REPORT

December 2012

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Winston Churchill Boulevard Underpass Highway 401 Widening from Credit River to Trafalgar Road, Regional Municipalities of Peel and Halton, W.O. 07-20021

Submitted to:
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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
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HIGHWAY 401 WIDENING
FROM EAST OF THE CREDIT RIVER TO TRAFALGAR ROAD
REGIONAL MUNICIPALITIES OF PEEL AND HALTON
W.O. 07-20021

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future widening of Highway 401 from east of the Credit River in the Regional Municipality of Peel to Trafalgar Road (approximately 9 km), in the Regional Municipality of Halton, Ontario.

This report addresses the results of the assessment for the proposed replacement of or modification to the existing Winston Churchill Boulevard underpass structure.

The terms of reference and scope of work for the foundation engineering services are outlined in Section 5.8 of MTO's Request for Proposal (RFP) for Assignment No. 2008-E-0015 dated February 2010, and in Section 5.8 of the *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The Winston Churchill Boulevard underpass is located at the intersection of Highway 401 and Winston Churchill Boulevard in the Regional Municipality of Peel, Ontario. The existing underpass consists of a 90 m long two-span six-lane structure, with a deck width (including both northbound and southbound lanes) of about 33 m, constructed in 1986. Based on the General Arrangement (GA) drawings for the existing structure (MTO GEOCRES No. 30M12-145), it is anticipated that the existing abutments and piers are supported on driven steel H-piles.

In general, the terrain in this area is relatively flat, with the natural ground surface in the immediate vicinity of the structure site at about Elevation 208 m to 209 m (682 ft. to 686 ft.). As indicated in the GA drawings for the existing structure, Highway 401 has been constructed in an approximately 3.0 m to 3.7 m deep cut at the location of the underpass. From the available information, the existing Highway 401 pavement grade at the underpass is between about Elevation 205.5 m and 206 m.

The Winston Churchill Boulevard underpass has been constructed on embankment fill that is about 5 m to 6 m in height relative to the original ground surface; the pavement deck of the Winston Churchill Boulevard underpass is at about Elevation 214.0 m at the central pier location, and decreases to Elevation 213.7 m at the locations of each of the underpass abutments. The existing Winston Churchill Boulevard embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V).

3.0 INVESTIGATION PROCEDURES

A geotechnical investigation has previously been carried out at the site from October 1979 to May 1980 (MTO GEOCRES No. 30M12-145). The investigation consisted of seven boreholes (designated as Borehole No. 1 to No. 7) advanced to depths ranging between 12.5 m and 36.4 m below the ground surface. The borehole records associated with this investigation are provided in Appendix A. All of the boreholes were advanced on the west side of the original Winston Churchill Boulevard underpass which has subsequently been removed; Boreholes No. 1, 2, and 6 were advanced through the median of Highway 401. The remaining four boreholes (Boreholes No. 3, 4, 5, and 7) were advanced at the locations of the proposed abutments at the time of the investigation. The previous investigation report in its entirety is contained in Appendix A.

The borehole locations (referenced to MTM NAD83 co-ordinate system – note that these have been converted from NAD27 coordinate system as shown on the 1979-1980 borehole records) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are indicated on Drawings 1 and 2. The original drawings provided as part of GEOCRES No. 30M12-145 are included in Appendix A.

Borehole No.	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
1	4,828,976.1	281,569.3	205.9	12.5
2	4,828,975.2	281,556.9	205.9	12.5
3	4,829,004.7	281,516.7	208.5	14.2
4	4,828,937.2	281,573.2	208.6	15.4
5	4,828,940.2	281,605.0	212.0	30.7
6	4,828,974.7	281,540.3	205.9	29.0
7	4,829,009.9	281,545.3	211.7	36.4

Soil samples were obtained in the boreholes using a split-spoon sampler. A dynamic cone penetration test (DCPT) was also conducted adjacent to Borehole No. 2 as part of the original investigation. Rock coring was conducted in Borehole No. 7 to a depth of about 1.0 m below the bedrock surface.

The groundwater conditions were observed in the open boreholes following the drilling operations. No standpipe piezometers were installed as part of the field investigation.

Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples collected during the field program in 1979 and 1980; these test results are included in the geotechnical report in Appendix A.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the Peel Plain physiographic region of the west St. Lawrence Lowlands, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984).

The Peel Plain physiographic region covers the central portions of the Regional Municipalities of York, Peel and Halton. The general topography of this region consists of level to gently rolling terrain, sloping gradually southward toward Lake Ontario with predominate land use being agricultural. A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till, which is mapped in this area as the Halton Till, typically consists of clayey silt to silty clay, with occasional sand to silt zones. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial meltwater ponds scattered throughout the Peel Plain and concentrated near river valleys. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay. The study area, in the western portion of the Peel Plain, is underlain by red shale of the Queenston Formation.

4.2 Subsurface Conditions

As noted above, a previous geotechnical investigation (MTO GEOCRES No. 30M12-145) was carried out by MTO at the site in 1979 and 1980 and forms the basis for this preliminary design report; no additional boreholes were completed by Golder as part of the current assignment. The borehole locations, ground surface elevations and interpreted stratigraphic conditions from the 1979/1980 investigation are shown on Drawings 1 and 2. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are indicated on the borehole records from the previous investigation, attached in Appendix A. For discussion purposes, the results of the Atterberg limits testing completed during the geotechnical investigation have been interpreted from the borehole records and are plotted on Figures B1 to B5 in Appendix B of this report.

The stratigraphic boundaries shown on the borehole records, and on the interpreted stratigraphic sections and profiles on the drawings, are inferred from non-continuous sampling and therefore represent transitions between soil types rather than exact planes of geological change. The subsoil conditions may vary between and beyond the borehole locations.

In summary, the subsoil conditions encountered at the site consist predominantly of clayey silt till, which is underlain by shale bedrock. Granular materials and clayey silt fill were also encountered overlying the till in several of the boreholes. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

In Boreholes No. 1, 2 and 6, which were advanced within the median of Highway 401, granular fill was encountered to a depth of 1.1 m (Elevation 204.9 m to 204.8 m). In the two boreholes advanced on the embankments adjacent to Winston Churchill Boulevard, clayey silt fill was encountered to a depth of about 3.7 m (Elevation 208.3 m to 208.1 m).

Atterberg limits tests conducted on two selected samples of the clayey silt fill material measured plastic limits of about 13 per cent and 16 per cent, liquid limits of about 27 per cent and 30 per cent, and plasticity indices of about 14 per cent. These results, which have been interpreted from the 1979/1980 borehole logs and are plotted on a plasticity chart on Figure B1, confirm that the fill is typically comprised of clayey silt of low plasticity (according to the January 1980 *MTO Soil Classification* document), although it is shown on the 1979/1980 borehole records as silty clay fill.

The measured SPT "N" values in the clayey silt fill range from 4 blows to 11 blows per 0.3 m of penetration, suggestive of a firm to stiff consistency.

4.2.2 Clayey Silt Till

A glacial till deposit was encountered in each of the boreholes. The till was encountered surficially in Borehole Nos. 3 and 4; below the granular fill in Borehole Nos. 1, 2, and 6; and below the clayey silt fill in Borehole Nos. 5 and 7. The till was fully penetrated in Borehole Nos. 6 and 7, with the deposit base encountered at depths of

27.7 m and 33.8 m (Elevation 178.2 m and 177.9 m). The remaining five boreholes were terminated in the till deposit.

The till deposit is predominantly comprised of clayey silt with sand and trace gravel, containing layers / pockets of sand and silt at varying depths throughout the deposit. Thirteen grain size distribution tests were conducted on samples of the till. An envelope of the test results for eleven of the samples, plus individual test results for two samples from zones containing sand/silt layers, are included in the 1979/1980 geotechnical report in Appendix A.

Atterberg limits testing carried out on eighteen selected samples of the till deposit measured plastic limits between 8 per cent and 16 per cent, liquid limits between 15 per cent and 27 per cent and plasticity indices between 3 per cent and 15 per cent. These results, which are plotted on the plasticity charts on Figures B2 to B5, confirm that the till deposit consists predominantly of clayey silt of low plasticity (according to the 1980 *MTO Soil Classification* document), although it is classified on the 1979/1980 borehole records as silty clay till. It is noted that one tested sample from Borehole 6 would be classified as a slightly plastic sandy silt, based on a plasticity index of 3 per cent.

The measured SPT "N" values within the till deposit range from 4 blows to greater than 100 blows per 0.3 m of penetration, suggestive of a firm to hard (but typically very stiff to hard) consistency.

4.2.3 Shale Bedrock

Shale bedrock was encountered below the till deposit in Borehole Nos. 6 and 7, at depths of about 27.7 m and 33.8 m (Elevation 178.2 m and 177.9 m). The shale bedrock is of the Queenston Formation, based on bedrock mapping for this area.

The bedrock was penetrated for 1.2 m by augering and split-spoon sampling in Borehole No. 6, with a measured SPT "N" value of 150 blows per 0.05 m of penetration; the bedrock at this location was described as weathered on the borehole record. The shale bedrock was cored in Borehole No. 7 using BXL rock core; a core recovery of 85 per cent was obtained from 0.9 m of coring.

4.3 Groundwater Conditions

Details of the water conditions and levels observed in the open boreholes at the time of drilling are summarized on the borehole records contained in the geotechnical report, in Appendix A. The overnight groundwater level in Borehole No. 3 was recorded at a depth of about 10.5 m (Elevation 198.0 m). Localized seepage was also noted in Borehole No. 1 at a depth of about 1.5 m (Elevation 204.4 m).

Based on the transition from brown to grey as noted on the records for Borehole Nos. 1 to 4, it is anticipated that the stabilized groundwater level is closer to Elevation 204 m (670 ft.), which is about 1 m to 2 m below the Highway 401 cut grade at this site. Groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year. In addition, "perched" groundwater may be encountered at the base of the granular fill as encountered in Boreholes No. 1, 2, and 6.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Nick La Posta, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Ty Garde, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
WINSTON CHURCHILL BOULEVARD UNDERPASS
HIGHWAY 401 WIDENING
FROM EAST OF THE CREDIT RIVER TO TRAFALGAR ROAD
REGIONAL MUNICIPALITIES OF PEEL AND HALTON
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of or modification to the existing Winston Churchill Boulevard underpass. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the previous investigation undertaken in 1979 and 1980 (MTO GEOCRES No. 30M12-145). The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing Winston Churchill Boulevard structure consists of a two-span underpass, with the existing abutments and centre pier supported on vertical and battered H-piles. Based on the *General Arrangement* and *Footing and Pier Details* drawings from Contract No. 86-72, WP No. 167-77-01, dated April and October 1980 (which are provided in Appendix C for reference) the existing foundation details are summarized as follows:

Foundation Element	Pile Cap Width (m)	Pile Cap Thickness (m)	Pile Cap Founding Elevation (m)	Pile Orientation	Number of Piles	Pile Depth (m)
North	4.3	1.1	206.0	Battered Vertical	34 4	17.4 to 18.3 17.4
Centre pier	4.9	1.7	203.8	Battered Vertical	48 16	15.5 15.2
South abutment	4.9	1.2	204.8	Battered Vertical	34 6	16.1 to 17.1 16.1

Relative to the Highway 401 widening grade of approximately Elevation 206 m, the abutment pile caps are therefore located at or up to about 1.2 m below the highway grade. The existing abutment foreslopes are sloped at approximately 2 horizontal to 1 vertical (2H:1V).

Based on the planning and preliminary design study completed to date for the future widening of Highway 401 from Trafalgar Road to east of the Credit River, two lanes will be added in both the eastbound and westbound directions on Highway 401 in the vicinity of the Winston Churchill Boulevard underpass structure. It is understood that the following options are being considered to facilitate the proposed highway widening at this structure site:

- **Option 1:** Accommodate the Highway 401 widening by cutting into the existing abutment foreslopes and constructing permanent retaining structures. Section 6.2 presents an overview of various retaining system options for this purpose.

- **Option 2:** If the Highway 401 widening cannot be accommodated by cutting into the abutment foreslopes, full replacement of the existing underpass will be required. Section 6.3 presents an overview of various foundation options for this purpose.

6.2 Overview of Retaining System Options

As noted in Section 6.1, the north and south abutments are supported on piles, with the pile caps founded at Elevation 206.0 m and 204.8 m, respectively, relative to the existing/proposed widened grade for Highway 401 of approximately Elevation 206 m. The abutment pile caps are therefore located at or about 1.2 m below the proposed widened highway grade.

Based on the preliminary highway geometry and structural assessment to date, the proposed retaining structures could be located at horizontal distances of approximately 3.8 m and 2.4 m in front of the existing abutment walls at the north and south abutments, respectively, and the vertical wall height (above the Highway 401 grade) would be approximately 1.5 m to 2 m. Based on the existing abutment founding conditions and Golder's preliminary geotechnical assessment, construction of retaining structures founded at or below the Highway 401 grade, and therefore at an elevation similar to or below the existing abutment pile caps, would not impact the stability or geotechnical resistance of the existing pile-supported abutments. The existing abutment foreslopes could be temporarily cut to an orientation of 1 horizontal to 1 vertical (1H:1V) to permit construction of the retaining systems, or a temporary protection system could be used to support the abutment foreslope fill during the retaining system construction.

If modification of the existing abutment foreslopes is adopted for the highway widening, the selected retaining system type(s) must be constructable within the limited available headroom beneath the underside of the existing bridge deck. Based on the existing Highway 401 grade and the underside elevation of the bridge deck, it is estimated that there is a maximum of approximately 6 m of headroom in front of the north and south abutments, although this available height would be reduced further by the presence of the existing abutment foreslopes.

Depending on the type of wall constructed, the combination of the wall embedment below the Highway 401 grade, the base friction, and/or the wall's structural capacity and rigidity may be sufficient to resist the horizontal earth load (from the retained abutment foreslope) and any imposed loads. If this is not feasible, horizontal displacement of the wall may be restrained by including steel rods or wires (strands) drilled into and anchored in the ground behind the wall (tie-backs, ground anchors, or soil nails).

The following sections discuss the temporary and/or permanent retaining structures that may be feasible and practical for the Winston Churchill Boulevard underpass site, as follows:

- Conventional concrete retaining walls (either cast-in-place or pre-cast);
- Retained soil system (RSS) walls;
- Secant pile (caisson) walls;
- Soldier pile and lagging walls; and
- Soil nail walls.

The feasibility of each retaining system type at this site has been assessed on a conceptual/preliminary level based on compatibility with the existing pile-supported abutment configuration, including the potential excavation depth required for the retaining systems, the distance between the retaining system and the existing abutment walls, the requirements for temporary cuts or protection systems, and the potential deformation of soils during/after wall construction. This conceptual/preliminary assessment also takes account of workspace and headroom requirements, and other constructability aspects as applicable.

Based on these factors for the Winston Churchill Boulevard site, the preferred retaining system alternative from a geotechnical/foundations perspective would be concrete retaining walls or retained soil system walls.

6.2.1 Conventional Concrete Retaining Walls

Conventional cast-in-place concrete walls are well-established in their design and construction methods. A number of pre-cast versions of concrete retaining walls are also available. In general, pre-cast wall panels would be constructed at a concrete pre-casting plant, then attached on-site to a cast-in-place concrete footing with similar dimensions as for cast-in-place concrete walls. Cast-in-place walls offer the advantage of construction speed and potentially reduced costs as formwork is largely eliminated. However, their cost can exceed the cost of other wall options for similar project conditions, especially if the walls are relatively high and/or if a temporary protection system is required for construction of the permanent wall.

