THURBER ENGINEERING LTD.

FOUNDATION DESKTOP STUDY REPORT
PRELIMINARY DESIGN AND ENVIRONMENTAL ASSESSMENT
BRUCE TRAIL PEDESTRIAN UNDERPASS AT HIGHWAY 6
STRUCTURE REHABILITATION
HAMILTON, ONTARIO
W.O. #16-20004
SITE 36-515

GEOCRES NO. 30M5-349

Latitude: 43.307336°
Longitude: -79.910977°

Report

to

AECOM

Date: November 29, 2022
File: 25963

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1.0 INTRODUCTION

This report presents the results of a foundation desktop study carried out by Thurber Engineering Ltd. (Thurber) for the preliminary design and environmental assessment of the Bruce Trail Pedestrian underpass rehabilitation at Highway 6 in Hamilton, Ontario.

This Phase 1 study is carried out for planning, structure evaluation and preliminary design purposes only. As part of the Phase 1 scope, a desktop study is to be carried out based on currently available subsurface and foundation information. Where this study determines that the existing information is insufficient to complete the preliminary design, additional foundation investigation and assessment will be recommended for completing Phase 1. It is understood that the budget for this additional investigation, should it be required, is to be drawn from the Phase 2 contingency upon approval by MTO.

Thurber was retained by AECOM to carry out this Phase 1 study under the Ministry of Transportation Ontario (MTO) Assignment Number 2016-E-0027.

This site is a part of the overall Highway 403 and Highway 6 Interchange Improvements project where 14 bridges, 3 structural culverts and 15 retaining walls are planned to be replaced, reconstructed or rehabilitated.

It is a condition of this report that Thurber's performance of its professional services be subject to the attached Statement of Limitations and Conditions.

The following references and drawings are available for this site.

- Foundation Investigation and Design Report, Bruce Trail Pedestrian Tunnel, Highway 6 Widening between Highway 403 and 5, W.P. 19-95-07, Report 001-1141F-4, Geocres 30M05-245, prepared by Golder Associates, dated July 2006. (Reference 1).
- Archive drawings, Bruce Trail Pedestrian Underpass, Contract No. 2005-2019, W.P. 19-95-07, prepared by URS, dated July 2005. (Reference 2).
  - General Arrangement, Sheet 275
  - Borehole Location and Soil Strata, Sheet 276
  - Culvert Details, Sheet 277

2.0 SITE AND PROJECT DESCRIPTION

The existing pedestrian underpass is located at the crossing of Highway 6 and Bruce Trail, approximately 775 m south of the Highway 5 and Highway 6 intersection, at Station 12+320 in Hamilton, Ontario. The existing pedestrian underpass (tunnel) carries Bruce Trail to cross under Highway 6 NBL (northbound lanes) and SBL (southbound lanes).

Highway 6 in the vicinity of the site generally runs in a north-south orientation. The north end of Old Guelph Road is located approximately 180 m west of the site. The lands adjacent to the site are generally vacant, and covered with dense vegetation including trees and bushes. Bruce Trail extends in both west and east directions from the pedestrian underpass.

The existing pedestrian underpass consists of a rigid frame concrete box structure. The width and the height of the concrete box structure is 3.7 m and 3.6 m, respectively, and the length is 50.5 m. At the site, the Highway 6 grade is at about Elevation 190.0. At the east end of the pedestrian underpass, the surrounding natural grade is at about Elevation 184.5, resulting in an embankment height of about 5.5 m. Archive drawing indicates that the embankment was constructed with a slope configuration of 3H : 1 V. At the west end of the pedestrian underpass, the ground slopes down to about Elevation 177.5. The upper 5.5 m of the embankment has a slope configuration of 3H : 1V, while the slope configuration of the lower 7.0 m of the embankment, just beyond the underpass entrance/access, is at an inclination of 2H : 1V. Stairs are located at each end of the pedestrian underpass to provide access to the Bruce Trail.

Selected photographs of the site are included in Appendix C.

Based on the preliminary GA drawing dated April 2022, the proposed rehabilitation of the existing structure includes:

- Widening of the existing Highway 6 to the east and west sides to accommodate one additional lane for each of the NBL and SBL.
- Four new retaining walls (wingwalls) are planned at this site, one at each corner of the existing pedestrian underpass.
- Removal and repair of deteriorated and delaminated concrete from soffit, walls and bottom slab.
- Removal and replacement of slab at the west entrance.
- Cleaning of existing graffiti, and application of anti-graffiti coating.

If this rehabilitation program is changed or modified at any stage of the project, the comments provided in this Desktop Study should be revised accordingly.

The project area is situated within the physiographic region known as the Niagara Escarpment, which forms a north-south trending strip, and is a major topographic break in the bedrock between the carbonate Amabel Formation to the west and the soft sediments of the Queenston Formation to the northeast. At many locations, the Queenston Formation consists of up to 1.2 m of very weathered bedrock (red clay) which grades downward into typical brick-red shale and often with green mottling. Thin to medium beds of grey-green and reddish argillaceous limestone are present in most sections. The Queenston shale is overlain by Halton Till in the area of the site. The Halton Till is a red clay to clayey silt till and is exposed in the form of a till plain extending from Lake Ontario southward to the Niagara escarpment.

