED CAPACITY AT ULS	CAPACITY AT SLS (Type II)	APPROX. ELEVATION			Limit
			Side	Centre	W-Side	
a. Abutment Footings on compact to very dense in situ Lake Iroquois Beach Formation or on newly constructed structural fill......	900 kPa	340 kPa	104.5	n/a	106.0	Upper
			-		-	Lower
b. Spread Footings on very dense cemented Lake Iroquois Beach Formation gravel or Till below initial refusal level (see Borehole Records)....................	1000 kPa	670 kPa	103.5	n/a	104.5	Upper
			102.5		100.5	Lower
c. Spread Footings on continuously sound bedrock below final water level (for elevations see Borehole Records)...............	1500 kPa	not a governing criterion	98.6	98.0	98.5	Upper
d. Spread Footings on weathered rock (not recommended due to extreme variability of material or individual clay seam thicknesses evidenced during drilling..........	To be determined on site for each individual case by plate load test		102.5	99.7	100.5	Upper
			98.6	98.0	98.5	Lower

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 1

METRIC

CO-ORDS 4811302 N; 289015 E

W O 82-26025 LOCATION W. Extension over Dorval Drive, Oakville, Ontario ORIGINATED BY BR

DIST 4 GO-ALRT BOREHOLE TYPE Solid Stem Auger, COMPILED BY BS

DATUM Geodetic (Metric) DATE February 24, 1984 CHECKED BY JDM

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT Wp, NATURAL MOISTURE CONTENT W, LIQUID LIMIT WL; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
105.93 / 0.0	Silt with SAND and GRAVEL, trace clay; compact; brown (Lake Iroquois Beach Formation)					FROZEN					
			1	SS	9		105				
			2	SS	65		104				
103.95 / 1.98	End of Borehole (Effective Refusal to Auger penetration in v. dense gravel or cobbles with shaley clay matrix.)					Feb.25/84					

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2

METRIC

CO-ORDS 4811310 N; 289003 E

W O 82-26025 — LOCATION W. Extension over Dorval Drive, Oakville, Ontario — ORIGINATED BY BR

DIST 4 GO-ALRT — BOREHOLE TYPE Solid Stem Auger, NXL Rock Core — COMPILED BY BS

DATUM Geodetic (Metric) — DATE February 24, 1984 — CHECKED BY JDM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS / DRILLING NOTES	RQD
106.43							
0.0	SILT with SAND and GRAVEL: trace clay, compact, brown (Glacial Lake Iroquois Beach Formation)	1	SS	18	FROZEN		
		2	SS	8			
		3	SS	52	2.9 Feb.25/84		
103.53 / 2.90	GRAVEL, some sand, very dense, tr. clay (Lake Iroquois Beach Formation)	4, 5, 6, 7, 8, 9	RC	78%		Difficult advance in 10 runs in clayey gravel or in weathered rock. Several clay seams noted. Heavy drill water loss.	0
101.89 / 4.55	BEDROCK SHALE (WEATHERED): reddish, soft to hard	10, 11, 12, 13	(NXL)				
100.89 / 5.54	← 80 mm clay seam. BEDROCK SHALE (SOUND): interbedded shales, mudstones, siltstones and limestones; thin-bedded; predom. sound with weathered inter-beds	14	RC (NXL)	100%		Good advance in last single run when in sound rock. Low to moderate drill water loss with good (continuous) core recovery.	43
99.39 / 7.04	End of Borehole						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (106, 105, 104, 103, 102, 101, 100); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%); UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3$, $\times^5$: Numbers refer to Sensitivity — 20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 3

METRIC

W O 82-26025 | LOCATION W. Extension over Dorval Drive, Oakville, Ontario | CO-ORDS 4811297 N; 288992 E | ORIGINATED BY BR

DIST 4 GO-ALRT | BOREHOLE TYPE Solid Stem Auger, NXL Rock Core | COMPILED BY BS

DATUM Geodetic (Metric) | DATE March 4, 1984 | CHECKED BY JDM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH o UNCONFINED + FIELD VANE • QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
100.79 / 0.0	SAND with SILT, traces clay and gravel, compact, brown (FILL)		1			FROZEN AS	100				
99.44 / 1.35	BEDROCK SHALE (WEATH.): interbedded shales, mudstones, siltstones and limestones; variably sound to weathered; thin-bedded; reddish or green		2			Mar.6/84					RQD
			3	RC			99				
98.59 / 2.20	BEDROCK SHALE (SOUND)		4	(NXL)	50%						0
			5								
98.10 / 2.69	End of Borehole										

DRILLING NOTES

Drilling advance in 4 runs due to bit blocking in clay or gouge (weathered) zones.
Heavy drill water loss except in last run, when in continuously sound rock.
Poor core recovery.

