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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN REPORT NOISE WALL BARRIER WEST OF MONTREAL STREET KINGSTON, ONTARIO G.W.P. 78-99-00

Submitted to:

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REPORT



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PART A

FOUNDATION INVESTIGATION REPORT
NOISE WALL BARRIER WEST OF MONTREAL STREET
KINGSTON, ONTARIO
G.W.P. 78-99-00



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation associated with the Highway 401 expansion in Kingston, Ontario. The section of Highway 401 included in this assignment (G.W.P. 78-99-00) extends from west of Montreal Street to about 1.8 kilometres east of the Canadian National Railway (CNR) structure.

Foundation investigation services are required for the following components:

- CNR Bridge Rehabilitation/Widening;
- Highway 401 Embankment Widening – Cataraqui wetlands;
- Montreal Street Underpass Replacement;
- Overhead Signs (total of 2); and,
- Noise Barrier Wall.

This report addresses the noise wall barrier component, Geocres Number 31C-203.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated April 2008. The work was carried out in accordance with Golder's Quality Control Plan dated November 2008.



2.0 SITE DESCRIPTION

The noise wall barrier site is located west of the existing Montreal Street underpass and south of Highway 401 between Stations 26+080 and 26+280, near Kingston, Ontario. Based on information provided by MRC, the roughly 245 m long noise wall will be constructed to separate the rear property line of eight residential properties at 1504 through 1530 Montreal Street from Highway 401 and the N/S-E ramp to the northwest. No specific information was provided on the proposed height or configuration of the noise wall.

The residential properties are situated on the tablelands some 15 to 20 m back from the crest of the near-vertical exposed rock side walls at Highway 401. Within the MTO right-of-way between the rear of the residential properties and the crest of the rock cut, vegetation cover generally consists of grass, bushes, and a few mature trees. Some fill has been placed at the rear of the residential properties, creating a slope between the noise wall and the crest of the rock cuts. The existing ground surface elevation along the noise wall barrier increases from 101.7 m at the west end, to a peak of 105.2 m in the middle, to 103 m at the east end. Highway 401 at this location is currently a four-lane divided highway with a rural cross section which runs northeast-southwest in a rock cut up to about 7 m high. Based on available information, the approximate existing grade of Highway 401 adjacent to the noise wall barrier is about elevation 100 m.



3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the proposed location of the noise wall barrier between February 16 and 18, 2010 and on March 11, 2010, at which time four boreholes and three augerholes (numbered NW1 to NW4 and NW5 to NW7, respectively) were advanced at the locations shown on Drawing 1. The testholes were put down at accessible locations along the proposed wall alignment at roughly 50 m spacing. The boreholes and augerholes were located within 5 m of the proposed noise wall foundation locations, with the exception of NW6 and NW7 which were located 9 and 12 m from the noise wall, respectfully, due to accessibility constraints.

Boreholes NW1 through NW4 were advanced using 108 mm inside diameter (I.D.) continuous-flight hollow stem augers on a track-mounted drill rig supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths ranging from 4.1 to 6.5 m below the existing ground surface. Augerholes NW5 through NW7 were advanced to auger refusal at 2.2 to 2.7 m below the existing ground surface using 200 mm diameter (O.D.) continuous flight solid stem augers on a truck-mounted drill rig supplied and operated by Sunrae Construction Ltd. of Kingston, Ontario.

Soil samples were obtained nearly continuously during the borehole drilling at intervals of 0.75 m depth using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) ASTM D1586 procedures. Bedrock was cored in NQ size at each borehole. The boreholes were backfilled with bentonite pellets, mixed with native soils, and the site conditions restored following completion of the work. Groundwater conditions in the open boreholes were observed throughout the drilling operation and upon completion of drilling. Soils were visually logged but no samples were obtained at augerholes NW5 to NW7.

The field work was supervised throughout by a member of our technical staff, who located the testholes, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The soil samples were identified in the field, placed in appropriate containers, labelled, and transported to our Ottawa geotechnical laboratories where the samples underwent further detailed visual examination and laboratory testing, including grain size distribution, water content. Continuous samples of bedrock core were stored in boxes and laboratory unconfined compressive strength testing was carried out on selected core samples at Golder's Mississauga geotechnical laboratory. Laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

The borehole locations and ground surface elevations were determined by Golder personnel at the site using a Trimble R8 GPS unit. The augerhole locations and ground surface elevations were estimated relative to existing site features. The testhole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawing 1.

Testhole No.	Borehole/Augerhole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Approximate Noise Wall Chainage (m)	Ground Surface Elevation(m)
NW1	NW of 1504 Montreal St. rear fence	4904023.6	306711.4	62	103.5
NW2	NW of 1522 Montreal St. rear fence	4904064.1	306734.0	109	104.4



FOUNDATION INVESTIGATION - G.W.P. 78-99-00

Testhole No.	Borehole/Augerhole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Approximate Noise Wall Chainage (m)	Ground Surface Elevation(m)
NW3	NW of 1528 Montreal St. rear fence	4904113.0	306771.0	171	103.8
NW4	North of 1530 Montreal St. side fence	4904120.2	306813.6	213	103.2
NW5	SW of 1504 Montreal St. (btwn ramp and property)	4903982.5	306757.8	3	101.8
NW6	SW of 1504 Montreal St. (btwn ramp and property)	4903996.4	306739.2	26	102.3
NW7	SW of 1504 Montreal St. (btwn ramp and property)	4904011.4	306722.5	47	102.9



4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located in the physiographic region of southern Ontario known as the Napanee Plain (The Physiographic of Southern Ontario, Chapman and Putnam, 3rd Edition, 1984). The overburden is typically shallow. The Napanee plain, which is generally flat to undulating, has been stripped of most of its overburden during the late Wisconsinian glaciation period some 11,000 years ago.

Geologic mapping (Map 2544, Ministry of Northern Development and Mines, 1991) indicates the bedrock at the site consists of Paleozoic rock of the middle Ordovician age. The predominant bedrock type in the area is limestone of the Gull River Formation. The local bedrock is generally located at or near the ground surface. Within the area of the Montreal Street Underpass and adjacent noise wall barrier, the existing Highway 401 has been constructed in cut, exposing the limestone bedrock on both sides of the highway.

4.2 Site Stratigraphy

The detailed subsurface soil, bedrock, and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are given on the attached Record of Borehole and Augerhole sheets and on Figures 1 to 4. The testhole locations and ground surface elevations, together with a stratigraphic profile along the noise wall barrier, are shown on Drawing 1.

The stratigraphic boundaries shown on the Record of Borehole sheets and stratigraphic section are inferred from non-continuous sampling and in-situ testing and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the testhole locations.

In summary, the subsurface conditions encountered consist of up to about 0.8 to 3.1 m of fill or native silty clay to clayey silt soil overlying limestone bedrock at an elevation of between 99.5 and 103 m. Along the western portion of the wall at testholes NW5, NW6, NW7, NW1 and NW2, the limestone bedrock is overlain by about 2.3 m of silty clay to clayey silt. At NW2 the silty clay is, in turn, underlain by a thin veneer or till. Along the eastern portion of the wall at NW3 and NW4, the limestone bedrock is overlain by 0.8 to 0.9 m of fill.

A more detailed description of the subsurface conditions encountered in the boreholes put down at the site of the proposed noise wall barrier is provided in the following sections of the report.

4.2.1 Topsoil

A 50 to 180 mm thick surficial layer of topsoil was encountered in boreholes NW1, NW2, and NW4 and at augerhole NW5.

4.2.2 Fill

At ground surface at borehole NW3 and beneath 50 mm of topsoil at borehole NW4, roughly 0.8 m of fill comprising predominantly silt, with clay, gravel, sand, roots and organics was encountered.

4.2.3 Silty Clay to Clayey Silt

At boreholes NW1 and NW2 and augerholes NW5 to NW7, the topsoil is underlain by a deposit of native silty clay to clayey silt, containing a trace of sand. The deposit was fully penetrated at these testhole locations and varies in thickness from about 2.1 to 2.5 m.



Standard Penetration Tests carried out within the silty clay to clayey silt deposit gave 'N' values ranging from 14 to 24 blows per 0.3 m of penetration, indicating a very stiff consistency. The results of Atterberg limit testing carried out on two samples of the silty clay and clayey silt are shown on Figure 1. As the results plotted at or below the A-line, the two tests were repeated as a check and gave the same results. Results indicate plasticity index values of 18 and 24 percent and liquid limit values of 46 to 52 percent, reflecting intermediate to high plasticity silty clay to clayey silt. The measured water content of the silty clay to clayey silt ranges from approximately 28 to 33 percent, which is generally close to the measured plastic limit.