3.0 SITE OBSERVATIONS

A site reconnaissance visit was conducted by a Thurber Senior Geotechnical Engineer on March 27, 2022 to observe conditions related to the foundation performance of the existing pedestrian underpass and approaches. The following observations have been noted during our site visit:

- There was no visible sign of settlement or distress along the pedestrian underpass alignment.
- The existing embankments appeared to be in good condition. The side slopes did not exhibit obvious sign of instability or bulging.

- The concrete structure shows no signs of structural distress.
- Few moderate longitudinal and transverse cracks were noted on the Highway 6 pavement in the vicinity of the structure.

Selected photographs of the site taken during the site visits are presented in Appendix C.

4.0 SUBSURFACE CONDITIONS

A foundation investigation was carried out in 2000 and 2002 (Reference 1) for the design and construction of the existing Bruce Trail pedestrian underpass. The investigation consisted of drilling and sampling four (4) boreholes (numbered E1, B1, B2 and B3). Borehole E1 was drilled on the west shoulder of Highway 6, and Boreholes B1 to B3 were drilled along the proposed underpass alignment. Boreholes B1 and B3 were advanced to 2.8 m and 4.3 m depth, respectively, and Boreholes B2 and E1 to 12.6 m and 12.8 m depth, respectively.

Record of Borehole Sheets of Boreholes from the previous investigation and borehole location plan are included in Appendix A.

In general, the soil stratigraphy encountered at the site, during the investigations conducted in 2000 and 2002, consisted of surficial topsoil or asphalt and embankment fill overlying native clayey silt, sands/silts and clayey silt till.

A 700 mm thick layer of topsoil was contacted surficially in Borehole B1. Pavement structure consisting of asphalt over granular fill (sand and gravel) was contacted at the surface in Borehole B2, drilled from Highway 6 shoulder. The pavement structure was 600 mm thick.

Embankment fill was contacted below the pavement structure in Boreholes B2 and E1, and surficially in Borehole B3. The embankment fill consisted of layers of cohesive and cohesionless soils. The cohesive fill was described as red-brown to reddish-grey clayey silt to silty clay containing trace to some sand, gravel, and shale and limestone fragments. Cobbles and boulders were encountered within the cohesive fill. The cohesionless fill consisted of layers of brown to red-brown gravel, silty sand, and cobbles containing some sand and silt and trace clay. The thickness of the embankment fill was 8.7 m and 10.2 m in Boreholes B2 and E1, respectively. The clayey silt fill was firm to hard in consistency, with SPT ‘N’ values ranging from 6 to 66 blows per 0.3 m of penetration. SPT ‘N’ values measured in the cohesionless fill ranged from 14 to 27 blows per 0.3 m of penetration indicating a compact state. A 1.5 m thick layer of clayey silt fill containing some sand and trace to some gravel was contacted surficially in Borehole B3. It has a firm to very stiff consistency with measured SPT ‘N’ values ranging from 5 to 19 blows per 0.3 m of penetration.

A layer of native mottled brown to red-brown clayey silt containing trace to some sand, trace gravel and silty sand pockets was contacted below the topsoil in Borehole B1 at Elevation 179.1, and below the embankment fill at Elevation 178.7 in Borehole E2. The thickness of the clayey silt was 800 mm and 600 mm respectively. SPT ‘N’ values in the clayey silt varied from 2 to 8 blows per 0.3 m of penetration indicating a soft to stiff consistency. A 300 mm thick layer of sand and silt containing trace gravel was contacted below the clayey silt in Borehole B1 at 1.5 m depth (Elevation 178.3). In Borehole B3, a 2.0 m thick layer of brown to red-brown silty sand to sand, containing trace gravel was contacted below the fill at 1.5 m depth (Elevation 184.6). Reference 1 described this layer as alluvium associated with the Grindstone Creek valley, located below the existing residential driveway embankment fill. This alluvial material was in a loose to compact condition based on measured SPT ‘N’ values ranging from 7 to 22 blows per 0.3 m of penetration.

Below the embankment fill and native surficial soils, a deposit of brown to red-brown clayey silt till was contacted at 1.8 m and 3.5 m depth (Elevations 178.0 and 182.6) in Boreholes B1 and B3, and at 9.3 and 10.8 m depth (Elevations 180.1 and 178.1) in Boreholes B2 and E1, respectively. The clayey silt till contained trace to some sand, gravel and shale and limestone fragments. SPT ‘N’ values measured in the clayey silt till typically varied from 19 to 73 blows per 0.3 m of penetration indicating a very stiff to hard consistency. An occasional value of 42 blows for less than 0.3 m of penetration indicated the presence of shale and limestone fragments. Moisture content in the cohesive till ranged from 12 to 15 percent. All four boreholes were terminated within the clayey silt till.

Groundwater levels in the open boreholes were measured at 1.5 m and 12.3 m depth (Elevations 178.3 and 177.1) in Boreholes B1 and B2, respectively.

5.0 EXISTING FOUNDATIONS

It is understood that construction of this pedestrian underpass was associated with the then planned widening of Highway 6 between Highway 403 and Highway 5 in Hamilton, Ontario. At the time of preparation of the previous foundation report (Reference 1), placement of up to 5.0 m of new fill to widen the Highway 6 embankment was anticipated at the west end of the proposed pedestrian underpass.