Conventional concrete wall systems could be constructed using a temporary 1H:1V cut into the existing abutment foreslopes if space permits, or using a temporary protection system. Based on the geometry of the existing pile-supported abutment and the likely excavation location for retaining systems, the temporary excavation to construct such retaining walls will not negatively impact the geotechnical resistance or stability of the existing abutments.

Concrete retaining walls can be of either a cantilever or gravity design, which support the retained earth through the following mechanisms:

- overturning moment is resisted by the counteracting direction of the soil weight on the footing;
- sliding of the wall is resisted by friction along the wall base and any soil in front of the wall; and
- the integrity of the wall is maintained by the structural capacity of the wall face and the footing connection.

Preliminary geotechnical resistances for the design of footings for concrete retaining walls are provided in Section 6.4. Select, free-draining granular fill meeting the specifications of OPSS 1010 Granular A or Granular B Type II (but with less than 5 percent passing the 200 sieve) should be used as backfill behind the concrete retaining wall.

6.2.2 Retained Soil System (RSS) Walls

In conventional retained soil system (RSS) wall construction, the retained soil mass is constructed using Granular A material that is placed and compacted in lifts with reinforcing strips placed at regular intervals within the soil mass. This type of wall can be constructed using conventional excavation equipment and conventional or hand-operated compaction equipment, and thus would fit within a low headroom environment at this site. The zone of reinforcing typically extends behind the wall a distance equivalent to approximately 0.7 to 0.8 times the height of the wall. Temporary excavation into the abutment foreslope at this site could be made using a 1H:1V slope, or potentially more of the foreslope fill could be removed without a negative impact on the pile-supported abutment.

A typical RSS wall has a front facing supported on a compacted granular pad or strip footing placed at shallow depth below the ground surface in front of the wall. This compacted pad or footing, and the reinforced soil mass, should be founded below any topsoil, loose fill or unsuitable native soils. For preliminary design at the Winston Churchill Boulevard site, the RSS facing footing and soil mass should be founded at or below approximately Elevation 205.5 m.

It has been assumed that a 1.5 m to 2 m high RSS wall at this site would act as a unit and use the full width of the reinforced soil mass, which has been taken to be 0.8 times the height of the wall. For this geometry, the preliminary design of RSS walls can be based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 200 kPa, and a geotechnical resistance at Serviceability Limit States (SLS) of 150 kPa.

6.2.3 Secant Pile (Caisson) Walls

A secant pile wall (or "caisson wall") is constructed by drilling holes between 0.9 m and 1.2 m in diameter to the full depth of the wall, inserting steel reinforcement in the form of steel beams or reinforcing bars, and filling the holes with concrete. The secant pile wall is formed by having each pile overlap the adjacent pile. A permanent secant pile wall often has a cast-in-place or precast concrete facing attached to the front surface to fill any gaps between piles and provide a smooth or architecturally appropriate surface finish. A secant wall can be designed as a cantilever wall (up to a site-specific limiting height), or with permanent tie-backs. Permanent walls must also include provisions for frost protection and control of any groundwater seepage.

The main advantages of a secant pile wall are increased wall stiffness compared to the more flexible sheet pile, soldier pile and lagging or soil nail wall systems, control of ground and groundwater by pile interlock, and the ability to be used in difficult ground containing cobbles or boulders. The main drawbacks are that vertical tolerances may be hard to achieve for deep piles (on the order of 30 m deep), they are relatively expensive to construct and waterproofing may be difficult to achieve at the joints. For the Winston Churchill Boulevard site, where the proposed walls would be approximately 1.5 m to 2 m in height, these considerations are not significant: the pile depths would be relatively shallow, groundwater seepage is not anticipated to be a significant issue, and greater wall stiffness (with lower deformations) are not an important consideration as the existing abutments are pile-supported.

Construction equipment for installing a secant pile wall can generally operate within a working area of about 7 m to 10 m width with the wall at nearly any position within that window. Equipment for constructing a secant pile wall generally consists of a mobile drill rig (which may be based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. In conventional construction, the height of caisson rigs can vary significantly; however, based on discussions with foundation contractors in the Toronto area, it is understood that smaller equipment is available that can work within heights of approximately 3.5 m to 5.5 m. To fit within the headroom available at the Winston Churchill Boulevard underpass structure, it may be necessary to excavate narrow slots (perpendicular to the abutment wall) within the abutment foreslopes to provide sufficient vertical clearance even for low headroom equipment. The use of smaller equipment may necessitate the construction of smaller diameter piles.

For the conceptual/preliminary assessment of retaining structure performance at the Winston Churchill Boulevard underpass site, it is anticipated that secant pile (caisson) walls would undergo a maximum horizontal and vertical displacement of about 0.1 per cent of the total excavation depth. For maximum wall heights of approximately 1.5 m to 2.0 m, this translates to less than about 5 mm of deformation at this site, provided that appropriate construction procedures and workmanship are adopted. For the Winston Churchill Boulevard site, where the existing abutments are pile-supported, this level of ground control is not considered necessary for the proposed retaining system.

6.2.4 Soldier Pile and Lagging Walls

Soldier pile and lagging systems are commonly used for temporary earth retention and can be constructed in a variety of ground conditions. Given the low headroom below the existing bridge deck, this type of wall could be installed by boring a series of 0.5 m to 1.0 m diameter holes, spaced 2 m to 3 m apart, into which H-piles (soldier piles) are installed; the annular space would then be filled with a relatively low strength sand-cement concrete mix. It may be necessary to splice sections of H-pile together given the low headroom.

After installation of the soldier piles the excavation would proceed with lagging (timber boards, concrete or steel sheeting) inserted behind the front flanges or placed against the piles and attached to the front flange using fasteners. Lagging is often installed in lifts of 1 m to 1.5 m; for the 1.5 m to 2 m high walls at this site, two lifts of excavation and lagging would likely be required. For permanent installations, pre-cast concrete lagging would normally be used but the alignment must be closely controlled during installation of the soldier piles to ensure a proper fit. Permanent soldier pile and lagging walls must include provisions for frost protection (as discussed further in Section 6.2.6) and control of any groundwater seepage (although seepage is not expected to be an issue at the Winston Churchill Boulevard site).

Depending on the wall height, to resist lateral forces and to control lateral wall movement, soldier pile and lagging walls typically require horizontal restraints such as tie-backs. However, at this site, given the proximity of the proposed retaining systems to the existing abutment walls, there may not be sufficient space within the abutment foreslopes to construct tie-backs, and it may be necessary for tie-backs to extend under the existing pile caps.

Construction equipment for installing a soldier-pile and lagging wall can generally operate within a horizontal working area of about 7 m to 10 m width with the wall at nearly any position within that area. Equipment for construction of soldier pile walls generally consists of mobile drill rigs (which may be based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. The height of conventional rigs can vary significantly; however, based on discussions with foundation contractors in the Toronto area, it is understood that smaller, "low headroom" equipment is available that can work within a maximum height of approximately 5.5 m. To fit within the headroom available at the Winston Churchill Boulevard underpass structure, it may be necessary to excavate narrow slots (perpendicular to the abutment wall) within the abutment foreslope to provide sufficient vertical clearance even for low headroom equipment.

For the conceptual/preliminary assessment of retaining structure performance at the Winston Churchill Boulevard underpass site, it is anticipated that soldier pile and lagging walls would undergo a maximum horizontal and vertical displacement of about 0.2 per cent of the total excavation depth. For maximum wall heights of approximately 1.5 m to 2 m, this translates to less than about 5 mm of deformation at this site. This estimated displacement assumes good quality workmanship; however, it is noted that ground loss can be more common with this type of retaining system, especially during lift excavation and installation of lagging elements, and there is some risk of potential for higher displacements in the event of ground loss or otherwise poor workmanship. For the Winston Churchill Boulevard site, where the existing abutments are pile-supported, this level of ground control is more than adequate for a proposed retaining system.

6.2.5 Soil Nail Walls

Temporary and permanent retaining walls can be constructed using the soil nailing technique whereby the ground is supported by inserting and grouting reinforcing steel rods (nails) into the ground on a regularly spaced grid and then covering the excavation face with steel mesh and shotcrete structurally connected to the nails. The permanent facing can then be constructed of successive layers of shotcrete, precast panels, or a cast-in-place concrete face. The solid steel reinforcing bars are the main component of the system and tensile stress is applied passively to the nails in response to the deformation of the retained materials during subsequent excavation activities. Where more stringent deformation control is required, post-tensioning of the soil nails can be undertaken, although this measure would increase the project cost.

Soil nail walls are constructed from within the area to be excavated and require between about 6 m and 10 m of working space in front of the wall for equipment. The working space required will depend on the equipment chosen, staging, and routes required for earth moving equipment. In general, the equipment is portable and can operate in limited headroom environments as the nail installation is horizontal to sub-horizontal. It requires relatively little space and generates less noise and requires less manpower than other methods.

The length of the soil nails is usually 0.6 to 1 times the height of the wall and less than what is used in conventional tie-back or soil anchor construction. Soil nailing is most economical in ground that can stand unsupported for at least one day on a vertical or steep slope cut 1 m to 1.8 m high and in which drill holes can remain open for at least several hours. This method of construction is best suited for use in deposits of dense granular and stiff low plasticity clayey soils. While the length of soil nails for proposed retaining walls at this site should fit within the available space behind the retaining system and in front of the abutment wall, the composition and consistency of the abutment foreslope soils is not known.

During construction and after its completion, a soil nail wall and the soil behind it tend to deform outwards. Maximum horizontal displacements occur at the top of the wall and decrease progressively toward the toe of the wall. Vertical displacements of the wall at the facing are generally small, and are on the same order of magnitude as the horizontal movements at the top of the wall. Empirical data show that for soil nail walls with a typical nail length to wall height ratio between 0.7 and 1.0, negligible surcharge loading, and a typical global factor of safety value of 1.5, the maximum long-term horizontal and vertical wall displacements at the top of the wall, δ_h and , δ_v can be estimated as follows:

$$\delta_h = (\delta_h/H)_i \times H$$

where $(\delta_h/H)_i$ = a ratio dependant on the soil conditions (assume 0.002 for Granular A material)

H = wall height

For the conceptual/preliminary assessment of the performance of retaining structures up to approximately 1.5 m to 2 m in height at this site, it is anticipated that soil nail walls will undergo maximum displacements of up to about 5 mm (similar to soldier pile and lagging walls). In addition, post-construction monitoring of soil nail wall displacements indicates that movements tend to continue after wall construction, sometimes for periods of up to six months, depending on the ground type. Typically, the post-construction deformation is about 15 per cent of the deformations observed soon after construction. Again, for the Winston Churchill Boulevard site, where the existing abutments are pile-supported, this level of ground control would be more than adequate for a proposed retaining system.

6.2.6 Frost Protection and Drainage Requirements for In Situ Walls

In situ walls will be subjected to freezing ambient temperatures at the wall face during winter. The walls will also be in direct contact with the ground behind the wall. It is anticipated that the wall materials will serve as a thermal conductor and unless insulation is provided at the wall face, the freezing temperatures may cause ice lenses and frost pressures behind the wall. The design and construction of such walls will require that consideration be given to providing the face of the wall with insulation and a protective wall facing. This is consistent with other projects in Ontario where permanent in situ walls have been fitted with an insulation layer to prevent such pressures.

For in situ wall systems, drainage can be achieved using prefabricated drainage elements placed behind the permanent facing. Where precast concrete panels are used, the space between the temporary wall face and the permanent facing may be backfilled with a drainage element consisting of an approved granular drainage layer or pre-fabricated drainage material. Water intercepted by the drainage layer will flow downward to the base of the wall where it can be removed by a subdrain arrangement, or conveyed through the permanent facing via a weep hole.

6.3 Overview of Foundation Options for Structure Replacement

If modifications to the abutment foreslopes do not result in sufficient space for the proposed Highway 401 widening, then full replacement of the existing underpass structure will be required. Based on preliminary drawings, it is understood that the preferred replacement alternative for the existing underpass is a two-span structure, roughly on the same alignment as the existing. The Highway 401 pavement grade is proposed to be maintained at approximately Elevation 205.5 m to 206 m at the structure site, and the finished grade for the Winston Churchill Boulevard embankments will be approximately Elevation 214 m at the north and south abutments, such that the approach embankments remain at a height of approximately 5 m to 6 m relative to the natural ground surface.

Based on the proposed underpass geometry and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments and centre pier for replacement of the Winston Churchill Boulevard underpass, or for support of concrete retaining walls constructed in front of the existing abutments. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded on the very stiff clayey silt till:** Strip or spread footings are feasible for support of the abutments and centre pier for a replacement structure; footings are also feasible for support of concrete retaining walls in front of the existing abutments at this site. It is expected that shallow excavations (relative to the Highway 401 pavement grade) would be required in order to extend into the very stiff clayey silt till for new abutments or for retaining systems in front of the abutments. Deeper excavations (about 3 m below highway grade to Elevation 203 m) would be required for the centre pier to extend below the stiff till into the very stiff portion of the till. It is expected that temporary protection systems would be required along the Winston Churchill Boulevard embankment and in the Highway 401 median to facilitate excavation while maintaining traffic.
- **Footings "perched" on a compacted granular pad in the approach embankment fill:** Although this option would be advantageous in minimizing the depth of excavation, "perched" footings are not recommended for support of the replacement structure due to the potential settlement associated with the firm to stiff clayey silt fill under a potentially widened embankment loading; however this could be mitigated by subexcavating and reconstructing the approach embankments under the abutment footing areas.
- **Driven steel H-piles:** Driven steel H-piles are suitable and feasible for support of the abutments and centre pier (and would permit integral abutment design), as well as for retaining structures in front of the existing abutments. This option is advantageous over shallow foundations for the abutments and pier for a replacement structure, as the abutment pile cap could be perched within the approach embankment fill, minimizing excavation through the existing Winston Churchill Boulevard embankment, and the centre pier pile cap could be maintained higher than for a spread footing foundation, minimizing the depth of excavation in the Highway 401 median area. The use of driving shoes is recommended due to the hard nature of the soils and the potential presence of cobbles and boulders in the glacially derived soils.
- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments and piers, as well as well as for retaining structures in front of the existing abutments. However, pipe piles are considered to have a slightly higher risk than H-piles for "hanging up" or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the glacially-derived soils at this site.
- **Caissons:** Caissons are feasible for this site but would require the use of temporary or permanent liners given the potential risk for "running soil" associated with perched water at the base of the fill and water-bearing interlayers or zones within the till through which the caissons would be constructed. As such, driven piles would be preferred over caissons as a deep foundation option.

The following sections provide recommendations for both shallow and deep foundations to support the proposed works. From a foundations perspective, based on the above considerations and the soil conditions at the site, the preferred option for a replacement structure from a geotechnical/foundations perspective is to support the abutments and centre pier on driven pile foundations to minimize the depth of excavation through the existing Winston Churchill embankment, and to minimize total and differential settlement associated with the stiff to very stiff portions of the till deposit. However, spread footings are also considered to be a feasible alternative for support of the new centre pier, although some differential settlement would occur relative to pile-supported abutments.