4.2.4 Sandy Silt Till

At borehole NW2, the silty clay and clayey silt are underlain by glacial till at a depth of 2.6 m below ground surface. The till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt with some clay. The till was fully penetrated at borehole NW2 and was 0.8 m thick. The recorded Standard Penetration Test 'N' value for this material was 27 blows per 0.3 m of penetration indicating a compact state of packing.

4.2.5 Auger Refusal and Bedrock

Bedrock was encountered beneath the fill, silty clay and glacial till, and cored for about 3 m depth, at boreholes NW1 through NW4. At augerholes NW5 through NW6, the top of bedrock was inferred from auger refusal

The following table summarizes the bedrock surface depths and elevations as encountered at the four borehole and three augerhole locations.

Borehole/ Augerhole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)	Approximate Noise Wall Chainage (m)
NW1	103.5	2.3	101.2	62
NW2	104.4	3.4	101.0	109
NW3	103.8	0.8	103.0	171
NW4	103.2	0.9	102.3	213
NW5	101.8	2.2	99.6	3
NW6	102.3	2.7	99.6	26
NW7	102.9	2.5	100.4	47

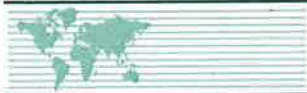
The bedrock encountered in boreholes NW1 to NW4 consists of fresh to slightly weathered grey limestone. The bedrock is generally strong and thinly to medium bedded. At boreholes NW2 and NW3 (from 4.6 to 4.9 m depth and 2.7 to 3.0 m depth, respectively), the rock core was highly fractured (disking) and friable. The Rock Quality Designation (RQD) values measured on recovered limestone core samples were quite variable and ranged from 0 to 67 percent, indicating a very poor to fair quality rock. The discontinuities observed in the rock core are typically horizontal, associated with the bedding planes. Some bedding joints were infilled with soil at boreholes NW-1 and NW-4. Borehole core photographs are presented in Figure 2. Photographs of the rock cut along Highway 401 adjacent to the proposed noise wall barrier are provided in Figures 3 and 4.



Laboratory unconfined compressive strength testing carried out on two intact samples of limestone core resulted in unconfined compressive strength (UCS) values of 76 and 110 MPa, indicating that the strength of the intact rock is strong to very strong. The results are summarized on Figure 5.

4.3 Groundwater Conditions

Groundwater was not encountered within the depth of our drilling investigation and no seepage or ice build-up was noted along the rock cut just north of the noise wall location. It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events and perched water conditions may form at the fill/rock or till/rock interface.



5.0 CLOSURE

This report was prepared by Ms. Erin O'Neill, P. Eng., under the direction of the Project Manager, Mr. Michael Snow, P. Eng.. Mr. Fintan Heffernan, P. Eng., Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

Yours truly,

GOLDER ASSOCIATES LTD.

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PART B

FOUNDATION DESIGN REPORT

NOISE WALL BARRIER WEST OF MONTREAL STREET

KINGSTON, ONTARIO

G.W.P. 78-99-00



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed noise wall barrier south of Highway 401 and west of Montreal Street between Stations 26-080 and 26-280 in Kingston, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes and augerholes advanced during the subsurface investigation at this site.

We understand that construction of the noise wall barrier will be administered as a design/build in accordance with SP 599F01. As such, the design alternatives provided herein are given with the understanding that the Contractor will retain an Engineer to design the noise wall barrier. The interpretation and recommendations herein are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed noise wall barrier foundations. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

The current plans for this project are to construct a new, roughly 245 m long noise wall barrier at the top of the limestone rock cut behind residential properties at 1504 to 1530 Montreal Street. Based on information provided to us by MRC, we understand that the noise wall is continuous with four segments from west to east:

- 60 m northwest-southeast section along west side of 1504 Montreal St. near Hwy 401 on-ramp;
- 95 m northeast-southwest section along rear of 1504 to 1526 Montreal St.;
- 20 m section along rear of 1528 Montreal St. parallel to Hwy 401; and,
- 70 m east-west section along rear property line of 1528 and 1530 Montreal Street.

The design of the retaining wall was underway at the time of preparation of this report; however, we understand that the central portion of the wall (noise wall chainage 145 to 185 m) will be built as a combination soil retaining wall and noise wall barrier. Preliminary sections provided by MRC indicate that the existing N/S-E ramp rock cut will be reconfigured such that the new cut will be as close as 3 m, horizontal distance, from the noise wall barrier and that, in places, up to 1.5 m of soil will need to be retained by the wall. We also understand that the existing 3-4 m high rock cuts may be deepened to a total of 5 to 6 m in height adjacent to the noise wall to accommodate the interim lane configurations, with the ultimate lane configuration resulting in a 3 m high exposed rock cut.

Preliminary sections indicate that the noise/retaining wall will be constructed a minimum of 3 m back from the crest of the new ramp N/S-E ramp rock cut and that retained soil heights will be about 1.5 m. Foundation engineering recommendations for the noise wall barrier are provided in Sections 6.3 to 6.5.

6.2 Noise Wall Barrier and Retaining Wall Foundation Options

The subsurface conditions encountered in the noise wall barrier area typically consist of up to about 0.8 to 3.1 m of fill or native soil overlying limestone bedrock. Along the western portion of the wall (west of NW2), the limestone bedrock is overlain by a thin veneer of till (at NW2 only) and about 2.3 m of silty clay to clayey silt.



Along the eastern portion of the wall at NW3 and NW4, the limestone bedrock is overlain by 0.8 to 0.9 m of fill. The surface of the underlying limestone bedrock was encountered between elevations 99.5 and 103 m.

The relatively shallow limestone bedrock is suitable for conventional spread or strip footings to support the proposed noise wall barrier. Suitable wall designs are cast-in-place concrete cantilever walls and gravity retaining walls. Consideration was also given to founding in the fill or native overburden deposits. It was noted, however, that overburden is not continuous over the length of the wall and that, once foundations are lowered to meet frost protection requirements, foundations will generally be at the top of rock or within about 1 m of the top of rock. For this reason, if conventional spread or strip footings are chosen, founding on the limestone bedrock rather than the overlying soils is recommended. It should be noted that spread footings may not be compatible with some proprietary noise wall designs.

Alternatively, the selected noise barrier wall design may be supported on drilled caissons which extend into the bedrock. Caisson foundations are better able to accommodate variability in depth to bedrock and provide improved resistance against lateral loads when compared to shallow foundations. Also, near the centre of the wall, bedrock is in the order of 3 to 4 metres below ground surface and approaching the practical limit of conventional spread/strip footing construction. For these reasons, a foundation solution using drilled caissons is the preferred technical solution. A comparison of foundation alternatives, including advantages, disadvantages, relative costs and risks associated with each foundation option are presented in Table 2 following the text of this report.

6.2.1 Strip/Spread Footings

6.2.1.1 Geotechnical Resistance

Strip or spread footings founded on the surface of the limestone bedrock below elevations 99.5 to 103 metres can be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 5 MPa for vertical concentric loads. Effects of load eccentricity need to be taken into account as appropriate in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* using the curve for “cohesive soil or rock”. Serviceability Limit States (SLS) conditions do not apply to footings placed on the limestone bedrock which is classified as non-yielding.

The factored geotechnical resistance value given above assumes that the bedrock at and below the founding level is not fractured and that no adverse jointing is present below the footings. Additional rock reinforcement (e.g. rock bolts, dowels, or shotcrete) may be required before the footings are constructed in order to ensure the integrity of the rock mass.

6.2.1.2 Resistance to Lateral Forces

The lateral pressures acting on the retained portion of the wall will depend on the type and method of placement of the backfill materials adjacent to the wall foundations and the subsequent lateral movement of the structure.

The resistance to lateral forces/sliding resistance between the concrete footings and bedrock should be calculated in accordance with Section 6.7.5 of the *Canadian Highway Bridge Design Code (CHBDC)*. Also, the retaining wall and noise wall barrier should be checked for overturning. Assuming that the founding rock is not loosened or disturbed during excavation and footing construction, the following angle of interface friction and corresponding unfactored coefficient of interface friction, $\tan \delta_i$, may be used for the interaction between the concrete and the founding rock:



Footings on limestone bedrock: Angle of interface friction $\delta_i = 35^\circ$

Unfactored coefficient of interface friction = $\tan \delta = 0.7$

In accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, sliding resistance can be supplemented by doweling the footings into the bedrock. The horizontal resistance of the dowels will be dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or is stronger than the concrete, the design of dowels in the rock may be handled the same way as the dowel embedded in concrete. The dowels should have a minimum embedded length of 1.0 m within the bedrock, and the structural strength of the dowel and the compressive strength of the grout should not be exceeded. If required, a Non Standard Special Provision for dowels in rock should be included in the contract documents. A sample has been included in Appendix B of this report.