Reference 1 recommended that the pedestrian underpass be constructed on the then existing embankment fill at the east end, and on new embankment fill at the west end at about Elevation 185.0. It was also recommended that the proposed underpass be designed based on a Factored Geotechnical Resistance at ULS of 250 kPa and a Geotechnical Resistance at SLS (less than 25mm settlement) of 200 kPa. Due to the highly variable subgrade conditions encountered and anticipated along the length of the underpass, it was recommended to place at least 500 mm of OPSS Granular A bedding below the base of the concrete box.

Archive drawings (Reference 2) show that the invert levels of the concrete box are at Elevations 184.7 and 185.0 at the west and east ends, respectively. The concrete box was founded on a 500 mm thick layer of compacted Granular A material.

6.0 PROPOSED RETAINING WALLS

As result of the proposed widening of Highway 6, four walls are proposed at this site, one at each corner of the existing pedestrian underpass. Based on a preliminary GA drawing dated April 2022, the north walls will be parallel to Highway 6 and the south walls will be at an approximate 45° skew to the centreline of the existing pedestrian underpass. The proposed lengths of the retaining walls are as follows:

Length of Proposed Retaining Walls	NE	NW	SE	SW
	11.5 m	22 m	6 m	10 m

The boreholes from Reference 1 did not provide subsurface information for the northwest quadrant of the existing underpass. Since new foundations will be required for the proposed retaining walls, additional boreholes will be required during detail design of the proposed retaining walls.

A foundation assessment for the proposed retaining walls, based on current information, has been carried out to provide preliminary information to the designers regarding the feasibility of the proposed foundations.

The designer should establish any additional loading imposed by the pedestrian underpass due to the proposed structural rehabilitation of the structure. Should the additional foundation loading be less than 10 percent of the existing loading and in accordance with current MTO practice, it is not anticipated that the proposed rehabilitation works for the underpass would have an impact on the existing structure, provided that the concrete box is structurally sound.

It is understood that the following retaining wall types are being considered at this site:

- Concrete retaining wall on spread footings
- Retained Soils Systems (RSS) walls

Archive drawings show that the invert levels of the structure at the west and east sides are at Elevations 184.7 and 185.0, respectively. If the base of the proposed retaining walls is to be close to these elevations, it is anticipated that the new walls on the east side of the pedestrian underpass will be founded on native compact silty sand to sand, while the proposed retaining walls on the west side will be founded on new embankment widening fill.

Concrete retaining wall on spread footings

For spread footings on the east side founded on native undisturbed, compact silty sand to sand below Elevation 184.5, it is assessed that the factored geotechnical resistance at Ultimate Limit States (ULS) is 300 kPa and the geotechnical resistance at Serviceability Limit State (SLS) is 200 kPa (corresponding up to 25 mm settlement).

For spread footings on the west side of the pedestrian underpass, there is insufficient information of the existing embankment fill and the underlying soils. For preliminary planning purposes and assuming well constructed embankment fill, a factored geotechnical resistance at Ultimate Limit States (ULS) of 225 kPa and a geotechnical resistance at Serviceability Limit State (SLS) of 150kPa may be assumed.

RSS walls

RSS walls used on this project must be specified to be “High Performance” and “High Appearance”. The soil conditions encountered near the wall alignments on the east side are generally suitable for the support of RSS walls. There is not enough data of the existing embankment fill and the underlying soils on the west side of the structure. For preliminary planning purposes, the geotechnical resistances quoted above may be used.

The RSS mass should be founded on a compacted granular pad as per MTO practices. Temporary protection (shoring) may be required to facilitate construction of this type of wall.

Based on the above assessment, it is considered feasible that the new retaining walls on the east side be supported on spread footings founded on native compact silty sand/sand, and walls on the west side be founded within embankment fill. Both concrete walls and RSS walls may be considered. A detail foundation investigation at the finalized wall alignments will be required to provide sufficient information for detail design.

7.0 EMBANKMENT DESIGN AND CONSTRUCTION

The April 2022 preliminary GA drawing indicates that the proposed embankment widenings will be about 5 m wide with side slopes at an inclination of 2H : 1V. Above the underpass, the embankment will be at an inclination 3H : 1V. A detail foundation investigation for the embankment widenings will be required to provide sufficient information for detail design.

In general, the new slopes should be designed to match the existing slope configuration with an inclination of 2H : 1V or flatter. Where applicable, benching of the existing earth slope surface should be carried out as per OPSD 208.010 in order to enhance the keying in of the new fill.

The subgrade for new fill is expected to be existing fill and native compact silty sand to sand, and possibly clayey silt till. No global stability issues are anticipated for the slopes at this site provided the approved new fill is placed and compacted in accordance with OPSS.PROV 206 and OPSS.PROV 501, and provided that all surficial vegetation, organics and topsoil, soft/loosened or wet soils and debris are removed from the proposed embankment footprints prior to fill placement.

It is recommended that all exposed permanent slope surfaces be vegetated and seeded in accordance with current MTO practice with reference to OPSS.PROV 804. Erosion protection measures must be provided for the slopes.

Drainage measures at the top of the embankment should be designed to minimize surface runoff and precipitation from flowing perpendicularly down the slope. This occurrence could increase surficial erosion on the embankment face.