6.4 Shallow Foundations

6.4.1 Founding Elevations

For support of the new abutments and centre pier, or for support of a concrete retaining wall in front of the existing abutments, strip or spread footings should be founded below the fill and any firm to stiff soils, on the generally very stiff clayey silt till. The founding elevation for the new footings should be a minimum of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration. The following maximum (highest) founding elevations are recommended for preliminary design of the footings for the replacement structure:

Foundation Element	Borehole Number	Founding Stratum	Strip/Spread Footing Founding Elevation (m)
North abutment/ retaining system	3, 7	Very stiff to hard clayey silt till	204.3*
Centre pier	1, 2, 6	Stiff to very stiff clayey silt till	203.0
South abutment/ retaining system	4, 5	Stiff to hard clayey silt till	204.3*

* Based on a finished pavement grade of approximately Elevation 205.5 m and a minimum depth of 1.2 m.

If a replacement is adopted with shallow foundations at or near the existing pier location, it would be necessary to remove the existing pile cap and to cut the existing piles off below the new founding level. Depending on the configuration of the replacement structure, consideration could be given to re-using the existing pile-supported centre pier; however, in this case, shallow foundations are not recommended for any widening of the existing centre pier due to the potential for differential settlement on the stiff to very stiff till.

In regard to the subexcavation for the centre pier, the excavation can be carried out to the elevation identified above, then backfilled with compacted Ontario Provincial Standard Specification (OPSS) 1010 Granular A or Granular B Type II fill prior to construction of the footings at a higher elevation. In this case, the founding elevation for the new footing should be a minimum of 1.2 m below the lowest surrounding grade (i.e., footing elevation of approximately 204.3 m) to provide adequate protection against frost penetration, as per Ontario Provincial Standards. The compacted granular pad should extend at least 1 m beyond the front and back edge of the new pier footing, then outward and downward at 1H:1V.

The footing subgrade should be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with Provincial Standards to check that all existing fill, firm to stiff soils, or other unsuitable material have been removed. The founding soils will be susceptible to disturbance and should be protected with a concrete working slab (as discussed further in Section 6.6.3) if the concrete for the footing is not placed within four hours of the inspection and approval of the subgrade.

6.4.2 Geotechnical Resistance/Reaction

Strip or spread footings placed on the properly prepared, very stiff to hard clayey silt till (or on compacted granular fill following subexcavation), at or below the design elevations given in the preceding section, should be designed based on the factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) given below. These recommendations are provided for footing widths of approximately 3 m to 4 m.

	Founded On Very Stiff to Hard Clayey Silt Till		Founded on Compacted Granular Fill at 1.2 m Depth	
Foundation Element	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS***	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS***
North abutment/ retaining system	500 kPa	350 kPa	N/A	N/A
Centre pier	450 kPa	300 kPa	500 kPa	350 kPa
South abutment/ retaining system	450 kPa	300 kPa	N/A	N/A

* For 25 mm of settlement

The geotechnical resistances/reactions should be reviewed if the selected footing width or founding elevation differs from those given above. In addition, these preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigations at the proposed abutment and centre pier locations for the replacement structure.

6.5 Driven Steel H-Pile or Steel Pipe Pile Foundations

6.5.1 Founding Elevations

The abutments and centre pier for a replacement structure, or concrete retaining walls in front of the existing abutments, may be supported on steel H-piles or steel pipe piles driven to found within the "100-blow" clayey silt till encountered in the deeper boreholes (Borehole Nos. 5, 6 and 7). The following pile tip elevations may be used for preliminary design purposes, assuming about 2 m of penetration into the "100-blow" till.

Foundation Element	Borehole Nos.	Estimated Design Pile Tip Elevation
North abutment/ retaining system	3, 7	187 m
Centre pier	1, 2, 6	184 m
South abutment/ retaining system	4, 5	187 m

The pile caps should be constructed at a minimum depth of 1.2 m for frost protection purposes. If a structure replacement is adopted with a new pile-supported centre pier at or near the existing pier location, it would be necessary to remove the existing pile cap and to consider the pile layout and spacing in the new design, with piles either re-used in the new pier configuration or cut off below the new underside of pile cap. Depending on the configuration of the replacement structure, consideration could also be given to re-using the existing pile-supported centre pier, with any necessary widening supported on pile foundations to minimize differential settlement.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and/or boulders within the soil deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of "hanging up" or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 *(Deep Foundations)*. In the hard soils which will be encountered at this site, driving shoes (such as Titus Standard "H" Bearing Pile Points) are preferred over flange plates.

6.5.2 Axial Geotechnical Resistance

For HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial resistance at ULS and the axial geotechnical reaction at SLS (for 25 mm of settlement) may be taken as follows for preliminary design:

Foundation Element	Borehole Nos.	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
North abutment/ retaining system	3, 7	1,400 kN	1,200 kN
Centre pier	1, 2, 6	1,400 kN	1,200 kN
South abutment/ retaining system	4, 5	1,400 kN	1,200 kN

Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.).

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation that will be carried out at the widened foundation elements.

6.6 Approach Embankments

The preliminary recommendations presented in this section assume that the replacement of the existing Winston Churchill underpass will involve minor widening of the existing approach embankments (less than approximately 3 m to 4 m) to the west and/or east.

6.6.1 Subgrade Preparation

To improve the performance of the widened portion relative to the existing embankments, it is recommended that all topsoil/organic material or existing surficial fill materials be stripped from the footprint of the proposed approach embankment widening areas. Consideration may also be given to stripping firm portions of the clayey silt till deposit, where these are present at the subgrade level. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available for the approach embankment areas.

6.6.2 Approach Embankment Stability

Slope stability analyses have been performed for a proposed embankment widening using the commercially available program SLIDE, produced by Rocscience Inc., to check that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for approach embankments on this project, considering the design requirements and the available field and laboratory testing data.

The stability analyses were completed for an approximately 5 m to 6 m high approach embankment, based on the subsurface conditions as encountered in the boreholes across the site. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations:

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill, assuming the use of clean earth fill for the widening	21	32°	-
Stiff to hard clayey silt till	21	32° to 35°	-

The results of the stability analyses are shown on Figure 1; the results indicate that a Factor of Safety greater than 1.3 is achieved for the approximately 5 m to 6 m high widened approach embankments with side slopes oriented no steeper than 2H:1V, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment widening footprint during detail design.

6.6.3 Approach Embankment Settlement

Settlement analyses for the anticipated soil conditions below the approach embankments were carried out using the commercially available computer program Settle-3D from Rocscience, using estimated elastic deformation moduli as given in the table below, based on correlations with the SPT "N" values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974). The analysis assumes that the existing firm to stiff embankment fill remains in place (either below the pile cap level or behind the backfill zone for new abutments) below the widened approach embankment side slopes.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
Firm to stiff clayey silt fill	20	12
Stiff to very stiff clayey silt till	21	25
Very stiff to hard clayey silt till	21	45

At this stage, it has been assumed that the existing Winston Churchill Boulevard approach embankments could be widened by a maximum of approximately 5 m. Based on this preliminary assessment, the settlement of the embankment soils under the widened 5 m to 6 m high approach embankments is estimated to be a maximum of approximately 40 mm to 50 mm, decreasing to less than 15 mm under the new widened embankment toe; the majority of this settlement will occur in the existing embankment fill (assuming it remains in place under the widening) and the stiff portions of the till deposit. This settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments based on the nature of the soils at the site. This estimated magnitude of settlement should be reassessed based on the soil and groundwater conditions under the widened approach embankments as determined during the detail design, with particular emphasis on the thickness and properties of the existing embankment fill and the upper portion of the till deposit within the embankment widening footprint.

The above preliminary estimates do not include compression of the new fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

6.7 Construction Considerations

The following subsections identify future construction issues that should be considered at this stage as they may impact the planning and preliminary design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during detail design for incorporation in the Contract Documents.

6.7.1 Excavation and Temporary Protection Systems

If retaining structures are constructed in front of the existing abutments, excavation will extend through the existing abutment foreslope material and potentially to a depth of about 1.2 m below the Highway 401 grade in front of the wall. If a replacement structure is adopted, the foundation excavations for spread footings are expected to extend to a depth of about 1.2 m to 3 m below the Highway 401 grade, through the existing fill and firm to stiff clayey silt till, and into the very stiff to hard clayey silt till. The excavations for pile caps for the abutments or the central pier could be maintained higher within the native soils (or may even be perched within the embankment fill), to satisfy the minimum founding depth of 1.2 m for frost protection purposes.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and the firm to stiff portions of the native soils would be classified as Type 3 soil; the very stiff clayey silt till is classified as Type 2 soil, and the hard clayey silt till is classified as Type 1 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

In order to facilitate the removal of the existing wingwalls/retaining walls adjacent to the existing abutments, and to excavate to the foundation level for the new abutments and associated wing walls or retaining walls while staging traffic on the local road throughout construction, it is anticipated that temporary roadway protection would likely be required along Winston Churchill Boulevard; temporary protection may also be required in the Highway 401 median to facilitate excavation for removal of the existing pile cap and/or construction of the new/widened pile cap. The temporary excavation support system should be designed and constructed in accordance with OPSS 539 *(Construction Specification for Temporary Protection Systems)*. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539.

It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support for a replacement structure, based on the subsurface soil and groundwater conditions.

6.7.2 Groundwater Control

During the drilling operations, groundwater was encountered at a depth of approximately 10.5 m in Borehole No. 3 (Elevation 198 m); however, the stabilized groundwater level is expected to be higher, at approximately Elevation 204 m (about 1 m to 2 m below the Highway 401 cut grade). Groundwater seepage is anticipated from "perched" water at the base of granular fills, as well as from cohesionless lenses or interlayers within the clayey silt till deposit, such as was encountered in Borehole No. 1 at a depth of 1.5 m (Elevation 204.4 m). For the potential depth of excavation associated with spread footings or pile caps, the seepage volume is expected to be relatively small, such that the water inflow can be handled by pumping from filtered sumps placed at the base of the excavations. Based on these small seepage volumes, it is expected that a Permit to Take Water (PTTW) would not be required for the groundwater control system at this site. Additional groundwater monitoring is recommended during the detail design stage.

6.7.3 Subgrade Protection

The clayey silt till soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP, which can be developed during the detail design stage.

6.7.4 Obstructions

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. The frequency of the occurrence of cobbles and boulders should be identified during future investigations as part of the detail design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.

6.7.5 Existing Structure Foundations

If a full structure replacement is adopted, consideration will need to be given to potential conflicts between the existing and new foundation elements, particularly at the existing pile-supported centre pier. Temporary protection systems would be required for removal of the existing pile cap and to facilitate cutting off the existing piles (where necessary) below the founding level for the new centre pier element.

6.7.6 Vibration Monitoring During Pile Installation

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities (such as pile driving) will reach this threshold level and, therefore, vibration monitoring for the existing structure is not expected to be required during construction at this site. However, there are several commercial buildings south of the site, and the requirements for monitoring of vibrations during construction should be evaluated during the detail design stage. If warranted, an NSSP should be included in the Contract Documents at the detail design stage to develop a vibration monitoring plan that would include appropriate review and alert levels for vibrations for the existing buildings and the rail tracks.

6.8 Recommendations for Further Work in Detail Design

Additional boreholes are recommended within each of the foundation areas (abutments and centre pier), any new retaining wall footprints, and the approach embankment areas during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report, as follows:

- Retaining system option:
  - Investigation of the subsurface conditions in front of the existing abutment walls to confirm the bearing resistances and provide information to allow the contractor to assess the potential behaviour of soils during construction.
- Underpass replacement option:
  - Assessment of the thickness and properties of any surficial deposits and the upper portion of the clayey silt till to confirm the founding elevation for spread footings within each foundation element, and to assess groundwater control requirements associated with seepage from lenses/interlayers in the till, if present.
  - Observation of the presence of cobbles and/or boulders within the soil deposits, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of driven steel H-pile foundations or temporary protection systems.
  - Assessment of vibration thresholds for the nearby commercial buildings, and if warranted development of an NSSP for a vibration monitoring plan.
- Approach embankments and adjacent high fill embankments:
  - Assessment of the depth and extent of stripping of topsoil/organics and weaker surficial soils within the footprint of the proposed widened approach embankments.
  - Further refinement of the embankment settlement analysis, in particular if variable surficial soils are encountered within the proposed approach embankment location.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Nick La Posta, P.Eng. and Mr. Mehdi Mostakhdemi, M.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Ty Garde, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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Ontario Provincial Standard Specifications (OPSS)

OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.101	Foundation Frost Depths for Southern Ontario

TABLE 1 – COMPARISON OF RETAINING SYSTEM ALTERNATIVES

	Conventional Concrete Retaining Wall	Retained Soil System (RSS) Walls	Secant Pile (Caisson) Wall	Soldier Pile and Lagging Wall	Soil Nail Wall
Advantages	• Relatively easy to construct and does not require specialized construction skills. Construction not impacted by lower headroom under existing bridge deck. • Excellent demonstrable performance behaviour. • Panels can be pre-cast which reduces the level of construction effort, increases the speed of construction and reduces formwork costs.	• Can be constructed using conventional excavating and compacting equipment in limited headroom • Flexible structure type with good performance • Lowest cost alternative	• Uses top-down construction • Can withstand relatively large horizontal wall loads compared to soldier pile and lagging walls, with less deformation behind the wall due to the greater stiffness of the retaining system (estimated at approximately 0.1% of wall height, or less than about 5 mm for this site); however, this advantage is not critical for the existing pile-supported abutment geometry at this site • Can be installed relatively close to the existing abutment pile cap. • Can serve for both temporary and permanent retaining structure solution.	• Uses top-down construction • Can withstand horizontal wall pressures without requiring a significant increase in wall section; however, this advantage is not critical for the existing pile-supported abutment geometry at this site. • Construction procedures are well understood by local contractors. • Proven reliable performance.	• Uses top-down construction, and structural elements don't need to extend below excavation base. • Installation is relatively rapid and uses less construction materials than soldier pile and lagging walls. • Easy adjustment of nail inclination and location can be made when obstructions are encountered. • Overhead construction requirements are reduced compared to some wall types. • Shotcrete facing is typically less costly than the structural facing required for other wall systems.
Disadvantages	• Cost increases significantly as wall height increases.		• Requires specialized equipment to work in the limited headroom. • Relatively expensive to construct, especially given the low headroom at this site.	• Requires specialized equipment to work in limited headroom. • More "flexible" system; not appropriate where very strict deformation control is required for structures located within the zone of influence behind the wall (although this is not a factor for the Winston Churchill Boulevard site). • Tie-backs, if required, may not fit within horizontal distance between retaining wall and abutment wall at this site. • Performance can be very dependent on construction technique; requires careful fitting of lagging elements. • Generally suitable as a temporary system to facilitate construction of permanent retaining structure, although it is also possible to adopt this as a permanent structure (with concrete panels instead of timber lagging).	• Not appropriate for applications where very strict deformation control is required for structures located within the zone of influence behind the wall (although this is not the case for the Winston Churchill Boulevard site). • Horizontal position of ground anchors is more difficult to modify making field adjustments costly. • Construction of the wall requires specialized and experienced contractors; there is not significant experience and information on soil nail wall performance locally.