FOOTNOTE:

Borehole located at base of roadway open-cut, at boulevard/ sidewalk level.

OFFICE REPORT ON SOIL EXPLORATION

$+^3, \times^5$: Numbers refer to Sensitivity

15 ⌖ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 4

METRIC

CO-ORDS 4811284 N; 288982 E

W O 82-26025 LOCATION West Extension over Dorval Drive, Oakville, Ontario ORIGINATED BY BR

DIST 4 GO-ALRT BOREHOLE TYPE Solid Stem Auger, NXL Rock Core COMPILED BY BS

DATUM Geodetic (Metric) DATE March 4, 1984 CHECKED BY JDM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
100.75											
0.0	SAND with silt, traces clay and gravel; compact, brown (FILL)		1	AS		FROZEN	100				
99.63 / 1.12	BEDROCK SHALE (WEATH.) red or green, interbedded sound limestones or siltstones with weathered claystone, shale or mudstone		2			Mar.5/84	1.1		DRILLING NOTES: Slow advance in 3 runs, with intermittent moderate to heavy loss of drill water. Bit tending to "mud" in clayey shale beds. Poor core recovery.		RQD 0
			3	RC (NXL)	65%		99				
			4				98				
97.97 / 2.78	BEDROCK SHALE (SOUND) sound red mudstone/shale		5	RC (NXL)	100%				Continuous core in red shale, with single thin green parting		97
97.52 / 3.23	End of Borehole										

FOOTNOTE:

Borehole located at base of roadway open-cut, at boulevard/sidewalk level.

+3, x5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 5

METRIC

CO-ORDS 4811267 N; 288969 E

W O 82-26025 LOCATION W. Extension over Dorval Drive, Oakville, Ontario ORIGINATED BY BR

DIST 4 GO-ALRT BOREHOLE TYPE Solid Stem Auger, NXL Rock Core COMPILED BY BS

DATUM Geodetic (Metric) DATE March 3, 1984 CHECKED BY JDM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
101.36							
0.0	SILT, SAND and GRAVEL: compact, brown. (FILL)	1	AS			101	
						100	
99.76 / 1.60	INTERBEDDED WEATH. & SOUND BEDROCK SHALE red and green shales, mudstones, siltstones and limestones; interbedded sound rock with zones of soft clayey weathered claystone or mudstone	2	RC		99.76 Mar.5/84	99.76	
		3	(NXL)	83%		99	RQD 0
98.36 / 3.0	SOUND SHALE BEDROCK	4					
98.11 / 3.25	End of Borehole						

DRILLING NOTES

Slow drilling advance in 3 runs in varied sound to weathered rock.
Heavy drill water loss throughout.

FOOTNOTE:

Borehole located at base of roadway open-cut, at boulevard/ sidewalk level.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%); UNIT WEIGHT γ; STRAT PLOT

+3, ×5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20, 10)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 6

METRIC

CO-ORDS 4811259 N; 288960 E

W O 82-26025 LOCATION W. Extension over Dorval Drive, Oakville, Ontario ORIGINATED BY BR

DIST 4 GO-ALRT BOREHOLE TYPE Solid Stem Auger COMPILED BY BS

DATUM Geodetic (Metric) DATE February 28, 1984 CHECKED BY JDM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
106.76											
0.0	SILT, SAND & GRAVEL: trace organics and clay, dense, brown (Lake Iroquois Beach Formation)		1	SS	46	(*) FROZEN	106				
105.70 / 1.06	Auger Refusal (End of Borehole)										
	(*) WL NOT ESTABLISHED										

$+^3, \times^5$: Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 7

METRIC

CO-ORDS 4811254 N; 288966 E

W O 82-26025 LOCATION W. Extension over Dorval Drive, Oakville, Ontario ORIGINATED BY BR

DIST 4 GO-ALRT BOREHOLE TYPE Solid Stem Auger, NXL Rock Core COMPILED BY BS

DATUM Geodetic (Metric) DATE February 28, 1984 CHECKED BY JDM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
106.48							
0.0	SILT and SAND with GRAVEL; dense, brown (Lake Iroquois Beach Formation)	1	SS	43/(0.15m)	FROZEN	106	
		2	SS	50			
						105	
104.50	Refusal to Augers						
1.98	GRAVEL with SAND and SILT; some clay (Beach Formation in part; possibly Till at base) very dense	3				104	
		4					
		5				103	
		6					
		7	RC				
		8	(NXL)	25%		102	RQD 0
		9					
		10				101	
		11					
		12				100	
99.70 / 6.78	INTERBEDDED WEATH. & SOUND BEDROCK SHALE: greenish grey or dk.red shales & mudstones with thin siltstones and limestones; some interbeds weathered to claystone or mudseams	13	RC		99.65 Feb.29/84		
99.18 / 7.30		14	(NXL)	76%		99	44
		15					
98.12	SOUND BEDROCK SHALE						
8.36	End of Borehole						

Columns also on form (no entries): STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%); UNIT WEIGHT γ.