Foundation settlement of the soil subgrade is expected to take place as the fill is placed and be completed by the end of construction. The magnitude of post construction settlement due to compression of the embankment fill itself depends on the type of materials to be used, but it is not anticipated to exceed 25 mm if the new fill is placed and compacted as outlined above.

8.0 BACKFILL TO RETAIING WALLS AND LATERAL EARTH PRESSURES

Backfill to the retaining walls should consist of free-draining granular material conforming to OPSS.PROV 1010 Granular A or B Type II specifications. Compaction should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501.

Earth pressures acting on the retaining walls may be assumed to impose a triangular distribution. For a fully drained backfill, the pressures should be computed in accordance with the CHBDC 2019 but are generally given by the expression:

$$p_h = K(\gamma h + q)$$

where
- p_h = horizontal pressure on the wall at depth h (kPa)
- K = earth pressure coefficient (see table below)
- γ = bulk unit weight of retained soil (see table below)
- h = depth below top of fill where pressure is computed (m)
- q = value of any surcharge (kPa)

Earth pressure coefficients for backfill are dependent on the material used as backfill. Recommended unfactored values are shown in the following Table 8.1. The at-rest coefficients should be employed for restrained walls. Active pressures should be used for unrestrained walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is generally recommended as it results in lower earth pressures acting on the wall.

Table 8.1 - Earth Pressure Coefficients (K)

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m^3		Embankment Fill $\phi = 30°, \gamma = 20.0$ kN/m^3	
	Horizontal Backfill	Sloping Backfill (2H : 1V)	Horizontal Backfill	Sloping Backfill (2H : 1V)
Active (Unrestrained Wall)	0.27	0.40	0.33	0.48
At-rest (Restrained Wall)	0.43	0.62	0.50	0.72
Passive	3.7	-	3.0	-

9.0 EXCAVATION AND GROUNDWATER CONTROL

All excavations must be carried out in accordance with OPSS.PROV 902 and the Occupational Health and Safety Act (OHSA). For the purposes of assessing excavation slope and temporary support requirements in compliance with the OHSA, the embankment fill and native compact silty sand/sand and clayey silt till are classified as Type 3 soils.

It is anticipated that excavations for construction of the new retaining walls will be carried out within the existing embankment fill and native silty sand/sand, and possibly within the very stiff to hard clayey silt till.

Reference 1 reported that groundwater levels were observed in the open boreholes at Elevations 177.1 and 178.3. Given the anticipated shallow excavations and the general layout of the site, it is anticipated that any excavation required to be carried out for construction of the new retaining walls will not extend below the groundwater level. Seepage or perched water from the approach fills and sand layers, as well as surface runoff and precipitation, are to be expected. All surface runoff should be diverted away from excavations.

The Contractor should be prepared to pump from properly filtered sumps to remove any seepage water or surface water collecting in an excavation. Unwatering must remain operational and effective until the excavation is backfilled.

The design of any dewatering system that may be required is the responsibility of the Contractor. Where required, construction will need to be carried out in conjunction with temporary protection.

Dewatering of all excavations should be carried out in accordance with OPSS.PROV 517, SP 517F01 Amendment to OPSS 517, November 2016 (issued July 2017).

10.0 TEMPORARY PROTECTION SYSTEMS

Temporary protection (shoring) systems will be required for construction of the new retaining walls in general accordance with OPSS.PROV 539. It is recommended that Performance Level 2 be specified.

The design of roadway protection should be the responsibility of the Contractor. All shoring systems must be designed by a Professional Engineer experienced in such designs.

11.0 ADJACENT STRUCTURES AND BURIED UTILITIES

It is recommended that the exact locations of any existing utilities that are present in the vicinity of the work areas be established by the designer and compared with the extent of the potential work zones related to the proposed rehabilitation of existing structure.

The utilities should not be undermined or damaged during rehabilitation of the existing pedestrian underpass. Relocation of, and/or special protective measures for, some or all of these affected utilities may be required.

12.0 INVESTIGATION FOR DETAIL DESIGN

References 1 and 2 are available from the GEOCRES library for this site. The existing subsurface information is insufficient to be used for detail design of the new works. It will be necessary to carry out additional site investigation and field testing to support the preparation of foundation design recommendations for detail design of the pedestrian underpass widening and retaining wall construction.

For detail design, it is recommended that Guidelines for MTO Foundation Engineering Services (Version 3.0 April 2022) be followed. For this pedestrian underpass widening and retaining wall construction, the minimum requirements are summarized as follows:

- One (1) borehole should be advanced at each widened side of foundation element to a minimum of 3 m below refusal. If bedrock is encountered, borehole shall be cored for a minimum depth of 3 m.
- One (1) borehole shall be advanced at each end of a retaining wall and at a maximum longitudinal spacing of 50 m. Boreholes shall be advanced to 3 m into a competent stratum or 10 m below the base of the wall, whichever is less. If bedrock is encountered, bedrock shall be cored for a minimum depth of 3 m.
- Additional requirements for RSS wall that may be applicable include boreholes behind and in front of the wall facing, minimum depth of boreholes along wall facing and retained zone area shall be 2H or 10 m below the base of RSS, minimum depth of H for boreholes along the fore-slope area where H is the proposed RSS wall height.