	Conventional Concrete Retaining Wall	Retained Soil System (RSS) Walls	Secant Pile (Caisson) Wall	Soldier Pile and Lagging Wall	Soil Nail Wall
Risks/Consequences	• Very low risk of failure. • Recommended for use along a wall alignment where wall deformations of approximately 0.2% (or greater) of the wall height/excavation depth can be tolerated, to account for use of temporary protection system if needed. This is more than adequate at this site where the existing abutments are pile-supported.	• Very low risk of failure or settlement. • Recommended for use along a wall alignment where deformations of approximately 0.2% (or greater) of the wall height/excavation depth can be tolerated. This is more than adequate at this site where the existing abutments are pile-supported.	• A temporary liner is recommended to minimize the risk of loss of ground. However, it may be difficult to use a temporary liner due to the lower torque associated with low headroom equipment. If a temporary liner can be employed in a low headroom environment, this option is recommended for use along a wall alignment located where the deformations must be restricted to less than 0.1% of the wall height/excavation depth (however, this degree of ground control is not necessary in front of the existing pile-supported abutments).	• Risk of higher wall deformation with associated ground settlement behind the wall (although this is not a significant concern for the pile-supported abutments at this site). • As for a secant pile wall, a temporary liner is recommended to minimize the risk of loss of ground in the clayey silt fill. However, it may be difficult to use a temporary liner to advance the caisson holes due to the lower torque associated with low headroom equipment. • If a temporary liner can be employed in a low headroom environment for temporary support of the soldier pile holes, this option is recommended for use along a wall alignment located where wall deformations of approximately 0.2% of the wall height/ excavation depth can be tolerated (however, this degree of ground control is not necessary in front of the existing pile-supported abutments).	• Risk of unacceptable wall deformation and consequent abutment footing settlement if the wall is constructed relatively close to the abutment footing. • Recommended for use along a wall alignment located where wall deformations of approximately 0.2% (or greater) of the wall height/excavation depth can be tolerated (however, this degree of ground control is not necessary in front of the existing pile-supported abutments).
Approximate Cost	Approximately $700/m^2	Approximately $650/m^2	Approximately $1,800/m^2	Approximately $1,500/m^2	Approximately $800/m^2

NOTES: Costs are preliminary estimates and are intended to provide a comparison between alternatives. Actual construction costs may vary.

TABLE 2 – COMPARISON OF FOUNDATION ALTERNATIVES FOR STRUCTURE REPLACEMENT

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Strip or spread footings founded in the very stiff to hard clayey silt till	• Feasible for support of the abutments, associated wing walls/retaining walls, and for the centre pier	• Relative ease of construction • Relatively minor groundwater seepage anticipated • Minor post-construction settlement • Lower vibration impacts on existing structures than for driven steel H-pile installation • Allows for semi-integral abutments	• Excavations for the centre pier would extend to a depth of up to 2.5 to 3 m below ground surface to extend below existing fill and the stiff clayey silt till; would require temporary protection systems • Precludes use of integral abutments; potentially greater maintenance required at abutments • Lower geotechnical resistances as compared with deep foundations • Potential conflicts with existing centre pier	• Conventional excavation and construction techniques	• Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration • Estimated cost is about $600/m^3 for a concrete unit for construction of shallow foundations, excluding deeper excavation and temporary protection system
Spread/strip footings perched on compacted granular pad in approach embankment fill	• Not recommended for support of the new abutments	• Abutment footings could be maintained higher than footings founded on till, reducing temporary protection system requirements	• Potential unacceptable settlement due to firm to stiff cohesive embankment fill and stiff clayey silt till underlying perched abutment footings; settlement would be differential with respect to centre pier • Precludes use of integral abutments; potentially greater maintenance required at abutments	• Conventional excavation and construction techniques	• Not assessed as this option not recommended at this site

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to the "100-blow" clayey silt till	• Feasible for support of new abutments, associated wing walls/retaining walls, and centre pier	• Abutment pile caps could be maintained higher than footings founded on very stiff to hard till, reducing depth of excavation and temporary protection system requirements adjacent to Winston Churchill Boulevard and in the Highway 401 median • Limited groundwater control required • Allows for integral abutment construction • Would minimize differential settlement between foundation elements	• Potential for encountering obstructions (cobbles and/or boulders) during pile driving; this could result in piles "hanging up" and lower geotechnical resistances • Potential for noise and/or vibration impacts on nearby buildings • Potential conflicts with existing centre pier foundations	• Conventional construction methods for H-pile foundations	• Lower relative cost compared with caisson option • Estimated unit cost is approximately \$250/linear metre for pile installation and \$600/m^3 for pile cap construction
Steel pipe (tube) piles, driven to the "100-blow" clayey silt till	• Feasible for support of new abutments, wing walls/retaining walls, and centre pier	• Abutment pile caps could be maintained higher than footings founded on very stiff to hard till, reducing depth of excavation and temporary protection system requirements adjacent to Winston Churchill Boulevard and in the Highway 401 median • Limited groundwater control required • Allows for semi-integral abutment construction • Would minimize differential settlement between foundation elements	• Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in piles "hanging up" and lower geotechnical resistances • Potential for noise and/or vibration impacts on nearby buildings • Potential conflicts with existing centre pier foundations	• Conventional construction methods	• Costs for steel pipe (tube) piles slightly higher than for steel H-piles

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Caissons founded in the hard (100-blow) clayey silt till	• Feasible but not recommended for support of abutments, wing walls/retaining walls and centre pier	• Abutment pile caps could be maintained higher than footings founded on very stiff to hard clayey silt till, reducing depth of excavation and temporary protection system requirements adjacent to Winston Churchill Boulevard and in the Highway 401 median • Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles	• Potential for loss of ground in water-bearing cohesionless lenses which are anticipated to be encountered in the clayey silt till unit • Temporary or permanent liners would be required; likely not possible to inspect caisson base • Precludes use of integral abutments • Potential conflicts with existing centre pier locations	• Conventional construction methods	• Higher cost compared with shallow foundations or steel H-piles

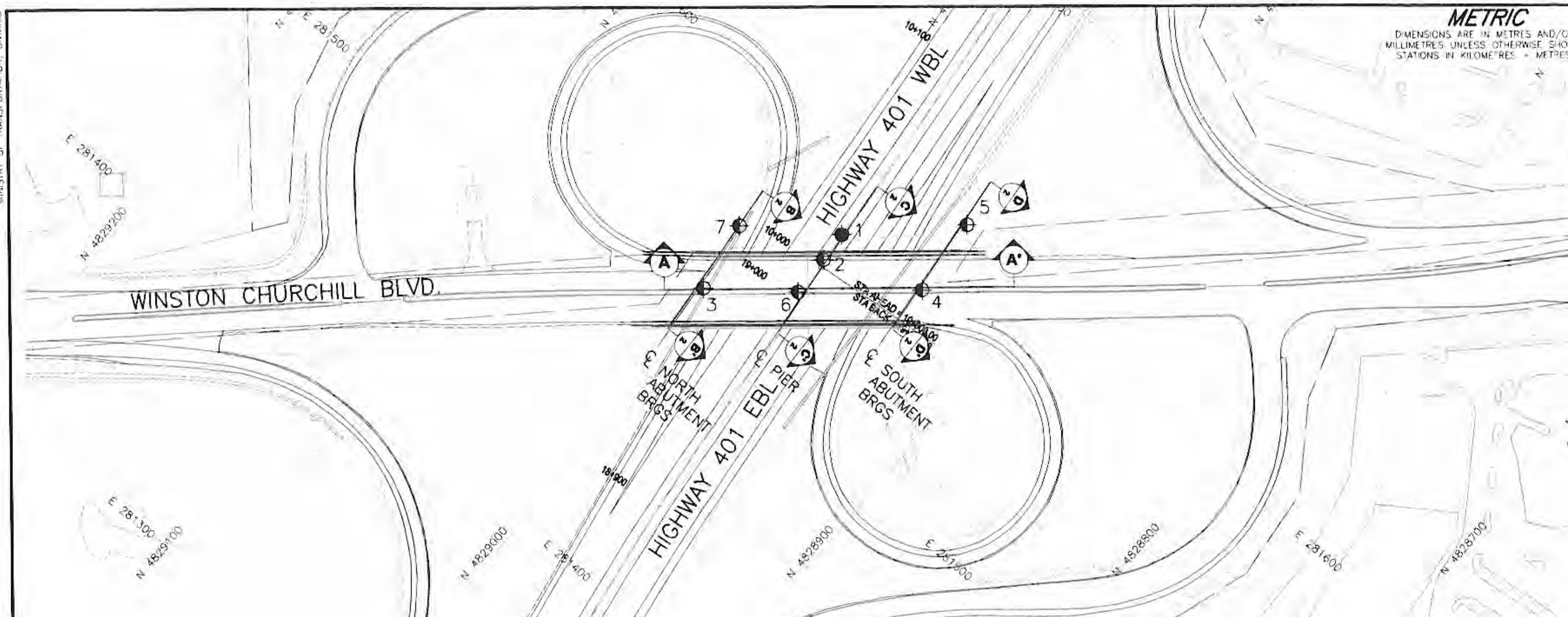
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 07-20021

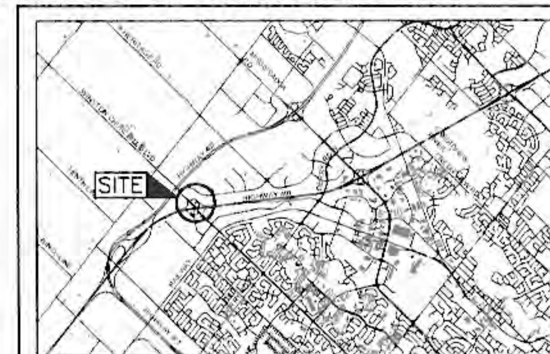
WINSTON CHURCHILL BLVD. UNDERPASS
HIGHWAY 401 WIDENING
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

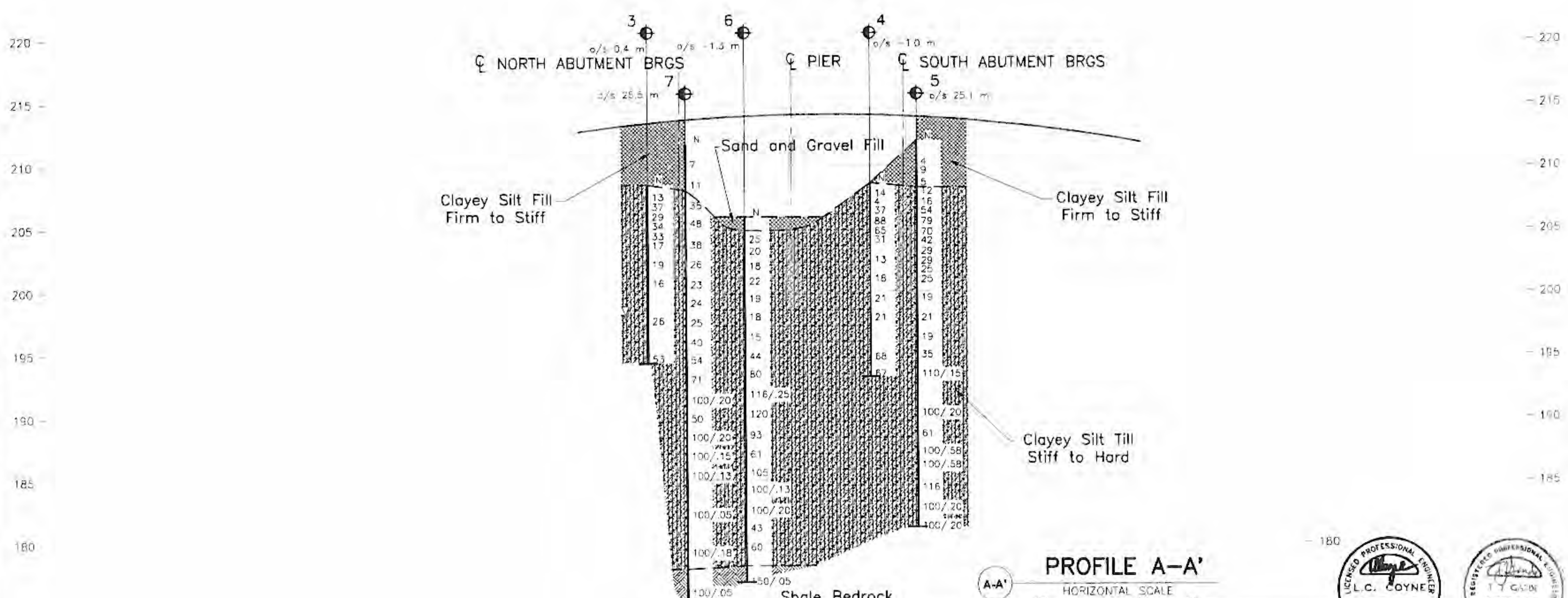
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
20 0 20 40 m



KEY PLAN
SCALE
1.5 0 1.5 3 km



PROFILE A-A'
HORIZONTAL SCALE 20 0 20 40 m
VERTICAL SCALE 4 0 4 8 m

LEGEND

- Borehole – Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 J/blow)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No	ELEVATION	NORTHING	EASTING
1	205.9	4828976.1	281569.3
2	205.9	4828975.2	281556.9
3	208.5	4829004.7	281516.7
4	208.6	4828937.2	281573.2
5	212.0	4828940.2	281605.0
6	205.9	4828974.7	281540.3
7	211.7	4829009.9	281545.3

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 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.

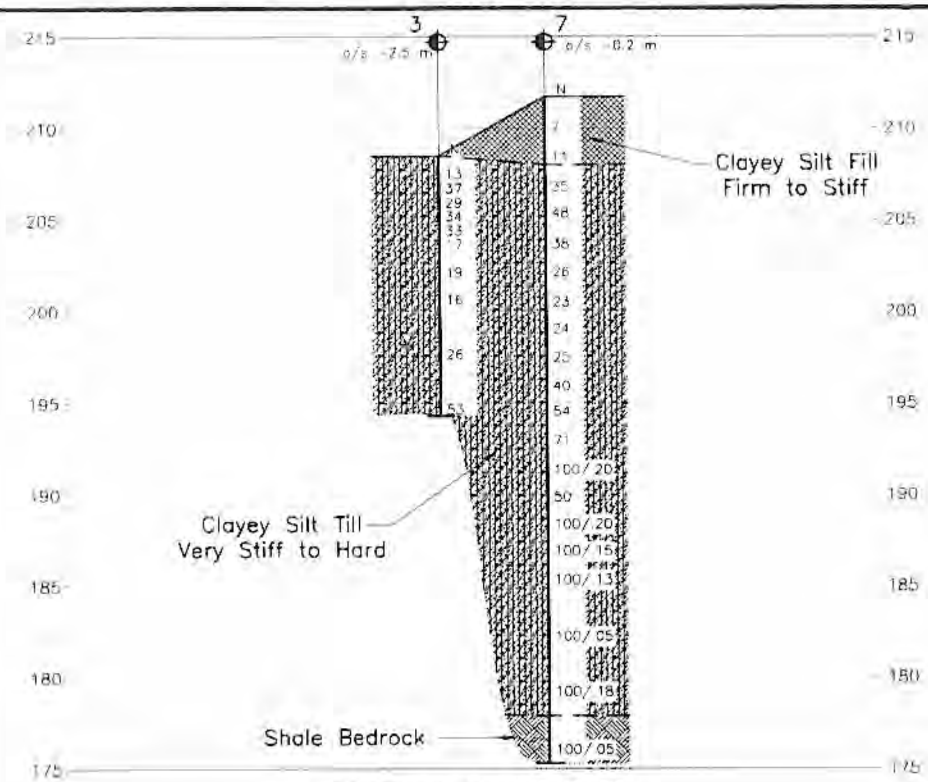
REFERENCE

Base plans provided in digital format by URS, drawing file has ACAD–X–base1_to_Trafalgar.dwg and ACAD–Aerials_MTO ROW_ Property Boundaries.dwg, received August 17, 2011 and August 29, 2011