The resistance to lateral loading on the foundations, as provided by the means of the geotechnical resistance or dowels, will be reduced wherever rock is not present below the founding level in the areas in front of the footings (e.g. the Highway 401 rock cut). As such, we recommend that all foundation elements be located a minimum of 2 m, horizontal distance, from the edge of the rock cut. A Non Standard Special Provision specifying that the foundation elements be situated a minimum of 2 m from the edge of the rock cut should be included in the contract documents. A sample has been included in Appendix B of this report.

6.2.1.3 Construction Considerations

Where strip/spread footings are constructed, all loose or highly fractured bedrock within the footprint of the footings should be removed prior to placing concrete. The cleaned excavation base should be inspected by the Quality Verification Engineer, in accordance with SP599F01, prior to placement of the concrete to ensure that the base has been adequately cleaned and that the bedrock conditions as exposed at the founding level are consistent with design assumptions. The design of the strip or spread footings should be flexible enough to allow for some variation in the bedrock surface elevation and placement of a nominal thickness (+/- 100 mm) of mass concrete to raise the grade of footings to the founding level after exposing the bedrock and removing any loosened/highly fractured bedrock, if required. Footing areas should be protected and the concrete for the footings poured as soon as possible after excavation.

For strip/spread footings placed on fresh limestone bedrock or mass concrete, frost protection cover is not required.

6.2.2 Drilled Caissons

Drilled caisson wall foundations should be designed and constructed in accordance with MTO's Special Provision SP599F01. If caissons are used for support of the noise barrier wall, a diameter of between 0.6 and 0.9 m should be used. As specified in Section 7.03 of SP599F01, the depth of footings shall be determined by the Contractor in accordance with CAN/CSA-S6, Canadian Highway Bridge Design Code, Clause 12.5.7, based on the soil and bedrock conditions encountered along the proposed noise barrier wall alignments and the geotechnical soil and rock design parameters provided below and summarized in Table 1.



6.2.2.1 *Geotechnical Resistance*

Due to the relatively shallow overburden composed of a mix of stiff to very stiff cohesive materials and surficial fills, augered caissons socketed into rock should be assumed to derive their load carrying capacity from rock socket resistance only. The ultimate shaft resistance, R_s may be computed as follows:

$$R_s = \pi b_s L_s q_s$$

Where: b_s = Socket diameter (m);
 L_s = Length of socket (m);
 q_s = Average unit shear resistance along the socket.

An average factored unit shear resistance along the socket, q_s , for unweathered limestone of 1.0 MPa can be used for design. Resistance factors of 0.4 for compression and 0.3 for tension should be applied. The SLS value is not applicable as the limestone bedrock at the base of the caisson is considered to be an unyielding material for the axial loads imposed by the noise wall foundations.

A Non Standard Special Provision should be included in the contract document to alert the Contractor of the limited thickness of overburden, and the influence of this condition on design of the caisson foundations.

6.2.2.2 *Resistance to Lateral Forces*

Lateral forces and moments imposed on noise wall foundations by wind and, where applicable, retained earth, will be resisted by soil and rock in front of (and behind) the wall.

Given the limited thickness of soils, sloping ground and design frost depth, passive resistance provided by soils in front of the noise wall barrier will be minimal. Where sufficient soil is present, passive resistance provided by soils adjacent to the caissons may be computed using the passive earth pressure coefficients, K_p , provided in Table 1 following the text of the report. The stratigraphy presented is on a station by station basis and has been simplified for the purposes of the noise barrier wall foundation design. These values should be incorporated into Section 7.03 of SP599F01. The passive resistance in front of the caisson within the upper 1.5 m below ground surface should be neglected to account for frost action. In addition, for foundation design, full passive resistance will only be mobilized where the ground surface in front of or behind the caissons (within a zone equal to eight caisson diameters) is level. This condition may not be met in the case where the wall alignment is close to the existing rock cut. Where sloping ground is present adjacent to the noise barrier wall, the K_p values used in the calculation of passive resistance should be adjusted accordingly. The adjusted K_p value is to be applied to that portion of the caisson that is above the elevation of the ground surface in the area of sloping ground; below this elevation the full K_p value may be applied.

The factored lateral resistance provided by the limestone bedrock will depend on the type of lateral loading imposed. Purely lateral loading would result in a block slip failure along horizontal bedding planes, which should be computed as sliding resistance of a block of rock using the unit weight of the rock and angle of internal friction. Moments induced by wind loads and lateral earth pressures from retained soils would result in a stepped failure in the shape of an inverted triangular wedge, defined by:

$$\Psi_p = 45 - \Psi_f - \phi/2$$



- where: Ψ_p is the failure wedge angle (degrees) measured from horizontal sloping down towards the caisson;
- Ψ_f is the angle of the rock surface measured from horizontal (in this case, 0 for near-horizontal bedrock); and,
- ϕ is the internal angle of friction of the bedrock.

Depending on the offset from the caisson foundation to the rock cut and the depth of the caisson foundation, the inverted triangular wedge may be partially truncated by the rock cut.

The passive resistance provided by the rock socket adjacent to the caissons may be computed using the internal friction angles and unit weights provided in Table 1 following the text of the report. These values should be incorporated into Section 7.03 of SP599F01.

6.2.2.3 Construction Considerations

Caisson construction for the noise barrier wall foundations will generally require excavation through the fill, clayey silt, and sandy silt till deposit, into the underlying limestone bedrock. These soils could be susceptible to disturbance during caisson excavation and construction. The use of a temporary liner to advance the holes within overburden soils is recommended, in order to minimize caving and protect the integrity of the caisson excavations during drilling and concrete placement. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to advise the Contractor of this condition since it may affect the installation of the noise barrier wall foundations. A sample NSSP is provided in Appendix B.

The limestone bedrock will be resistant to auger advance. The use of churn drilling and/or coring will be required to form the socket for caissons installed in the rock. In addition, the Contractor's proposed excavation techniques should be able to accommodate removal or breaking up of boulders and/or other obstructions which may be present in the fill and native soils.

In accordance with MTO's SP 599F01, following construction the Quality Verification Engineer shall submit a Certificate of Conformance confirming that the noise barrier wall foundations have been constructed in general conformance with the contract documents.

6.2.3 Lateral Earth Pressures

The lateral earth pressures acting on the retained portion of the wall will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutment stems and retaining walls in accordance with the CHBDC:

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II but with less than 5 percent passing the 75 micron (#200) sieve should be used as backfill behind the walls. This granular fill should be placed in accordance with Ontario Provincial Standard Drawing (OPSD) 3121.150 and compacted in loose lifts not greater than 200 mm in thickness in accordance with OPSS 539.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains should be in accordance with OPSD 3190.100.



- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS 539. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case (a) in Figure C6.19 of the Commentary to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case (b) in Figure C6.19 of the Commentary to the CHBDC).
- For either Case (a) or Case (b) above, the following unfactored lateral earth pressure parameters may be used assuming the use of compacted granular fill material:

Soil Unit Weight:	Granular 'A'	Granular 'B' Type II
	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.7	3.7

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
 - Rotation of approximately 0.002 about the base of a vertical wall;
 - Horizontal translation of 0.001 times the height of the wall; or,
 - A combination of both.
- If the abutment support does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

It should be noted that these earth pressure coefficients assume level backfill. If the final design slopes differ, these parameters should be adjusted as in CHBDC C6.9.1 (e).

6.2.4 Excavation and Temporary Cut Slopes

If strip/spread footings are selected, excavations for the wall footings will extend through the existing surficial fill materials and native silty clay overburden soils to the underlying limestone bedrock. If caisson foundations are selected, some regrading of the site may be required to facilitate caisson drilling equipment access.

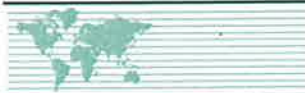
Temporary open cut slopes within the fill and overburden materials should be maintained no steeper than 1 horizontal to 1 vertical. Temporary shoring may be required if there is insufficient space for open cuts and space is restricted (e.g., adjacent to the rear fence line of private residences). The temporary excavation support



system should be designed and constructed in accordance with OPSS 539. The lateral movement of the temporary shoring system should meet Performance Level 2.

Excavations are not expected to penetrate the groundwater level which is anticipated to be below elevation 100 m, however surface water runoff should be expected and will need to be directed away from the excavations at all times. The appropriate Non Standard Special Provision (NSSP) should be included in the Contract Documents.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials at this site would be classified as Type 3 soils. The native silty clay and clayey silt soils would be considered Type 2 or 3 soils, depending on consistency or density, as applicable. The limestone bedrock is considered to be a Type 1 material.



7.0 CLOSURE

This report was prepared by Ms. Erin O'Neill, P.Eng., under the direction of the Project Manager, Mr. Michael Snow, P.Eng.. Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

Yours truly,

GOLDER ASSOCIATES LTD.