13.0 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Rocio Reyna, P,Eng. The report was reviewed by Sydney Pang, P.Eng. and P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THURBER'S WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS THURBER MAY EXPRESSLY APPROVE. Ownership in and copyright for the contents of the Report belong to Thurber. Any use which a third party makes of the Report, is the sole responsibility of such third party. Thurber accepts no responsibility whatsoever for damages suffered by any third party resulting from use of the Report without Thurber's express written permission.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

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Appendix A

Record of Borehole Sheets and Borehole Plan
(Geocres)

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u,s_u kPa	c_u,s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

S:\FINALDAT\ABBREV\2000\LOFA-D00.DOC

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s/\rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:
1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2
* density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

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Foundation Design

Golder Associates

RECORD OF BOREHOLE No B1

METRIC

PROJECT 001-1141F

W.P. 19-95-00 LOCATION N 4,796,226.2 E 271,421.8

DIST Central HWY 6 BOREHOLE TYPE Continuous Split-Spoon Sampling

DATUM Geodetic DATE Oct.15/02

ORIGINATED BY GM

COMPILED BY LCC

CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
179.8	GROUND SURFACE										
0.0	Topsoil	1	SS	4 2 2							
179.1 0.7	Clayey Silt, trace to some sand, trace gravel, containing silty sand pockets Soft to stiff Mottled brown to red-brown Moist to wet	2	SS	6 8	∇	179					
178.3											
178.0 1.8	Sand and Silt, trace gravel Loose Moist to wet	3	SS	9 19		178		3	46	33	18
	Clayey Silt, some sand, trace to some gravel, shale and limestone fragments (Till) Very stiff to hard Brown to red-brown Moist	4	SS	~~35~~ 42 15/.0?				3	19	48	30
177.0 2.8	END OF BOREHOLE					177					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Notes:

1. Water level in open borehole on completion of drilling at 1.5m depth (Elev.178.3m).

2. Borehole advanced using portable drilling equipment with a half-weight hammer. The SPT "N" values have been adjusted on this log to reflect the values that would be obtained using a standard-weight hammer.

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

$O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No B2

METRIC

PROJECT 001-1141F

W.P. 19-95-00

DIST Central HWY 6

DATUM Geodetic

LOCATION N 4,796,254.2 E 271,445.1

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

DATE Oct.18/02

ORIGINATED BY PKS

COMPILED BY LCC

CHECKED BY ASP

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
189.4	GROUND SURFACE
0.0	Asphalt
	Sand and Gravel (Fill) Compact Moist
188.8 / 0.6	Clayey Silt, some sand and gravel, trace organics (Fill) Stiff to very stiff Brown/black Moist
188.2 / 1.2	Clayey Silt to Silty Clay, some sand, some gravel, shale, limestone and siltstone fragments (Fill) Firm to very stiff Red-brown to reddish-grey Moist to wet Spoon bouncing on gravel/cobble in sample 6
183.9 / 5.5	Gravel, some sand and silt, trace clay to Silty Sand, some gravel, trace clay, containing clayey silt pockets (Fill) Compact Brown to red-brown Moist
180.1 / 9.3	Clayey Silt, some sand, trace to some gravel, shale and limestone fragments (Till) Hard Red-brown to reddish-grey Moist
176.8 / 12.6	END OF BOREHOLE

Note:

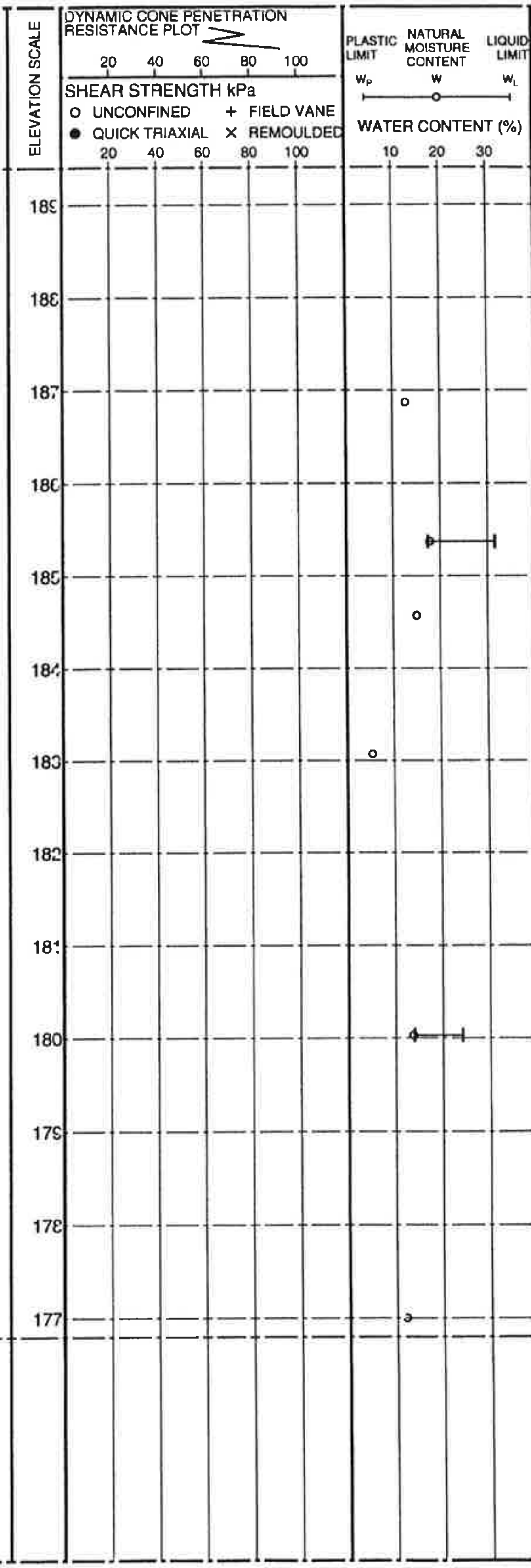
1. Water level in open borehole on completion of drilling at 12.3m depth (Elev.177.1m)