NO.	DATE	BY	REVISION

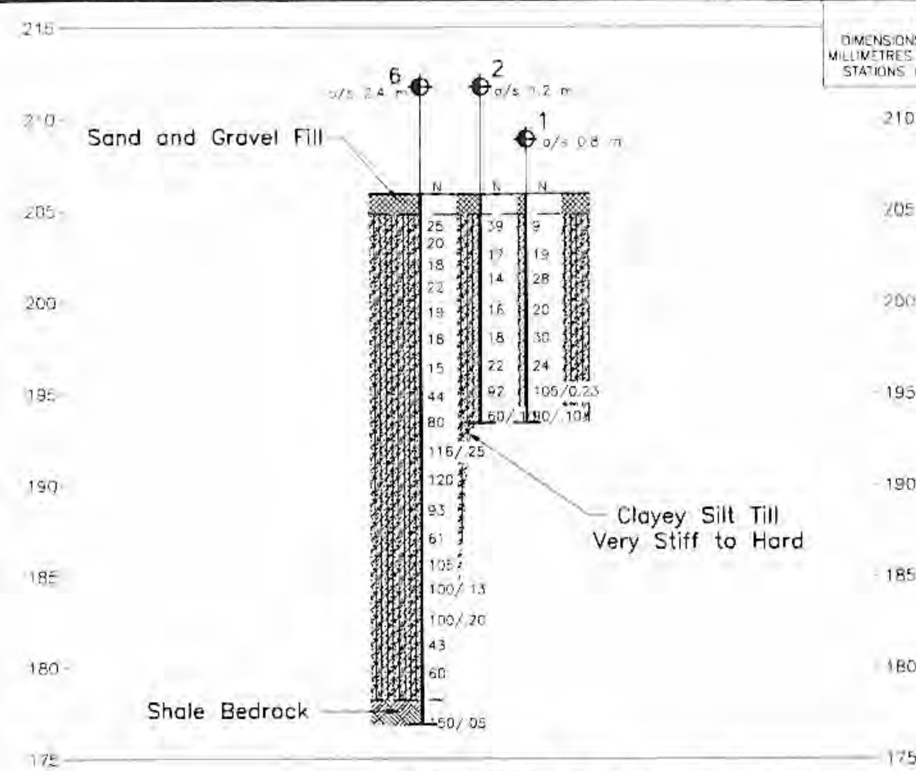
Geocres No. 30M12-358			
HWY 401		PROJECT NO. 10-1111-0040	DIST
SUBM'D NL	CHKD LCC	DATE 12/19/2012	SITE
DRAWN JFC	CHKD NL	APPD LCC	DWG 1

MINISTRY OF TRANSPORTATION, ONTARIO



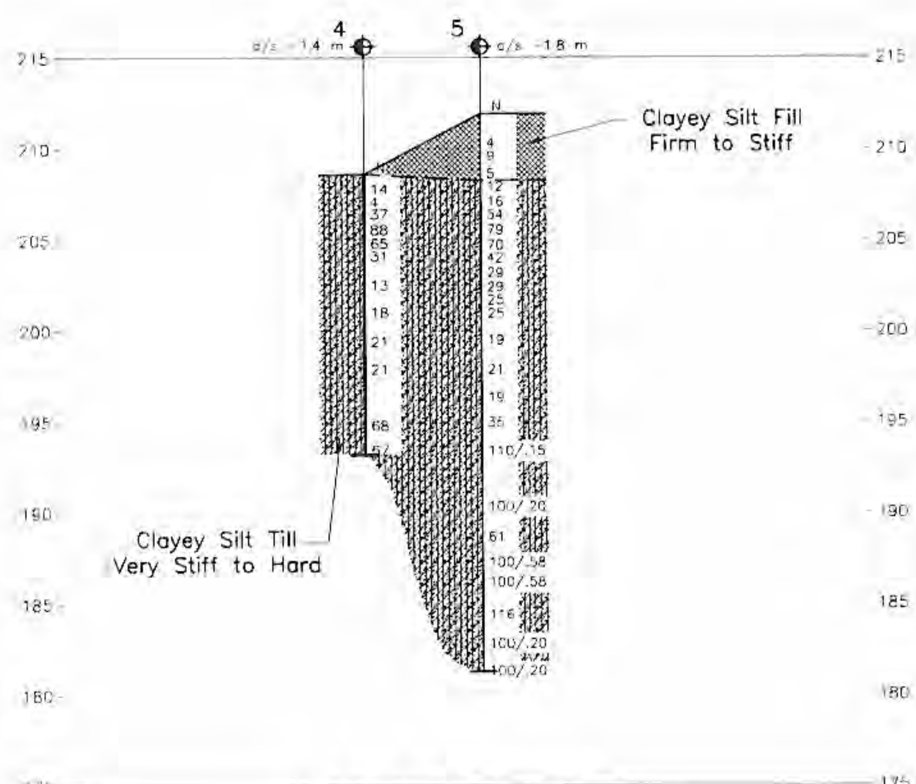
(B-B') NORTH ABUTMENT
SECTION B–B'

HORIZONTAL SCALE 20 0 20 40 m
4 0 4 8 m VERITCAL SCALE



(C-C') CENTRE PIER
SECTION C–C'

HORIZONTAL SCALE 20 0 20 40 m
4 0 4 8 m VERITCAL SCALE



(D-D') SOUTH ABUTMENT
SECTION D–D'

HORIZONTAL SCALE 20 0 20 40 m
4 0 4 8 m VERITCAL SCALE

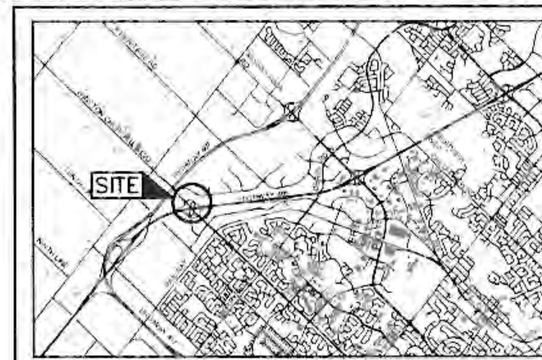
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No.
WO No. 07-20021

WINSTON CHURCHILL BLVD. UNDERPASS
HIGHWAY 401 WIDENING
SOIL STRATA

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE 1.5 0 1.5 3 km

LEGEND

- Borehole — Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ∇ WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
1	205.9	4828976.1	281569.3
2	205.9	4828975.2	281555.9
3	208.5	4829004.7	281516.7
4	208.6	4828937.2	281573.2
5	212.0	4828940.2	281605.0
6	205.9	4828974.7	281540.3
7	211.7	4829009.9	281545.3

NOTES

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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 URS, drawing file nos. ACAD-X-base1_to_Trafalgar.dwg and ACAD-Aerials_MTO ROW_ Property Boundaries..dwg, received August 17, 2011 and August 29, 2011

L.C. COYNE
LICENSED PROFESSIONAL ENGINEER
PROVINCE OF ONTARIO

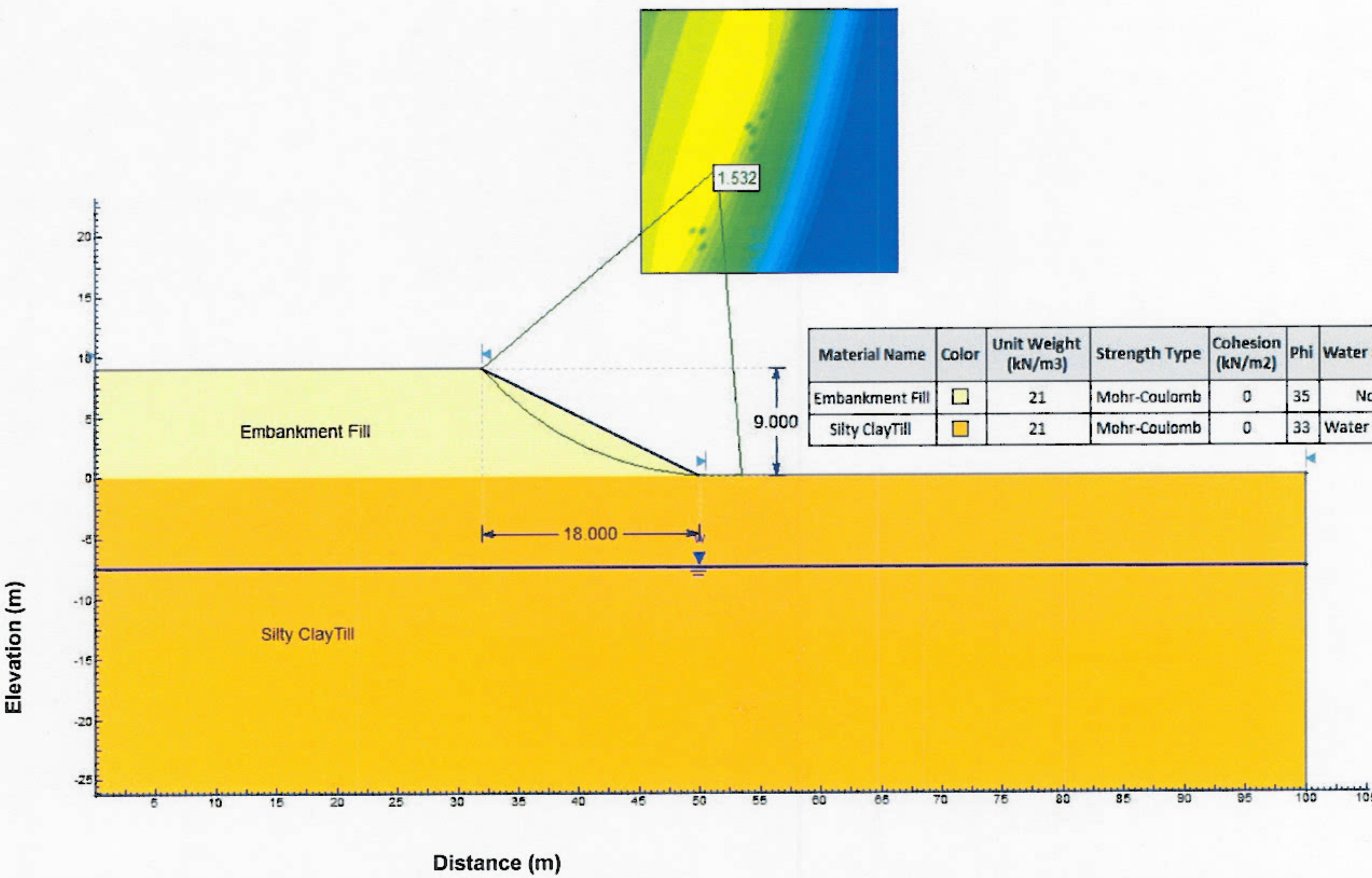
NO.	DATE	BY	REVISION

Geocres No. 30M12-358

HWY. 401		PROJECT NO. 10-1111-0040	DIST.
SUBM'D. NL	CHKD. LCC	DATE 12/19/2012	SITE:
DRAWN: JFC	CHKD. NL	APPD. LCC	DWG. 2

Static Global Stability – Winston Churchill Boulevard Approach Embankment

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Water Surface	Hu Type	Ru
Embankment Fill		21	Mohr-Coulomb	0	35	None		0
Silty ClayTill		21	Mohr-Coulomb	0	33	Water Surface	Constant	

Date: December 2011
Project No: 10-1111-0040

Analysis By: NLP **Reviewed By:** LCC

Golder Associates

APPENDIX A

Previous Geotechnical Investigation
MTO GEOCRES No. 30M12-145

RECORD OF BOREHOLE No 1

W P 167-77-02 LOCATION Co-ords. N 15 842 366, E 923 695 ORIGINATED BY B.L.

DIST 6 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY T.J.K.

DATUM Geodetic DATE October 17, 1979 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) (W_p – W – W_L)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
675.5	Roadway Surface							
0.0	Granular Fill							
672.0 / 3.5	Silty Clay With Sand Trace of Gravel Low Plasticity (Glacial Till) Stiff to Very Stiff — Weathered, Brownish Grey / Grey							
		1	SS	9		670		
		2	SS	19				5-27-43-25
		3	SS	28		660		
		4	SS	20				
		5	SS	30		650		
		6	SS	24				
	Hard	7	SS	105/9"		640		
	- Excess of Gravel	8	SS	90/4"				3-35-43-17
634.6 / 40.9	End of Borehole							

Note: Localized seepage into B.H. encountered at a depth of 5.0 feet.

Dynamic cone penetration resistance plot: 20 40 60 80 100. Shear strength: O Unconfined, + Field Vane, ● Quick Triaxial, × Lab Vane. Water content scale: 10 20 30. Unit weight γ.

Ontario HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2

W P 167-77-02 LOCATION Co-ords. N 15 842 363, E 923 [illegible] ORIGINATED BY B.L.

DIST 6 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY T.J.K.

DATUM Geodetic DATE October 17, 1979 CHECKED BY [illegible]

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
675.5	Roadway Surface						
0.0	Granular Fill						
672.0 / 3.5	- occasional sand seams	1	SS	39	670		
	Hard						
	Grey Silty Clay With Sand Trace of Gravel Low Plasticity (Glacial Till)	2	SS	17			7-27-46-20
		3	SS	14	660		
	Stiff to Very Stiff	4	SS	16			
		5	SS	18	650		
		6	SS	22			
	Hard	7	SS	92	640		
	- Excess of Gravel						
634.6		8	SS	60/4"			
40.9	End of Borehole						

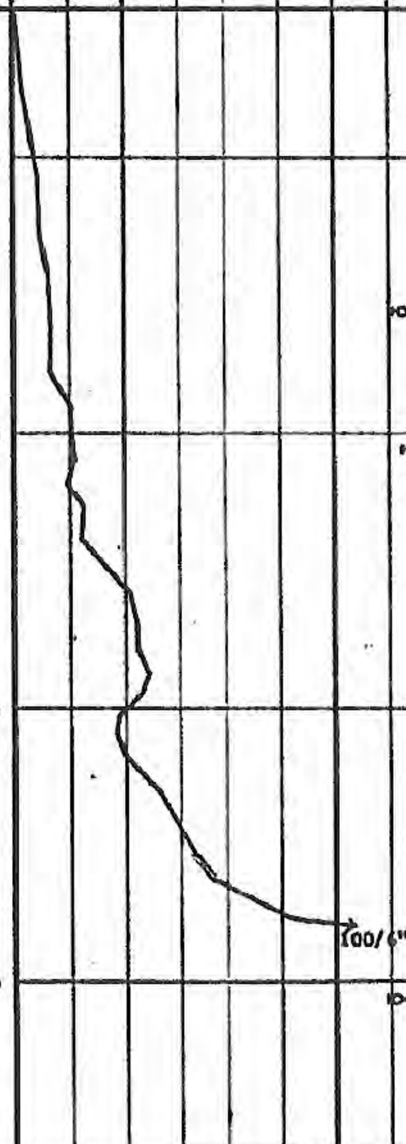
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ



RECORD OF BOREHOLE No 3

W P 167-77-02 LOCATION Co-ords. N 15 842 462, E 923 520 ORIGINATED BY T.J.K.
DIST 6 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY T.J.K.
DATUM Geodetic DATE January 7, 1980 CHECKED BY [signature]

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
684.0	Ground Surface							
0.0	Silty Clay with Sand Trace gravel		1	SS	13		680	
	Sand & Gravel Seam		2	SS	37			42-42-11-5
	Weathered		3	SS	29			
			4	SS	34			8-20-49-23
	Grey Silty Clay With Sand Trace of Gravel Low Plasticity (Glacial Till) Very Stiff to Hard		5	SS	33		670	
			6	SS	17			
			7	SS	19		660	
			8	SS	16			
			9	SS	26	▽	650	
	Hard -Excess of Gravel						640	
637.5			10	SS	53			28-22-28-11
46.5	End of Borehole							

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 (%) 10 20 30
UNIT WEIGHT γ

HIGHWAY ENGINEERING DIVISION ENGINEERING MATERIALS OFFICE · SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 4

W P 167-77-02 LOCATION Co-ords. N 15 842 238, E 923 706 ORIGINATED BY T.J.K.