Erin O'Neill, P.Eng.
Geotechnical Engineer

Michael Snow, P.Eng.
Principal

Fintan Heffernan, P.Eng.
Designated MTO Contact



ESO/MSS/FJH/tm

n:\active\2008\1111\08-1111-0044 mrc hwy 401\reports\noise wall barrier report\08-1111-0044 montreal street noise wall rpt-001-final part a & b_17jan11.docx



TABLE 1
LATERAL PASSIVE RESISTANCE DESIGN PARAMETERS FOR NOISE WALL BARRIER³

Approx. Station	Reference Boreholes	Stratum	Depth or Elevation Interval (m)	Design Parameters						Depth to Water (m)
				c_u ¹	ϕ' ¹	δ_i	q_s	K_p ²	γ	
0 - 40 m	NW5 NW6	Very stiff silty clay/ clayey silt	Above elev. 99.5 m	120	33			3.4	19	Below elev. 100 m
		Limestone bedrock	Below elev. 99.5 m		35	35	1.0		26	
40 - 80 m	NW7 NW1	Very stiff silty clay/ clayey silt	Above elev. 100.5 m	120	33			3.4	19	
		Limestone bedrock	Below elev. 100.5 m		35	35	1.0		26	
80 - 140 m	NW2	Very stiff silty clay/ clayey silt	Above elev. 101.5 m	120	33			3.4	19	
		Sandy silt till	Btwn 101.5 and 101 m		35			3.7	21	
		Limestone bedrock	Below elev. 101 m		35	35	1.0		26	
140 - 190 m	NW3	Fill	Above elev. 102.5 m		28			2.8	18	
		Limestone bedrock	Below elev. 102.5 m		35	35	1.0		26	
190 - 240 m	NW4	Fill	Above elev. 102.5 m		28			2.8	18	
		Limestone bedrock	Below elev. 102.5 m		35	35	1.0		26	

Notes:

c_u = Undrained shear strength of soil (kPa);

ϕ' = Effective angle of friction in soil (degrees), internal angle of friction in rock (degrees);

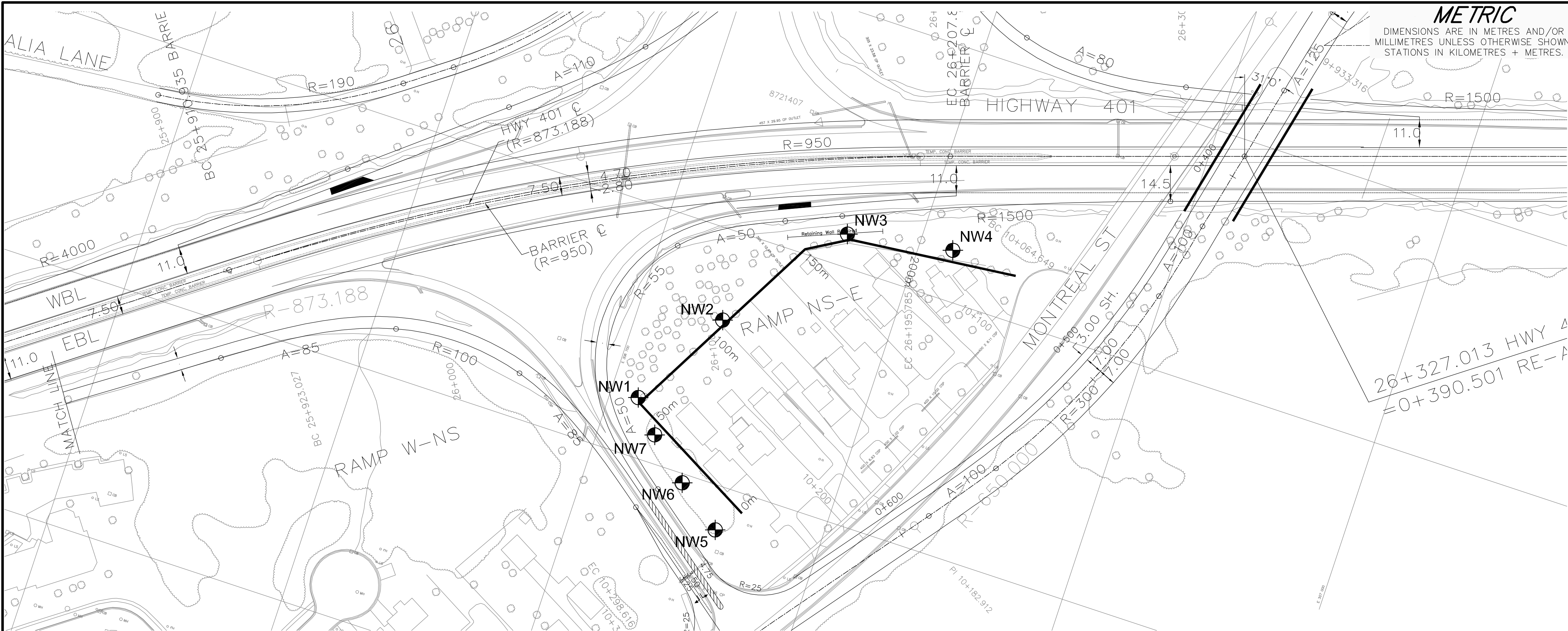
δ_i = interface friction angle between concrete and founding rock (degrees);

q_s = Average factored unit shear resistance along the socket for unweathered limestone (MPa).

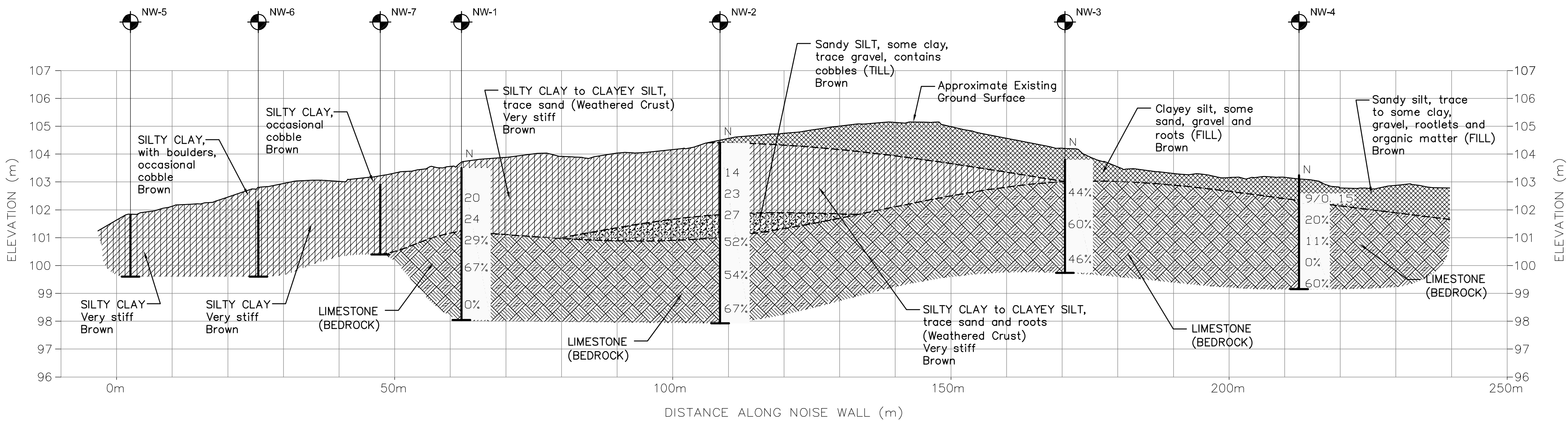
K_p = Coefficient of passive earth pressure; and,

γ = Bulk unit weight of soil (kN/cu. m).

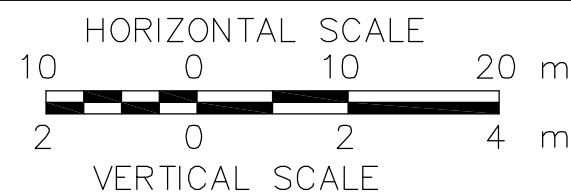
1. Where both c_u and ϕ' have been given for a specific stratum, the foundation design should be checked for both the undrained and the drained conditions, and the larger of the two calculated foundation depths shall govern. Internal angles of friction for the rock are given as a means of calculating the lateral resistance of the rock socket.
2. Passive earth pressure coefficient (K_p) values are provided for level ground in front of or behind caissons for a distance ≥ 8 diameters. Where sloping ground is present adjacent to the composite retaining/noise barrier wall, adjusted K_p values must be used in the foundation design.
3. The passive resistance in front of the caisson within the upper 1.5 m below ground surface should be neglected to account for frost action.