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	15	
2	SS	19	
3	SS	8	
4	SS	6	
5	SS	7	18 16 44 22
6	SS	42	
7	SS	14	64 14 14 6
8	SS	27	
9	SS	39	1 24 46 29
10	SS	33	
11	SS	38	

GROUND WATER CONDITIONS: ∇ (at approx. elev. 177)

UNIT WEIGHT γ kN/m³



+3, ×3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No B3

METRIC

PROJECT 001-1141F
W.P. 19-95-00
DIST Central HWY 6
DATUM Geodetic

LOCATION N 4,796,266.8 E 271459.9
BOREHOLE TYPE Continuous Split-Spoon Sampling
DATE Oct.11/02

ORIGINATED BY GM
COMPILED BY LCC
CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
186.1	GROUND SURFACE				186	
0.0	Clayey Silt, some sand, trace to some gravel (Fill) Firm to very stiff Mottled red-brown to green Dry	1	SS	5 13 19		
		2	SS	16 8	185	
184.6 1.5	Silty Sand to Sand, some silt, trace gravel Loose to compact Brown to red-brown Dry becoming moist below 2.3m depth	3	SS	7 12 18	184	2 74 13 11
		4	SS	22 19	183	
182.6 3.5	Clayey Silt with sand to some sand, some gravel (Till) Very stiff to hard Brown to red-brown Moist	5	SS	20 33 28		
		6	SS	39	182	4 20 46 30
181.8 4.3	END OF BOREHOLE					

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

Notes:

1. Borehole dry on completion of drilling operations.
2. Borehole advanced using portable drilling equipment with a half-weight hammer. The SPT "N" values have been adjusted on this log to reflect the values that would be obtained using a standard-weight hammer.

$+^3, \times^3$: Numbers refer to Sensitivity   O $^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No E1

METRIC

PROJECT 001-1141F			
W.P. 19-95-00	LOCATION N 4,796,235.1 E 271,437.6		ORIGINATED BY AS/GM
DIST Central HWY 6	BOREHOLE TYPE 108mm Diameter Solid Stem Augers		COMPILED BY LCC
DATUM Geodetic	DATE Nov.21&23/00		CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
188.9	GROUND SURFACE				
0.0	Gravel and cobbles, some sand and silt (Fill) Compact Brown Dry				
		1	SS	16	
		2	SS	26	
184.9 4.0	Clayey Silt, trace to some sand, gravel and shale fragments (Fill) Stiff to hard Brown Moist. Cobbles/boulders inferred from 5.2m to 6.1m and from 6.7m to 7.6m due to slow advance and grinding of augers.				
		3	SS	11	
		4	SS	66	
		5	SS	38	27 17 34 22
		6	SS	18	
178.7 10.2	Clayey Silt, trace sand, gravel and rootlets Very stiff Brown Moist				
178.1 10.8	Clayey Silt, some sand, trace gravel (Till) Very stiff to hard Brown Moist	7	SS	28	3 19 48 30
		8	SS	73	
176.1 12.8	END OF BOREHOLE. Note: Borehole dry on completion of drilling operations.				