DIST 6 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY T.J.K.

DATUM Geodetic DATE January 7, 1980 CHECKED BY [signature]

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
684.4	Ground Surface					
0.0	Weathered Silty Clay with Sand Trace of Gravel Low Plasticity Firm to Hard	1	SS	14		
		2	SS	4	680	7-29-43-21
		3	SS	37		5-27-49-19
		4	SS	88		
	Brown / Grey	5	SS	65		4-32-46-18
		6	SS	31	670	
	Silty Clay With Sand Trace of Gravel Low Plasticity (Glacial Till)	7	SS	13		
	Stiff to Very Stiff	8	SS	18	660	
		9	SS	21		
		10	SS	21	650	
	Hard					
	- Occasional Fine Sand Seams	11	SS	68	640	
633.9		12	SS	67		37-21-32-10
50.5	End of Borehole					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (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 W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 5

W P 167-77-07 LOCATION Co-ords. N 15 842 250; E 923 812 ORIGINATED BY PJS

DIST 6 HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY PJS

DATUM Geodetic DATE April 28th & 29th, 1980 CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L – WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
695.4	Ground Surface										
0.0	Fill Silty Clay of Low Plasticity Stiff		1	SS	4		690				
			2	SS	9						
683.4			3	SS	5						
12.0	Silty Clay (Low Plasticity) With Sand Trace of Gravel Very Stiff to Hard (Glacial Till)		4	SS	12						
			5	SS	16		680				
			6	SS	34						
			7	SS	79						
			8	SS	70						
			9	SS	42		670				
			10	SS	29						
			11	SS	29						
			12	SS	25						
			13	SS	25		660				
			14	SS	19						
			15	SS	21		650				
			16	SS	19						
			17	SS	35		640				
			18	SS	110/ 6"						
							630				
			19	SS	100/ 8"						
			20	SS	61		620				
	Occasional Layers and Pockets of Sandy Silt Very Dense		21	SS	100/ 7"						
			22	SS	100/ 7"		610				
			23	SS	116						
			24	SS	100/ 8"		600				
594.7			25	SS	100/ 8"						
100.7	End of Borehole										

$+^3$, $\times^5$: 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 7

W P 167-77-02 | LOCATION Co-ords. N 15 842 478; E 923 616 | ORIGINATED BY PJE
DIST 6 HWY 401 | BOREHOLE TYPE Solid Auger | COMPILED BY PJE
DATUM Geodetic | DATE May 1st to 5th, 1980 | CHECKED BY

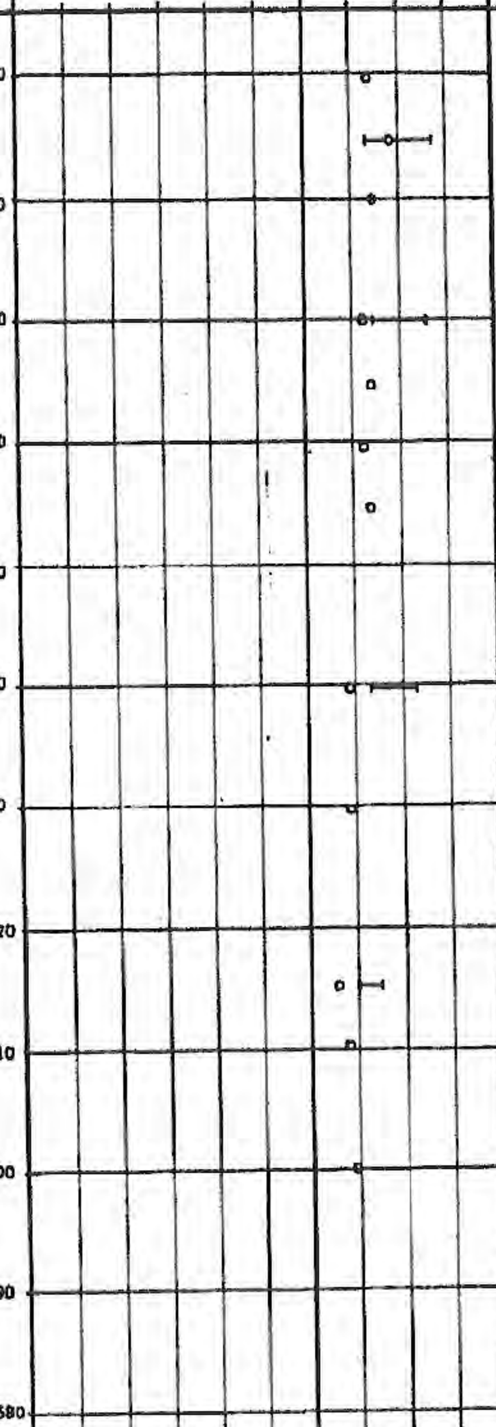
Soil Profile

ELEV / DEPTH	DESCRIPTION
694.6	Ground Level
0.0	Fill Silty Clay of Low Plasticity Stiff to Very Stiff
682.6 / 12.0	Silty Clay (Low Plasticity) With Sand Trace of Gravel Very Stiff to Hard (Glacial Till) Occasional Layers and Pockets of Sandy Silt Very Dense
583.6 / 111.0	Shale Bedrock
575.2 / 119.4	End of Borehole

Samples

NUMBER	TYPE	'N' VALUES	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	
2	SS	11	
3	SS	35	
4	SS	48	
5	SS	38	
6	SS	26	
7	SS	23	
8	SS	24	11 23 43 23
9	SS	25	
10	SS	40	
11	SS	54	
12	SS	71	
13	SS	100/8"	
14	SS	50	
15	SS	100/6"	
16	SS	100/6"	
17	SS	100/5"	
18	SS	100/12"	17 45 31 7
19	SS	100/7"	
20	SS	100/2"	
21	BXL RC	85% REC	

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (690, 680, 670, 660, 650, 640, 630, 620, 610, 600, 590, 580); 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 (%) 10 20 30; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



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

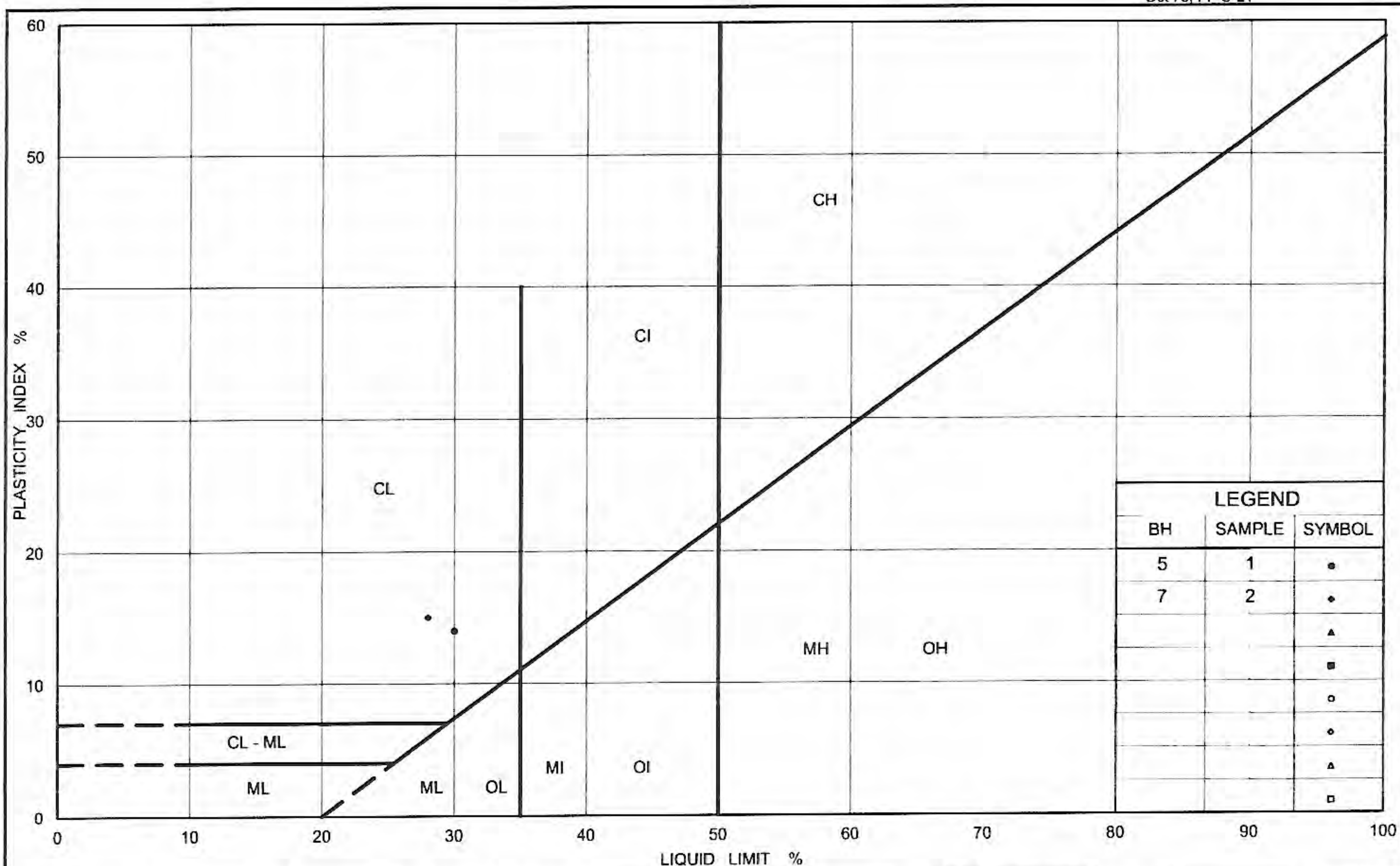
$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX B

Laboratory Test Results

Oct 75, FF-S-21



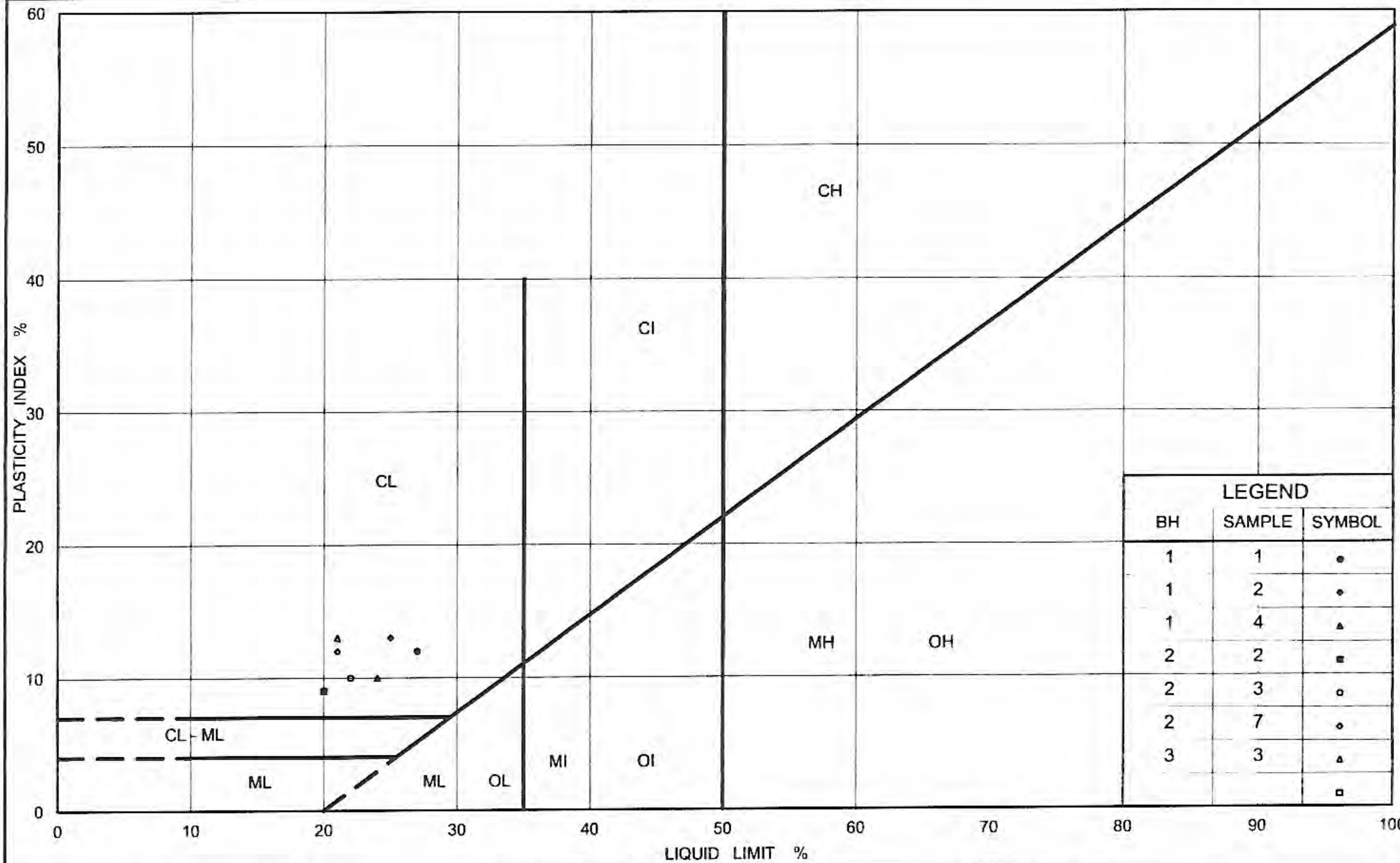
Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay Fill

Figure No. B1	
Project No. 10-1111-0040	
Checked By:	