DRILLING NOTES

Intermittent to poor recovery in very dense, stoney overburden formations of clayey beach gravel and possibly glacial till containing slablike cobbles or boulders.

Total drill water loss in fractures.

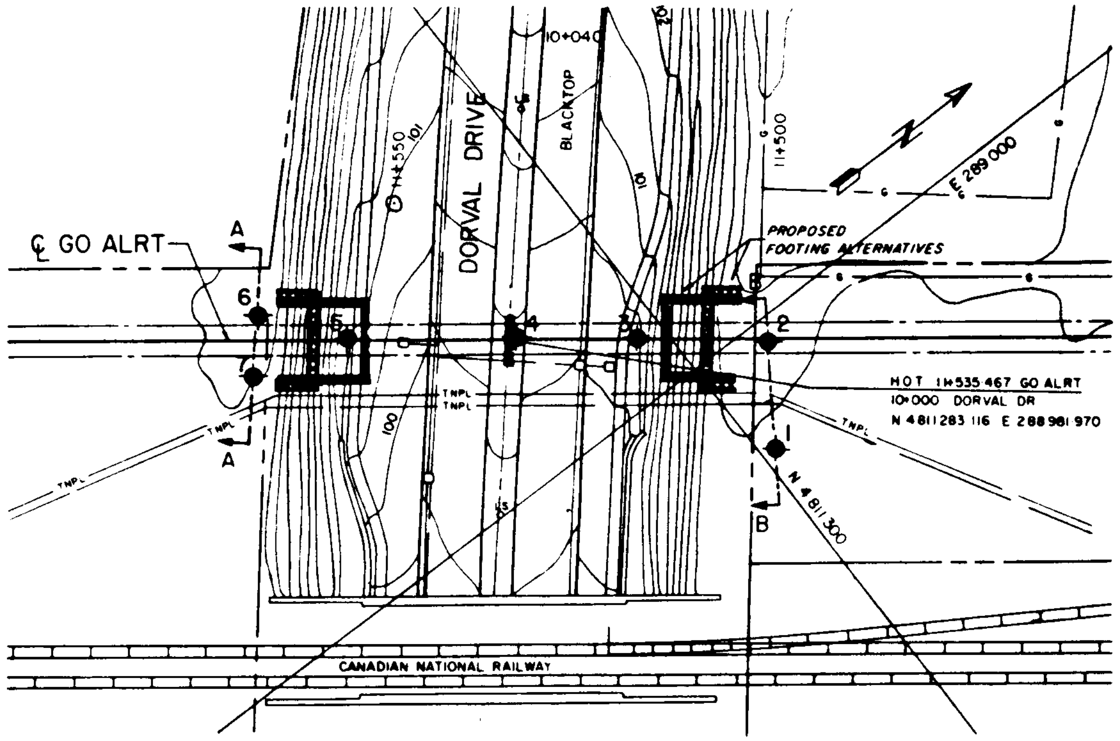
Moderately fast core bit penetration in layered sound and weathered rock; some blocking ("mudding") of bit when in softer weathered shale or mudstone; partial to total drill water loss in fractures or fissures during 3 core runs.

$+^3, \times^5$: Numbers refer to Sensitivity

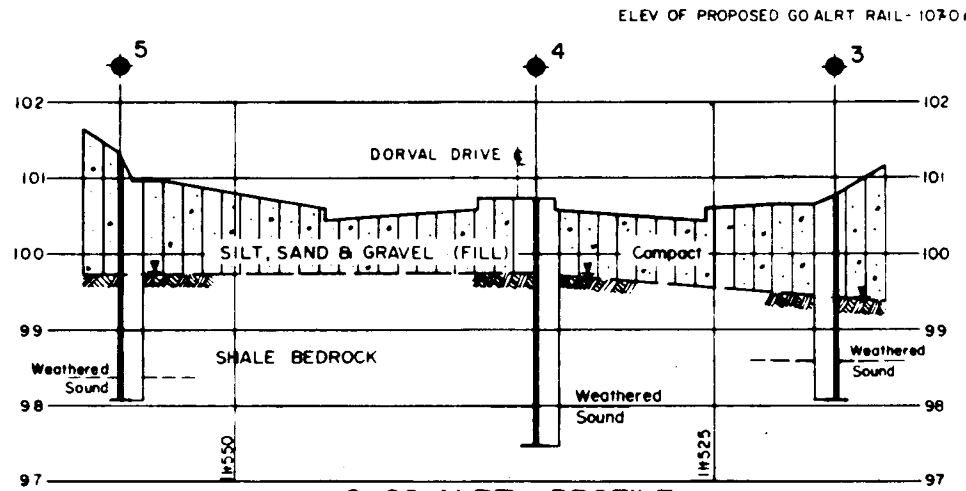
15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