PLAN



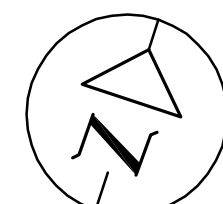
PROFILE ALONG NOISE WALL



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 78-99-00

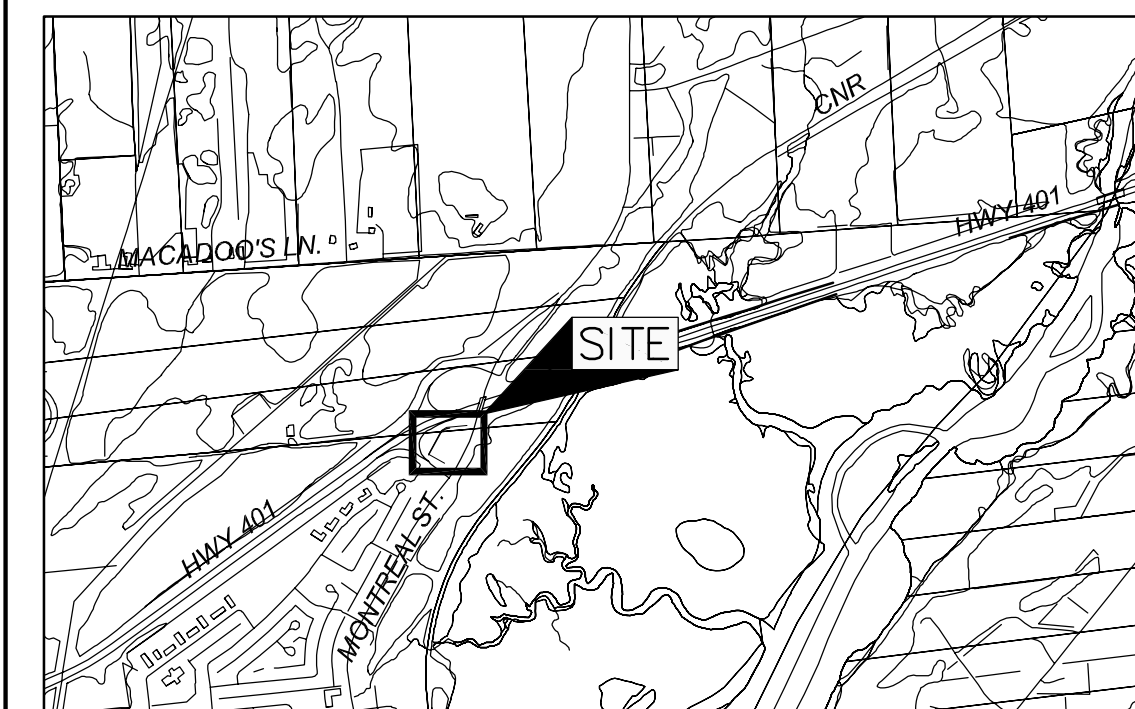


HIGHWAY 401
NOISE WALL BARRIER
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN



LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock quality designation

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
NW-1	103.5	4904023.6	306711.4
NW-2	104.4	4904064.1	306734.0
NW-3	103.8	4904113.0	306771.0
NW-4	103.2	4904120.2	306813.6
NW-5	101.8	4903982.5	306757.8
NW-6	102.3	4903996.4	306739.2
NW-7	102.9	4904011.4	306722.5

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

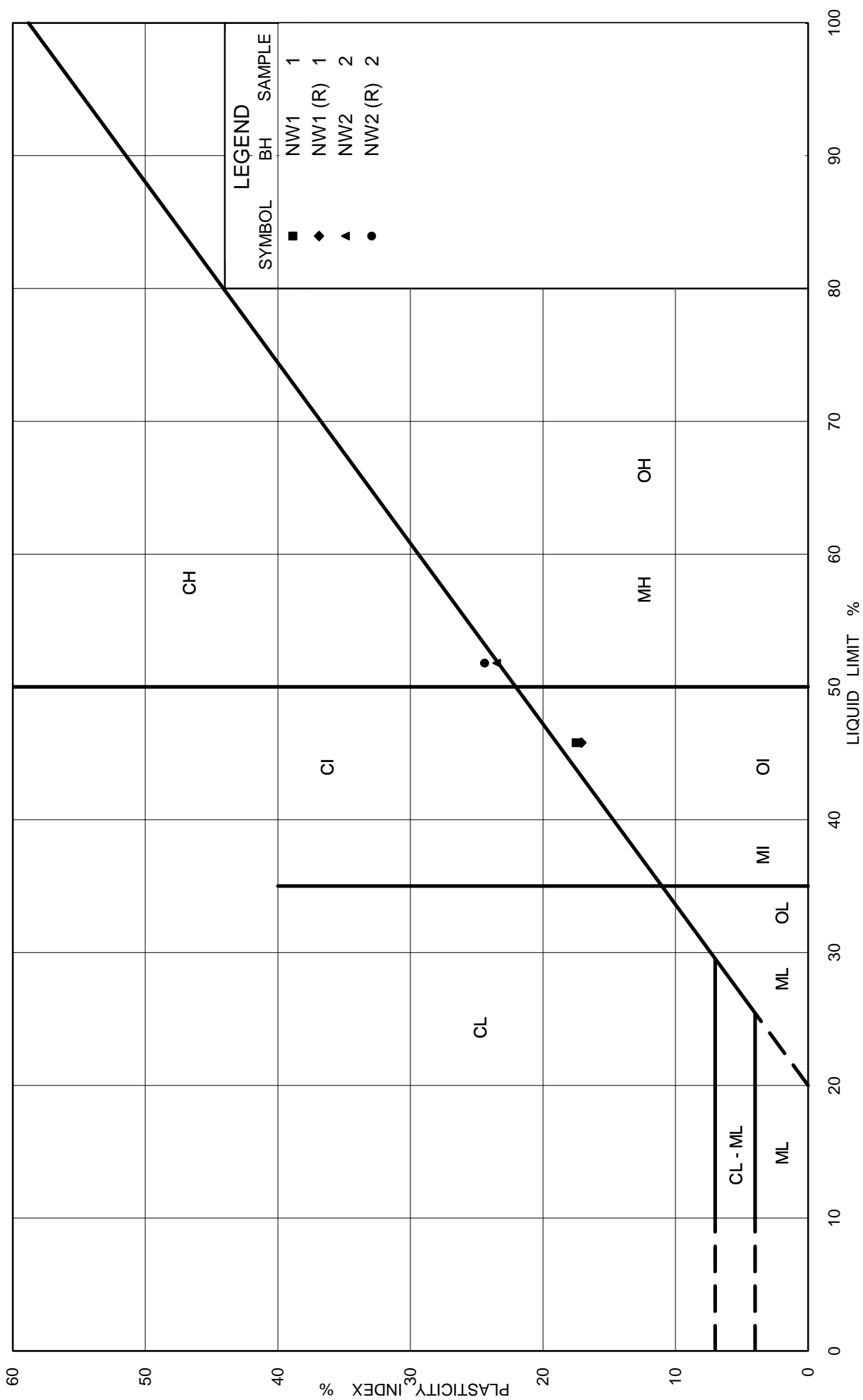
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

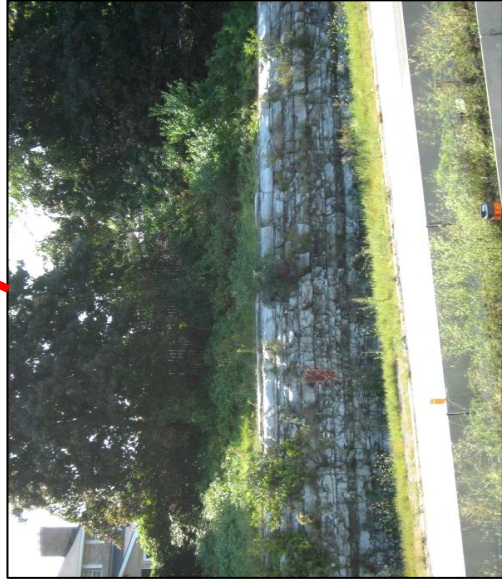
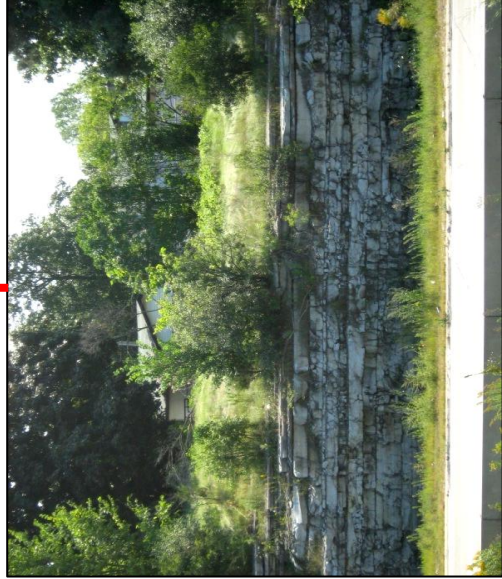
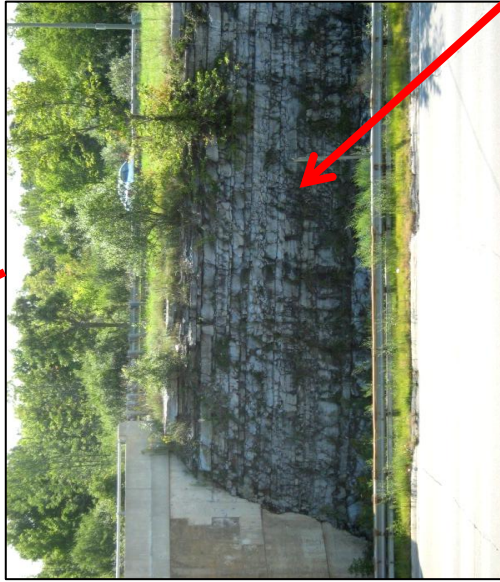
REFERENCE

Base plans provided in digital format by MRC (Drawing File No. "Plan-Base (BA).dwg", received Jan. 16, 2009 and "NS-E Property Plan and Profile.dwg", received on Mar. 8, 2010).