Oct 75, FF-S-21



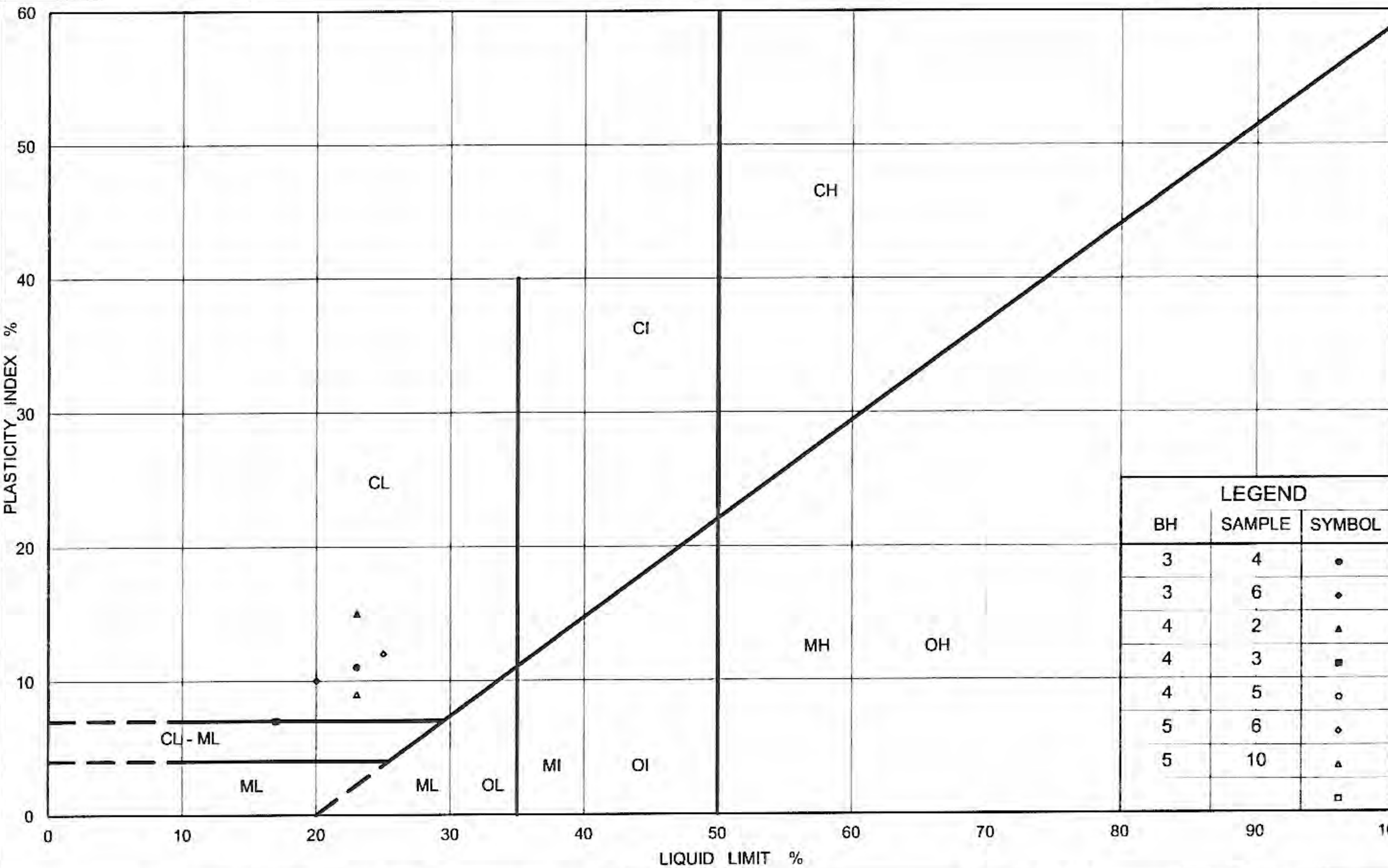
Ministry of Transportation Ontario	PLASTICITY CHART Silty Clay Till

Figure No. B2

Project No. 10-1111-0040

Checked By:

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
3	4	●
3	6	◆
4	2	▲
4	3	■
4	5	○
5	6	◇
5	10	△
		□

Ministry of Transportation

Ontario

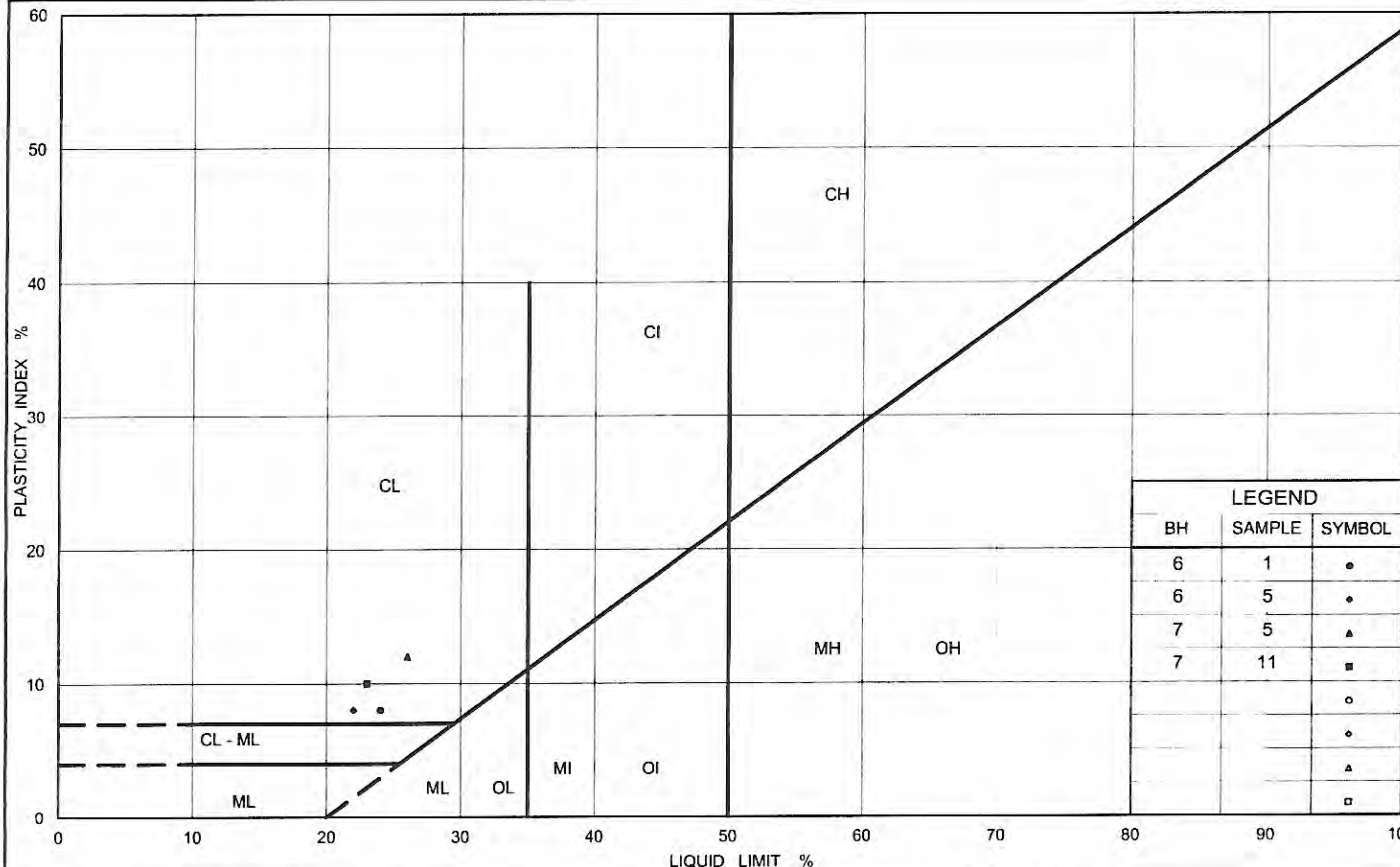
PLASTICITY CHART
Silty Clay Till

Figure No. B3

Project No. 10-1111-0040

Checked By:

Oct 75, FF-S-21



Ministry of Transportation

Ontario

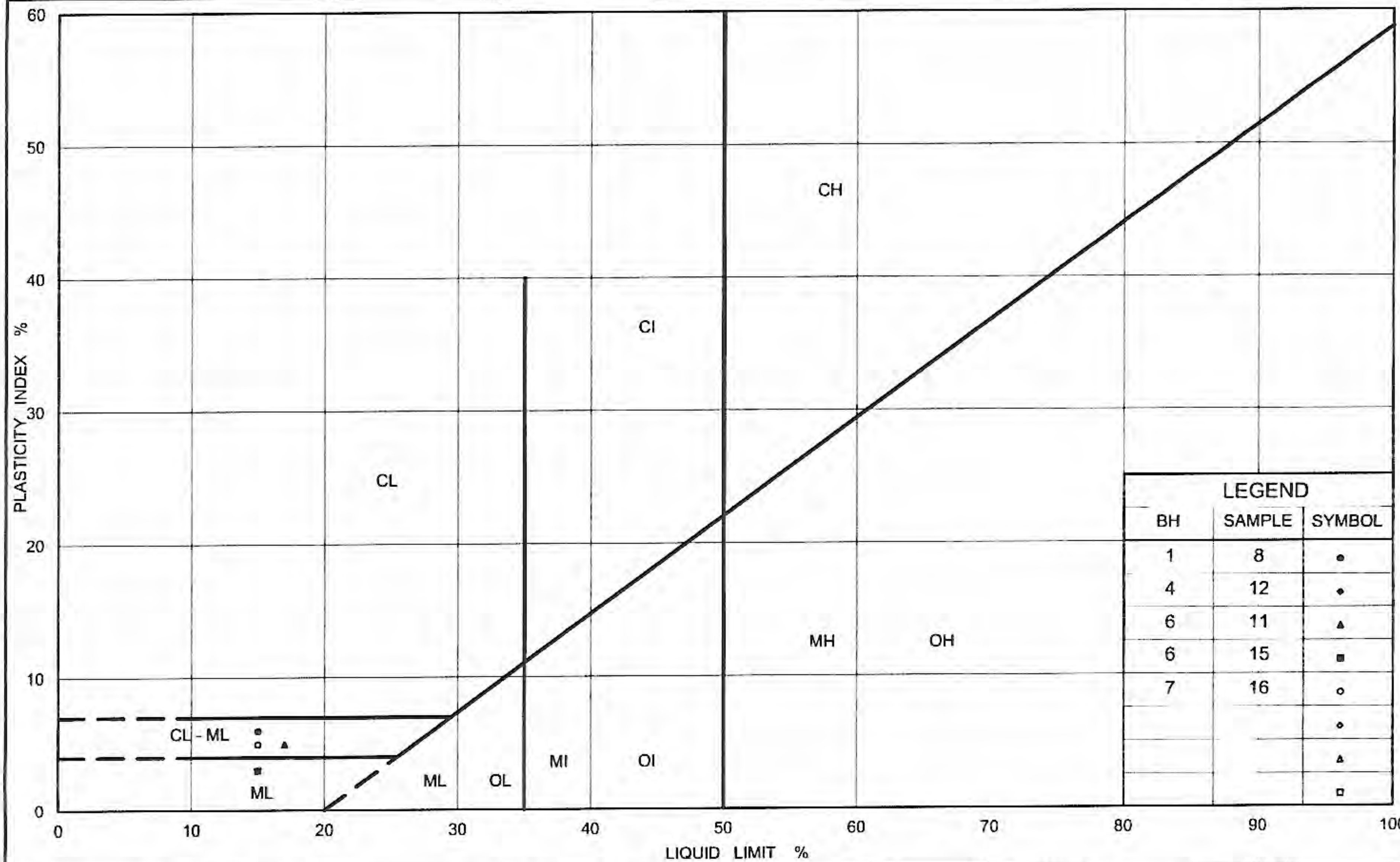
PLASTICITY CHART
Silty Clay Till

Figure No. B4

Project No. 10-1111-0040

Checked By:

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
1	8	●
4	12	◆
6	11	▲
6	15	■
7	16	○
		◇
		△
		□

Ministry of Transportation

Ontario

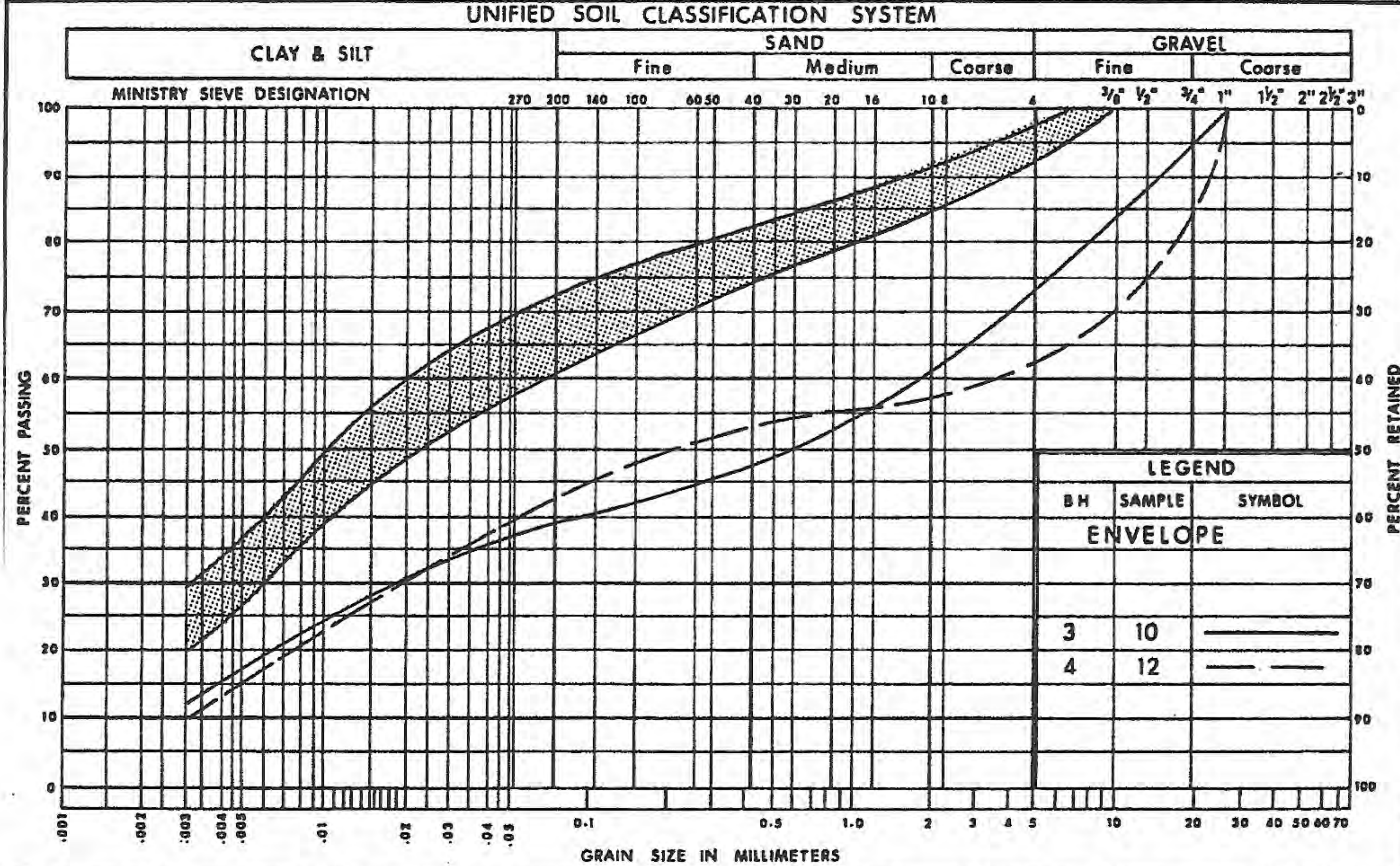
PLASTICITY CHART
Silt to Clayey Silt Till

Figure No. B5

Project No. 10-1111-0040

Checked By:

Oct 75, FF-S-22



Ministry of Transportation and Communications

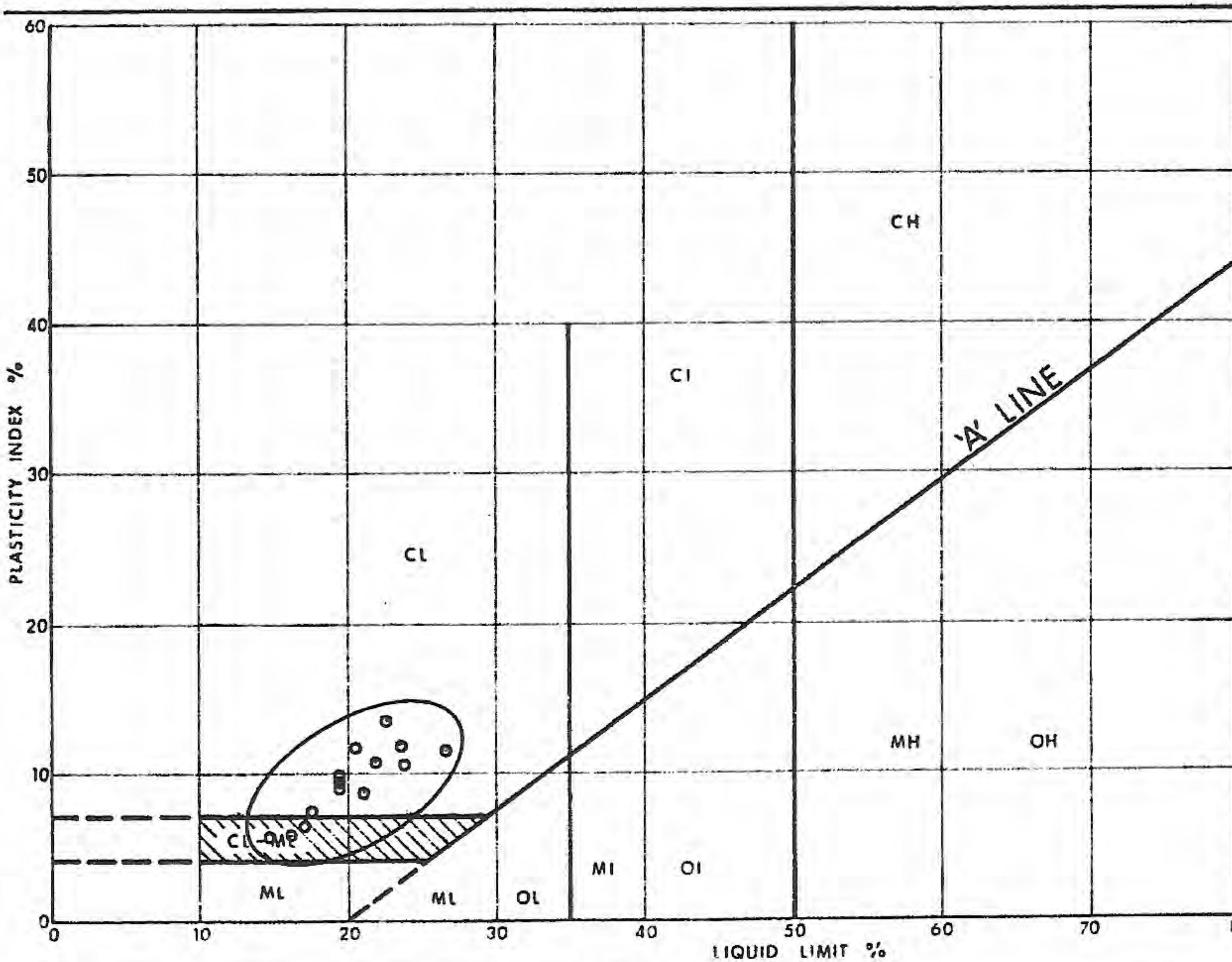
Ontario

GRAIN SIZE DISTRIBUTION
SILTY CLAY
WITH SAND TRACE OF GRAVEL

FIG No 1

W P 167-77-02

Oct 75, FF-5-21



LEGEND		
BH	SAMPLE	SYMBOL

Ministry of Transportation and Communications
Ontario

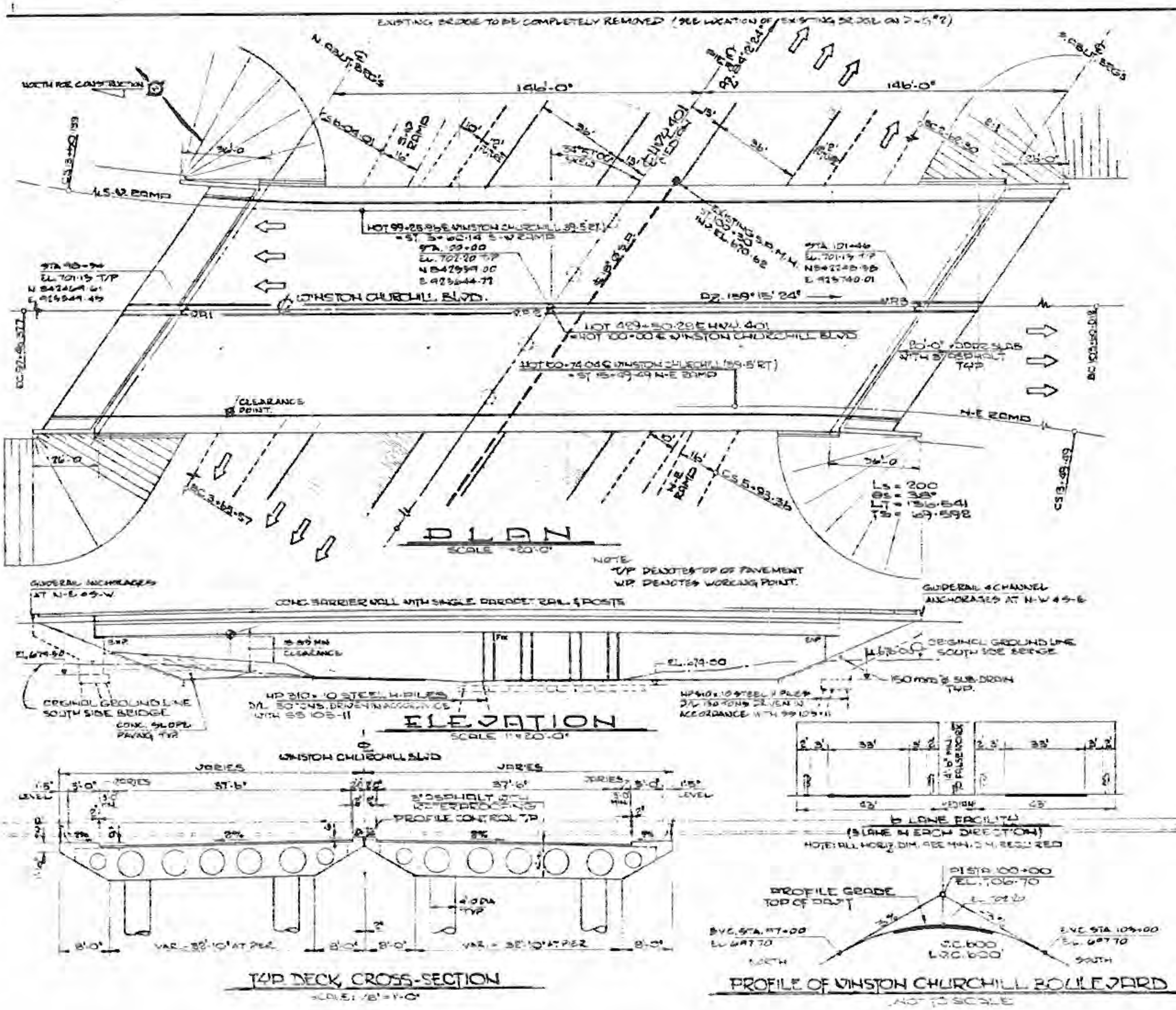
PLASTICITY CHART
SILTY CLAY
WITH SAND TRACE OF GRAVEL LOW PLASTICITY

FIG No 2
W P 167-77-02

APPENDIX C

Structural Drawings for Existing Underpass

DIST. 6
CONT No
WP No 167–
WINSTON CHURCHILL BLVD
GENERAL ARRANGEMENT



GENERAL NOTES

CLASS OF CONCRETE

DECK & PIERS
ABUTMENTS, WINGWALLS,
BARRIER WALLS & PIER & ABUTMENT FOOTIN
REMAINDER

GRADE OF REINFORCING ST

REINFORCING SHALL BE GRADE 400
BAR MARKS WITH THE SUFFIX 'C'
SHALL BE COATED BARS.

CLEAR COVER TO REINFORC

FOOTINGS & ABUTMENTS — 3"
COLUMNS — 3
DECK: TOP 2" BOTT. 2"
BARRIER WALLS — AS NOTED
APPROACH SLABS — 2"

CONSTRUCTION NOTES

- THE CONTRACTOR IS RESPONSI THE BEARING SEATS DEAD LEVEL TO TH ELEVATIONS WITH A TOLERANCE O
- NO CONCRETE SHALL BE PLACED ABO BEARING SEATS UNTIL THE CONCRETE PLACED, STRESSED AND GROUTED

CONCRETE QUANTITIES

CONCRETE QUANTITIES ARE LISTED IN APPROPRIATE CONCRETE LUMP SUM T
CONCRETE IN ABUTMENTS & WINGWAL
CONCRETE IN PIER
CONCRETE IN PRESTRESSED CONC. BE
CONCRETE IN APPROACH SLABS
CONCRETE IN BARRIER WALL
CONCRETE IN SLOPED PAVING

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. CONSTRUCTION STAGES
3. BOREHOLE LOCATION & SOIL STR
4. FOOTING & PIER DETAILS
5. FOOTING & PIER REINFORCEMENT
6. ABUTMENT LAYOUT
7. NORTH ABUTMENT REINFORCING
8. SOUTH ABUTMENT REINFORCING
9. DECK LAYOUT
10. LONGITUDINAL CABLE DETAILS
11. TRANSVERSE CABLE DETAILS
12. CABLE DETAILS & SCREED ELEVA
13. DECK REINFORCING I
14. DECK REINFORCING II
15. BARRIER WALL ON SIDEWALK
16. RAILING FOR BARRIER WALL
17. 20 FT APPROACH SLAB
18. DETAILS OF CONCRETE SLOPE PA
19 TO 22 STANDARD DETAILS I TO IV
23. BRIDGE DATE & SITE NUMBER ON
24. AS CONSTRUCTED ELEV. & DIM.

FOR REDUCED PLAN
USE SCALE BELOW
3 INCHES ON ORIGINAL PLAN

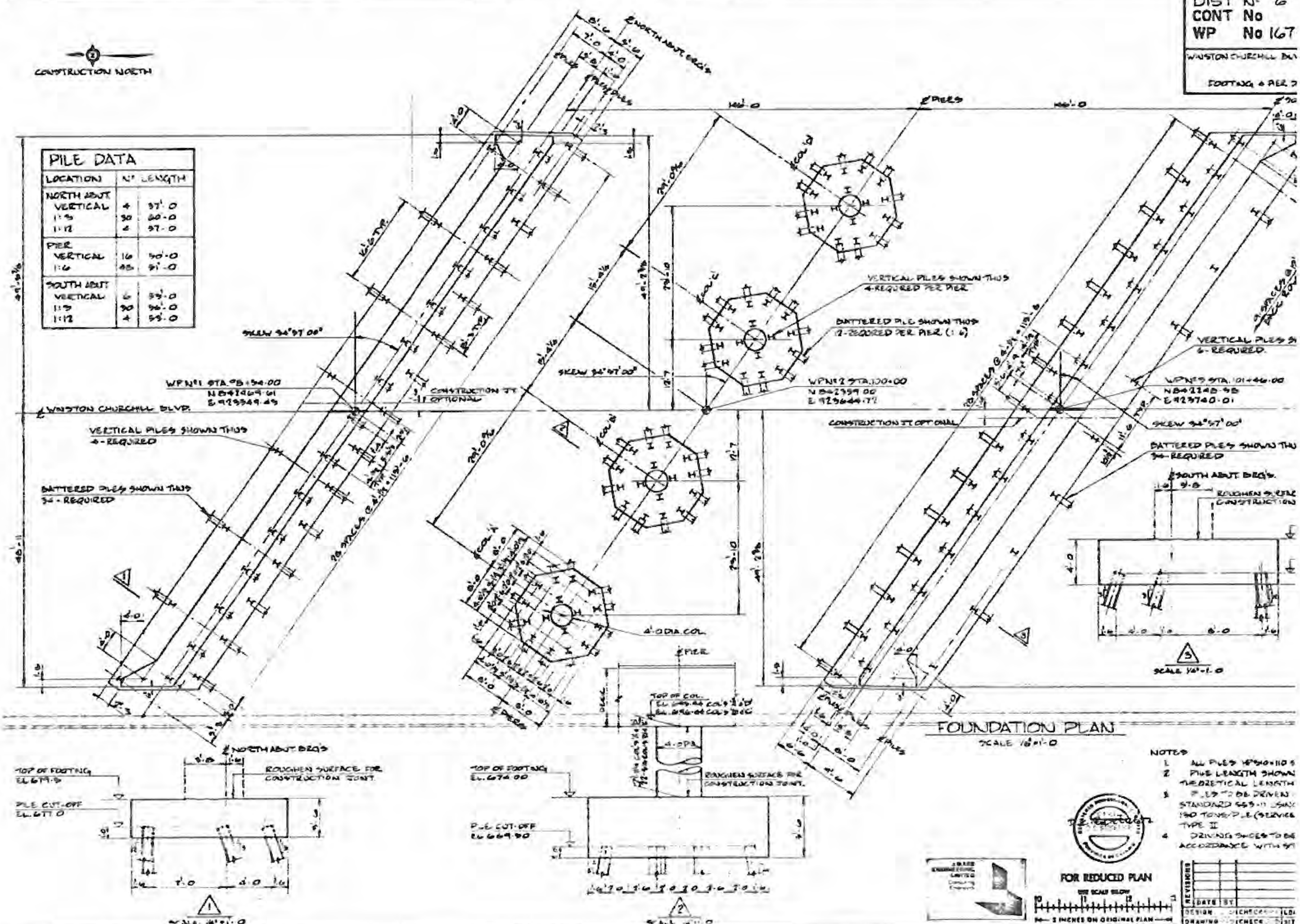
REVISIONS	DATE	BY
DESIGN		CHECK
DRAWING		CHECK

DIST Nº 6
CONT No
WP No 167

WINSTON CHURCHILL BLV

FOOTING & PIER 2

PILE DATA		
LOCATION	Nº	LENGTH
NORTH ABUT.		
VERTICAL	4	37'-0
1:3	30	60'-0
1:12	4	57'-0
PIER		
VERTICAL	16	50'-0
1:6	48	51'-0
SOUTH ABUT.		
VERTICAL	6	39'-0
1:3	30	56'-0
1:12	4	55'-0



FOUNDATION PLAN

SCALE 1/8"=1'-0

NOTES
1. ALL PILES HP310×110 S
2. PILE LENGTH SHOWN THEORETICAL LENGTH
3. PILES TO BE DRIVEN STANDARD SS3-11 USIN 130 TONS/PILE (SERVICE TYPE II
4. DRIVING SHOES TO BE ACCORDANCE WITH

FOR REDUCED PLAN
USE SCALE BELOW
10 11 12 13
2 INCHES ON ORIGINAL PLAN

REVISIONS
DATE BY
DESIGN CHECKED
DRAWING CHECK