Elevation scale: 188, 187, 186, 185, 184, 183, 182, 181, 180, 179, 178, 177

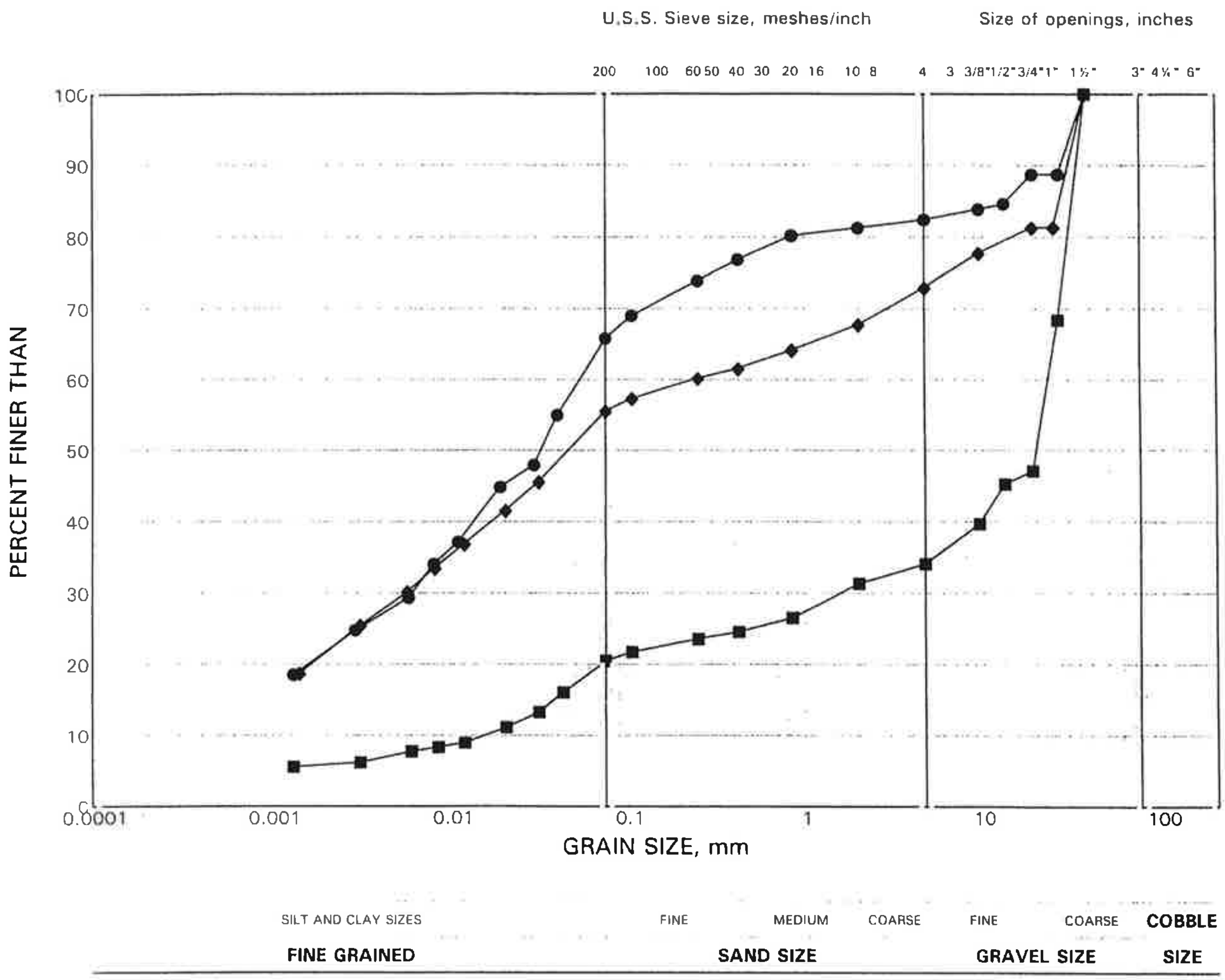
Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ON_MOT 0011141F.GPJ ON_MOT.GDT 19/12/02

GRAIN SIZE DISTRIBUTION TEST RESULTS
Embankment Fill

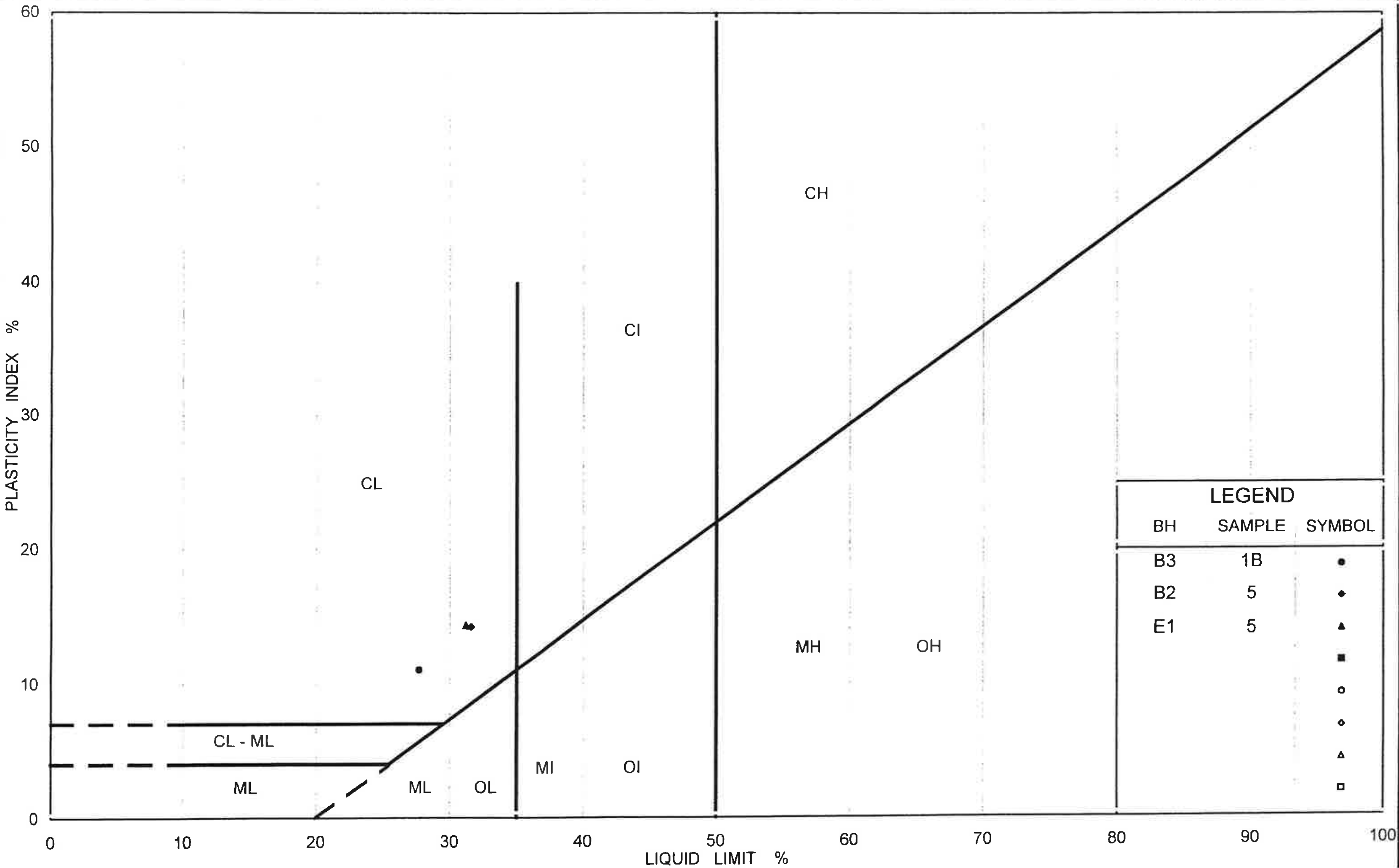
FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B2	5	185.4
■	B2	7	183.1
◆	E1	5	181.0