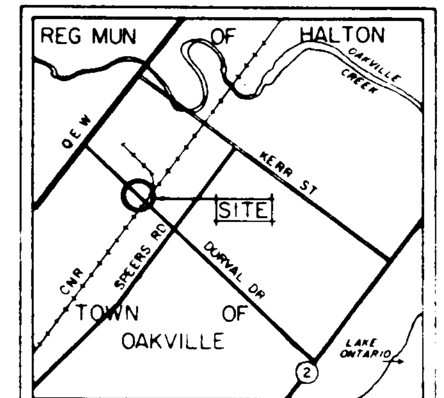
METRIC
ALL DIMENSIONS SHOWN ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE NOTED



PLAN
SCALE
10 5 0 10 20m



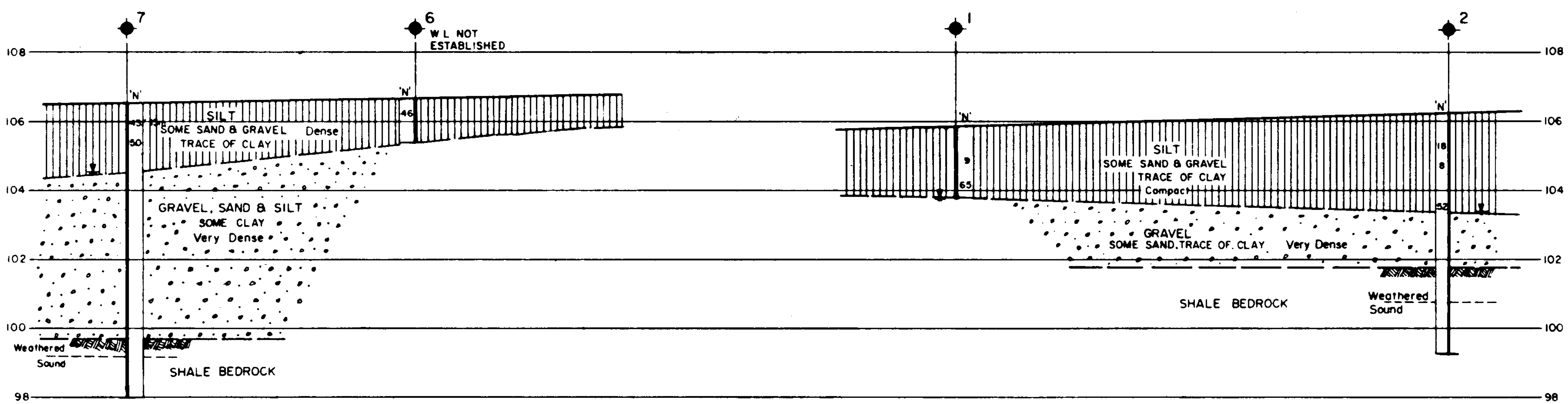
℄ GO ALRT PROFILE
SCALE
HOR 5 0 5 10 m
VERT 1.25 0 1.25 2.5 m



KEY PLAN
SCALE
500 0 500 1000 m

LEGEND	
◆	Bore Hole
⊕	Dynamic Cone Penetration Test (Cone)
◈	Bore Hole & Cone
N	Blows/0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/0.3m (60° Cone, 475 J/blow)
▼	W.L. at time of investigation 84 02 03

No	ELEVATION	BOREHOLE NORTH	CO-ORDINATES EAST
1	105 93	4811302	289015
2	106 43	4811310	289003
3	100 79	4811297	288992
4	100 75	4811284	288982
5	101 36	4811267	288969
6	106 76	4811259	288960
7	106 48	4811254	288966



A-A SECTIONS B-B
SCALE
2 0 2 4 m

Geocres No

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102.2 of Form 100.

GO-ALRT REF PD2-300-

REFERENCE DRAWINGS	REVISIONS	DRAWN BY: 1984 03 27	DESIGNED BY:
		CHK'D BY:	APPROVED BY:
		SCALE: AS SHOWN	FULL SIZE ONLY

MORTON & PARTNERS LTD
TORONTO ONTARIO

GO-ALRT
Ministry of Transportation and Communications
OAKVILLE PROJECT WEST EXTENSION
PROJECT MANAGER

HALTON REGION
DORVAL DRIVE SUBWAY
BOREHOLE LOCATIONS & SOIL STRATA
STA 11+535.467

CONTRACT NO	DWG NO	REV	SHEET