NO.	DATE	BY	REVISION
Geocres No. 31C-204			
HWY. 401	PROJECT NO. 08-1111-0044		DIST.
SUBM'D. E.O.	CHKD. MSS	DATE: 1/18/2011	SITE:
DRAWN: JM	CHKD. MSS	APPD. FJH	DWG. 1







Example of blocky
structure and
ravelling due to
ongoing freeze-thaw
processes

Dampness due to
seepage lower down
on face

View looking west from the south side of
the Hwy 401-Montreal Street Underpass.



Looking east along rock cut (east of NW3)



Looking east along rock cut (west of NW3)

3-4 m crest
height



Looking east along rock cut (west of NW3)

Minimal seepage
indicated by lack
of ice build-up
along rock cut

Example of blocky
structure

Overburden slope
between top of rock
and noise wall barrier

**SUMMARY OF LABORATORY COMPRESSIVE STRENGTH
MEASUREMENTS**

FIGURE 5

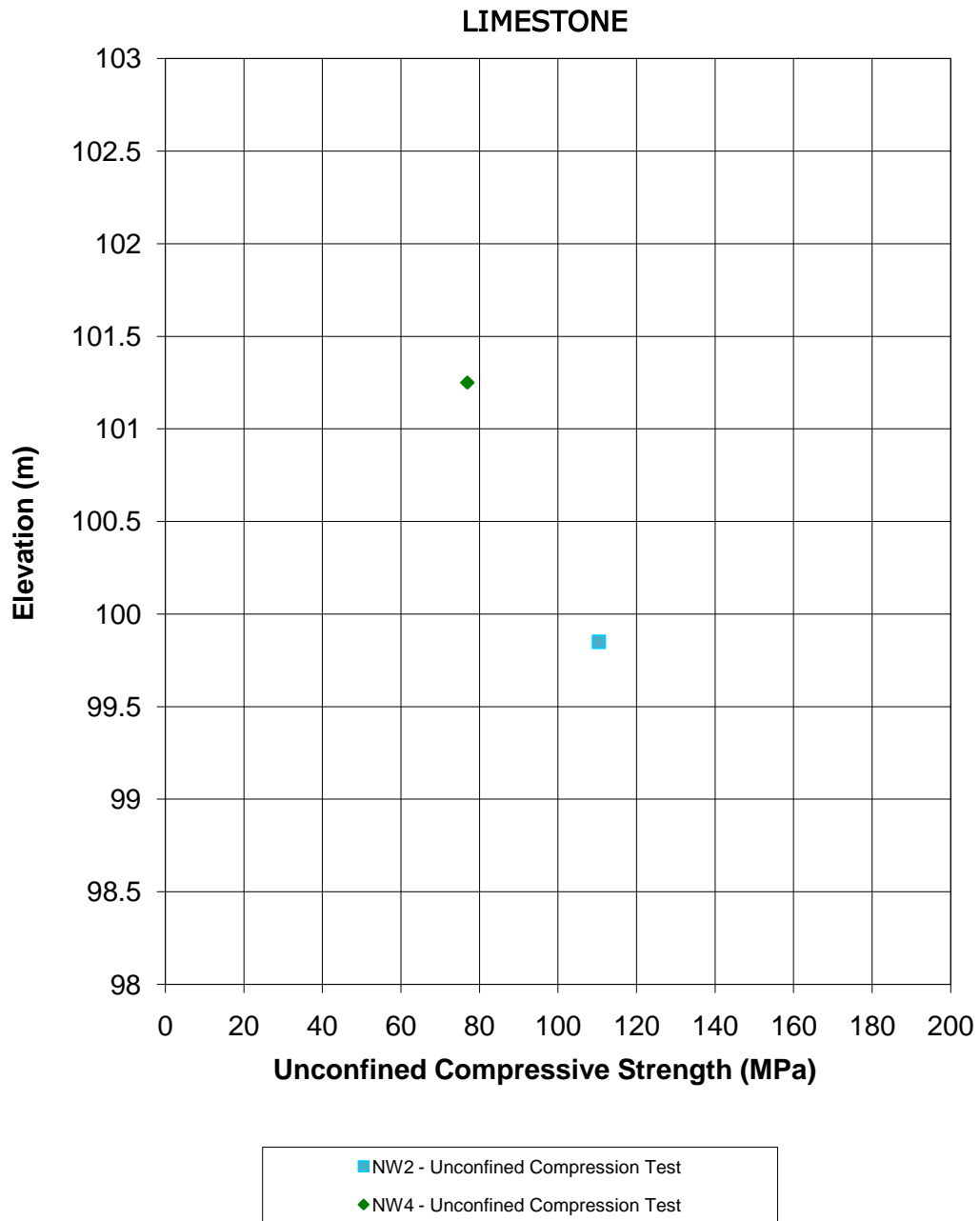




TABLE 2

EVALUATION OF NOISE WALL BARRIER AND RETAINING WALL
FOUNDATIONS/CONSTRUCTION ALTERNATIVES
MONTREAL STREET, HIGHWAY 401, KINGSTON
G.W.P. 78-99-00

Foundation Option	Feasibility	Advantages	Disadvantages	Risks/Consequences
Spread footings supported on limestone bedrock	<ul style="list-style-type: none"> Feasible where bedrock is less than 3 m below ground surface 	<ul style="list-style-type: none"> May be less expensive solution in areas with very shallow bedrock (within about 1 m of existing ground surface) 	<ul style="list-style-type: none"> May not be compatible with some proprietary noise wall designs Construction may be difficult due to variability in depth to bedrock and proximity of rock cut May not provide sufficient resistance to wind loads for noise wall barriers 	<ul style="list-style-type: none"> Construction becomes difficult and costs can escalate quickly in sections where bedrock is greater than 1.5 m below ground surface Potential incompatibility with proprietary noise wall designs
Drilled caissons socketed into limestone bedrock	<ul style="list-style-type: none"> Preferred technical solution for noise wall and retaining wall foundations 	<ul style="list-style-type: none"> Minimizes excavation Rapid construction Provides best resistance to wind loads Likely to be compatible with several proprietary noise wall designs Easily adaptable to varying bedrock depth 	<ul style="list-style-type: none"> If liner not used, sidewall collapse could result in poor bearing surface if the base is not cleaned (particularly difficult for small diameter caissons) Coring or churn drilling will be required to form socket in medium strong bedrock 	<ul style="list-style-type: none"> Some settlement if caisson constructed without a liner or socket not adequately cleaned prior to pouring concrete



APPENDIX A

List of Abbreviations and Symbols
Rock Description Terminology
Record of Borehole Sheets

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE		III. SOIL DESCRIPTION	
AS	Auger sample	(a)	Cohesionless Soils
BS	Block sample		
CS	Chunk sample	Density Index	N
DO	Drive open	(Relative Density)	Blows/300 mm
DS	Denison type sample		Or Blows/ft.
FS	Foil sample	Very loose	0 to 4
RC	Rock core	Loose	4 to 10
SC	Soil core	Compact	10 to 30
ST	Slotted tube	Dense	30 to 50
TO	Thin-walled, open	Very dense	over 50
TP	Thin-walled, piston		
WS	Wash sample	(b)	Cohesive Soils
DT	Dual Tube sample	Consistency	C _n or S _u
II. PENETRATION RESISTANCE			
Standard Penetration Resistance (SPT), N:			
The number of blows by a 63.5 kg. (140 lb.)			
hammer dropped 760 mm (30 in.) required			
to drive a 50 mm (2 in.) drive open			
Sampler for a distance of 300 mm (12 in.)			
DD- Diamond Drilling			
Dynamic Penetration Resistance; N_d:			
The number of blows by a 63.5 kg (140 lb.)			
hammer dropped 760 mm (30 in.) to drive			
Uncased a 50 mm (2 in.) diameter, 60° cone			
attached to "A" size drill rods for a distance			
of 300 mm (12 in.).			
PH:	Sampler advanced by hydraulic pressure		
PM:	Sampler advanced by manual pressure		
WH:	Sampler advanced by static weight of hammer		
WR:	Sampler advanced by weight of sampler and rod		
Peizo-Cone Penetration Test (CPT):			
An electronic cone penetrometer with			
a 60° conical tip and a projected end area			
of 10 cm ² pushed through ground			
at a penetration rate of 2 cm/s. Measurements			
of tip resistance (Q _t), porewater pressure			
(PWP) and friction along a sleeve are recorded			
Electronically at 25 mm penetration intervals.			
		IV. SOIL TESTS	
		w	water content
		w _p	plastic limited
		w _l	liquid limit
		C	consolidaiton (oedometer) test
		CHEM	chemical analysis (refer to text)
		CID	consolidated isotropically drained triaxial test ¹
		CIU	consolidated isotropically undrained triaxial test ¹
			with porewater pressure measurement ¹
		D _R	relative density (specific gravity, G _s)
		DS	direct shear test
		M	sieve analysis for particle size
		MH	combined sieve and hydrometer (H) analysis
		MPC	modified Proctor compaction test
		SPC	standard Proctor compaction test
		OC	organic content test
		SO ₄	concentration of water-soluble sulphates
		UC	unconfined compression test
		UU	unconsolidated undrained triaxial test
		V	field vane test (LV-laboratory vane test)
		γ	unit weight

Note:

1. Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	= 3.1416
$\ln x$,	natural logarithm of x
$\log_{10} x$ or $\log x$,	logarithm of x to base 10
g	Acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma'$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1 \sigma_2 \sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is p . Unit weight symbol is γ where $\gamma = pg$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity Index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p)/I_p$
I_c	consistency index = $(w_l - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e)/(e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio = σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p \tau_r$	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi=0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = $(\text{Compressive strength})/2$

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly Weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	>2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	<6 mm

JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	>3 m
Wide	1 – 3 m
Moderately close	0.3 – 1 m
Close	50 – 300 mm
Very close	<50 mm

GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	>60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns - 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	<2 microns

Note: *Grains >60 microns diameter are visible to the naked eye.

O:\ Templates\Rock Description Terminology

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B -	Bedding	Ca -	Calcite
FO -	Foliation/Schistosity	P -	Polished
CL -	Cleavage	S -	Slickensided
SH -	Shear Plane/Zone	SM -	Smooth
VN -	Vein	R -	Ridged/Rough
F -	Fault	ST -	Stepped
CO -	Contact	PL -	Planar
J -	Joint	FL -	Flexured
FR -	Fracture	UE -	Uneven
MF -	Mechanical	W -	Wavy
A -	Angular	C -	Curved
BP -	Bedding Plane	H -	Hackly
BL -	Blast Induced	SL -	Sludge Coated
-	Parallel To	TCA -	To Core Axis
⊥ -	Perpendicular To	STR -	Stress Induced

PROJECT 08-1111-0044		RECORD OF BOREHOLE No NW-1		1 OF 1 METRIC	
G.W.P. 78-99-01		LOCATION N 4904023.6 ; E 306711.4		ORIGINATED BY DWM	
DIST HWY 401		BOREHOLE TYPE CME 55, Power Auger, 200mm Diam. Hollow Stem		COMPILED BY JM	
DATUM Geodetic		DATE February 16, 2010		CHECKED BY EO	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
103.5	GROUND SURFACE													
0.0	TOPSOIL													
0.1	SILTY CLAY to CLAYEY SILT, trace sand (Weathered Crust) Very stiff Brown Damp		1	SS	20									
			2	SS	24									
101.2	LIMESTONE (BEDROCK) Fresh Thinly to medium bedded Strong Grey		C1	NQ RC	DD									
2.3	- Soil-filled seam from 4.0 m to 4.1 m depth		C2	NQ RC	DD									
99.2	LIMESTONE (BEDROCK) Fresh to slightly weathered Strong Grey		C3	NQ RC	DD									
4.3	- Soil-filled joint at 5.3 m depth													
98.0	Note: Bedrock cored between 2.3 m and 5.5 m depth. For bedrock coring details refer to Record of Drillhole NW-1.													
5.5	End of Borehole													
	Note: Open borehole dry upon completion of drilling													

MIS-MTO 001 08-1111-0044 GPJ GAL-MISS GDT 1/18/11 DD

PROJECT 08-1111-0044				RECORD OF BOREHOLE No NW-2				1 OF 1 METRIC							
G.W.P. 78-99-01				LOCATION N 4904064.1 ; E 306734.0				ORIGINATED BY DWM							
DIST HWY 401				BOREHOLE TYPE CME 55, Power Auger, 200mm Diam, Hollow Stem				COMPILED BY JM							
DATUM Geodetic				DATE February 16, 2010				CHECKED BY EO							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60					
104.4	GROUND SURFACE														
0.0	TOPSOIL														
0.2	SILTY CLAY to CLAYEY SILT, trace sand and roots (Weathered Crust) Very stiff Brown Damp - Contains roots in upper 1.5 m		1	SS	14										
			2	SS	23										
101.8			3	SS	27										
2.6	Sandy SILT, some clay, trace gravel, contains cobbles (TILL) Brown Damp														
101.0															
3.4	LIMESTONE (BEDROCK) Fresh to slightly weathered Thinly to medium bedded Strong Grey - Core diskings and friable from 4.6 m to 4.9 m depth Note: Bedrock cored between 3.4 m and 6.5 m depth. For bedrock coring details refer to Record of Drillhole NW-2.		C1	NQ RC	DD										
			C2	NQ RC	DD										
			C3	NQ RC	DD										
97.9															
6.5	End of Borehole Note: Open borehole dry upon completion of drilling														

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PROJECT: 08-1111-0044

RECORD OF DRILLHOLE: NW-2

SHEET 1 OF 1

LOCATION: N 4904064.1 ; E 306734.0

DRILLING DATE: February 16, 2010

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG:

DRILLING CONTRACTOR:

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	COLOUR (m/min)	FR/FX-FRACTURE F-FAULT			SM-SMOOTH			FL-FLEXURED			BC-BROKEN CORE			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION			
									CL-CLEAVAGE			J-JOINT			R-ROUGH			UE-UNEVEN					MB-MECH. BREAK		
									SH-SHEAR			P-POLISHED			ST-STEPPED			W-WAVY					B-BEDDING		
		GROUND SURFACE		101.00																					
4		LIMESTONE (BEDROCK) Fresh to slightly weathered Thinly to medium bedded Strong Grey - Core diskings and friable from 4.6 m to 4.9 m depth		3.40	1																				
5					2																				
6					3																				
7		End of Borehole Note: Open borehole dry upon completion of drilling		97.90 6.50																					
8																									
9																									
10																									
11																									
12																									
13																									
14																									
15																									
16																									
17																									
18																									

DEPTH SCALE

1 : 75



LOGGED: DWM

CHECKED: EO

MIS-RCK 001 08-1111-0044 (ROCK) GPJ GAL-MISS GDT 1/18/11 DD

PROJECT 08-1111-0044		RECORD OF BOREHOLE No NW-3				1 OF 1 METRIC							
G.W.P. 78-99-01		LOCATION N 4904113.0 ; E 306771.0				ORIGINATED BY DWM							
DIST HWY 401		BOREHOLE TYPE CME 55, Power Auger, 200mm Diam. Hollow Stem				COMPILED BY JM							
DATUM Geodetic		DATE February 18, 2010				CHECKED BY EO							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	25 50 75	γ	GR SA SI CL	
103.8 0.0	GROUND SURFACE Clayey silt, some sand, gravel and roots (FILL) Brown												
103.0 0.8	LIMESTONE (BEDROCK) Fresh to slightly weathered Medium to thinly bedded Strong Grey - Core diskings and friable from 2.7 m to 3.0 m depth Note: Bedrock cored between 0.8 m and 4.1 m depth. For bedrock coring details refer to Record of Drillhole NW-3.		C1	NQ RC	DD		103						
			C2	NQ RC	DD		102						
			C3	NQ RC	DD		101						
99.7 4.1	End of Borehole Note: Open borehole dry upon completion of drilling						100						

PROJECT: 08-1111-0044

RECORD OF DRILLHOLE: NW-3

SHEET 1 OF 1

LOCATION: N 4904113.0 ; E 306771.0

DRILLING DATE: February 18, 2010

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR:

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	RECOVERY	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE		103.00										
1		LIMESTONE (BEDROCK) Fresh to slightly weathered Medium to thinly bedded Strong Grey		0.80	1									
2		- Core diskings and friable from 2.7 m to 3.0 m depth			2									
3					3									
4		End of Borehole		99.70 4.10										
5		Note: Open borehole dry upon completion of drilling												
6														
7														
8														
9														
10														
11														
12														
13														
14														
15														

DEPTH SCALE

1 : 75



LOGGED: DWM

CHECKED: EO


MIS-RCK-001 08-1111-0044 (ROCK) GPJ GAL-MISS GDT 1/18/11 DD

PROJECT 08-1111-0044			RECORD OF BOREHOLE No NW-4			1 OF 1 METRIC																	
G.W.P. 78-99-01			LOCATION N 4904120.2 ; E 306813.6			ORIGINATED BY DWM																	
DIST HWY 401			BOREHOLE TYPE CME 55, Power Auger, 200mm Diam. Hollow Stem			COMPILED BY JM																	
DATUM Geodetic			DATE February 17, 2010			CHECKED BY EO																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20	40	60	80	100	W _p	W	W _L	γ	GR	SA	SI	CL		
103.2		GROUND SURFACE																					
102.3	0.9	TOPSOIL Sandy silt, trace to some clay, gravel, rootlets and organic matter (FILL) Brown		1	SS	9/0.15		103															
		LIMESTONE (BEDROCK) Fresh to slightly weathered Thinly to medium bedded Strong Grey		C1	NQ RC	DD		102															
		- Soil-infilled seams from 1.4 m to 1.5 m, 2.1 m to 2.2, 2.6 m to 2.7 m, and at 4.0m depths		C2	NQ RC	DD		101															
		Note: Bedrock cored between 0.9 m and 4.1 m depth. For bedrock coring details refer to Record of Drillhole NW-4.		C3	NQ RC	DD		100															
				C4	NQ RC	DD																	
99.2	4.1	End of Borehole																					
		Note: Open borehole dry upon completion of drilling																					

PROJECT 08-1111-0044		RECORD OF BOREHOLE No NW-5				1 OF 1 METRIC										
G.W.P. 78-99-01		LOCATION N 4903982.5 ; E 306757.8				ORIGINATED BY RS										
DIST HWY 401		BOREHOLE TYPE Power Auger, 200mm Diam, Solid Stern				COMPILED BY JM										
DATUM Geodetic		DATE March 11, 2010				CHECKED BY EQ										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) W _p — W — W _L			γ	GR SA SI CL	
101.8	GROUND SURFACE							20 40 60 80 100								
0.0	Clay (TOPSOIL) Brown							20 40 60 80 100								
0.1	SILTY CLAY Very stiff Brown Moist						101									
99.6							100									
2.2	End of Augerhole Auger Refusal Inferred top of bedrock															

MIS-MTO 001 08-1111-0044.GPJ GAL-MISS.GDT 1/16/11 DD

PROJECT 08-1111-0044		RECORD OF BOREHOLE No NW-6		1 OF 1 METRIC	
G.W.P. 78-99-01		LOCATION N 4903996.4 ; E 306739.2		ORIGINATED BY RS	
DIST HWY 401		BOREHOLE TYPE Power Auger, 200mm Diam. Solid Stem		COMPILED BY JM	
DATUM Geodetic		DATE March 11, 2010		CHECKED BY EO	

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	20 40 60 80 100			20 40 60 80 100	W _P W W _L						
102.3	GROUND SURFACE															
0.0	SILTY CLAY, with boulders, occasional cobble															
0.2	Brown															
	SILTY CLAY Very stiff Brown Moist															
99.6	End of Augerhole Auger Refusal Inferred top of bedrock															
2.7																

PROJECT <u>08-1111-0044</u>		RECORD OF BOREHOLE No NW-7				1 OF 1 METRIC						
G.W.P. <u>78-99-01</u>		LOCATION <u>N 4904011.4 ;E 306722.5</u>				ORIGINATED BY <u>RS</u>						
DIST <u>HWY 401</u>		BOREHOLE TYPE <u>Power Auger, 200mm Diam. Solid Stem</u>				COMPILED BY <u>JM</u>						
DATUM <u>Geodetic</u>		DATE <u>March 11, 2010</u>				CHECKED BY <u>EO</u>						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L			WATER CONTENT (%) 25 50 75
102.9	GROUND SURFACE											
0.0	SILTY CLAY, occasional cobble											
0.2	Brown											
	SILTY CLAY											
	Very stiff											
	Brown											
	Moist											
100.4												
2.5	End of Augerhole											
	Auger Refusal											
	Inferred top of bedrock											



APPENDIX B

Non-Standard Special Provisions

**CONTROL OF OVERBURDEN SOILS DURING NOISE WALL BARRIER
FOUNDATION INSTALLATION - Item No.**

Special Provision

Excavations for the noise wall barrier support foundations will be advanced through fill. Appropriate construction procedures and equipment (e.g., temporary or permanent liners) are recommended to minimize ground loss during drilling and concrete placement.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

DOWELS INTO ROCK – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

The work for the above noted tender item shall be in accordance with OPSS 904, including all special provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the structure footings.

1.2 Instructions to Contractor

- 1.2.1 These instructions are to be read in conjunction with the Contract Drawings.
- 1.2.2 A total of 1 test Dowels into Rock are required for the Dowels into Rock at each structure footing.
- 1.2.3 Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

1.3 Qualifications

- 1.3.1 **Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock:** All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.
- 1.3.2 **Qualifications of the Quality Verification Engineer:** A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.
- 1.3.3 **Qualifications of the Design Engineer:** A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer

licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

1.4 Responsibilities of the Contractor

- 1.4.1 The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.
- 1.4.2 The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.
- 1.4.3 The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.
- 1.4.4 The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 1.6.

1.5 Definitions

- 1.5.1 Dowels into Rock: reinforcing steel bar and non-shrink grout.
- 1.5.2 Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.
- 1.5.3 Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

1.6 Submissions and Working Drawings

- 1.6.1 Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

- 1.6.2 The Contractor shall submit Working Drawings to the Contract Administrator as follows:
- All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.
 - All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.
- 1.6.3 Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.
- 1.6.4 Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.
- 1.6.5 Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:
- Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.
 - Test results verifying the 28 day strength of non-shrink grout.
 - The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
 - The procedures to verify hole length. Records of measurements that verify the hole length.
 - Records of all drilling procedures, rock conditions encountered, and installation times.
 - Test procedures for Dowels into Rock.
 - Drawings and design calculations for a suitable reaction system for the applied test loads.

- Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.
- Drawings and details for reference system arrangement.
- Current calibration curves shall be provided for all gauges.
- Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
- Remedial measures for unacceptable stressing results.

1.7 Subsurface Conditions

- 1.7.1 Soils, rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

2.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.10.35 non-shrink grout.

The Contractor shall provide the following information from the manufacturer for non-shrink grout:

- Data sheets for the non-shrink grout,
- installation procedures

3.0 EQUIPMENT

3.1 General

- 3.1.1 All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.
- 3.1.2 The equipment shall not cause damage to the reinforcing steel bars.

4.0 INSTALLATION

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

4.1 Construction of Holes

- 4.1.1 The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.
- 4.1.2 The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.
- 4.1.3 At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

4.2 Installation of Reinforcing Steel Bar

- 4.2.1 Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.
- 4.2.2 Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.
- 4.2.3 Dowels shall extend through the tremie concrete for the footing and into sound bedrock.
- 4.2.4 Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

4.3 Grout

- 4.3.1 The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.
- 4.3.2 The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.
- 4.3.3 Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

5.0 TESTING REQUIREMENTS

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

5.1 General Testing Requirements

- 5.1.1 Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.
- 5.1.2 The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.
- 5.1.3 The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the each structure location.
- 5.1.4 The Contractor shall submit Working Drawings that include proposed procedures for testing of the dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.
- 5.1.5 The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.
- 5.1.6 The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.
- 5.1.7 The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

5.2 Testing Location

- 5.2.1 The Contractor shall remove all loose rock down to sound bedrock at the test location.

5.2.2 The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator.

5.2.3 If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

5.3 Testing Equipment

5.3.1 The dowels into rock will be carried out generally in accordance with the prevailing requirements of A.S.T.M. (Designation D1143-81) superseded where applicable by the procedures specified in this document.

5.3.2 The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

5.3.3 The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:

- The beams shall be independently supported with the support firmly embedded in the ground.
- The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.
- Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

5.3.4 The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

5.4 Testing for Dowels Into Rock, and Report

5.4.1 At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall

submit Working Drawings that include the above noted records to the Contract Administrator.

- 5.4.2 Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.3 Jacks used for reinforcing steel bars shall have a minimum ram dimension of 153 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.4 Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

5.5 Testing Loading

- 5.5.1 The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the specified test load. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.
- 5.5.2 Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

5.6 Acceptance Criteria

- 5.6.1 The following acceptance criteria apply:

The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at each structure location.

Tests for Dowels into Rock shall have a capacity of at least [insert value] kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

6.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.