Oct 75, FF-S-21



Ministry of Transportation

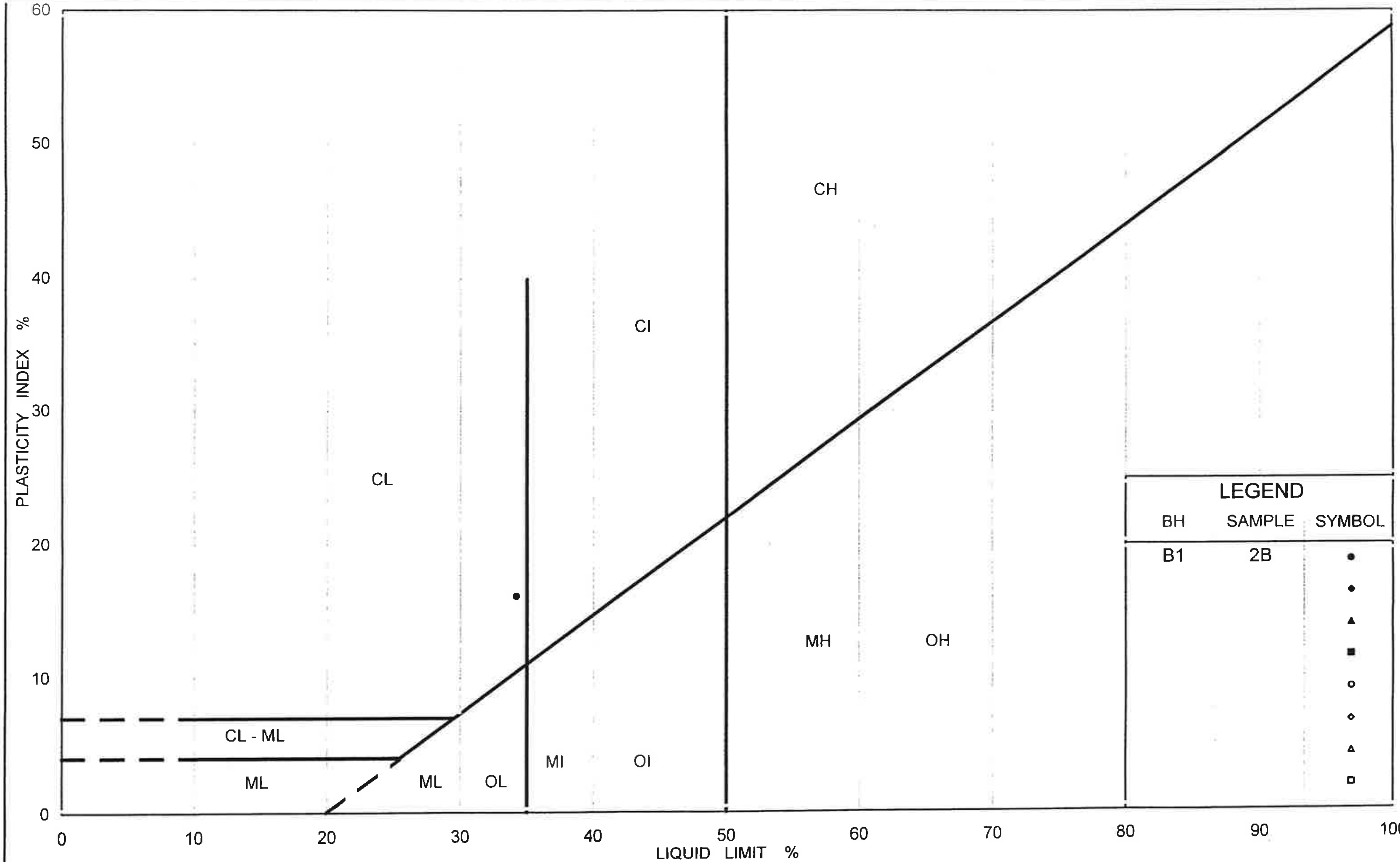
Ontario

PLASTICITY CHART
Embankment Fill

FIG No.2

Project No. 001-1141F

Oct 75, FF-S-21



Ministry of Transportation

Ontario

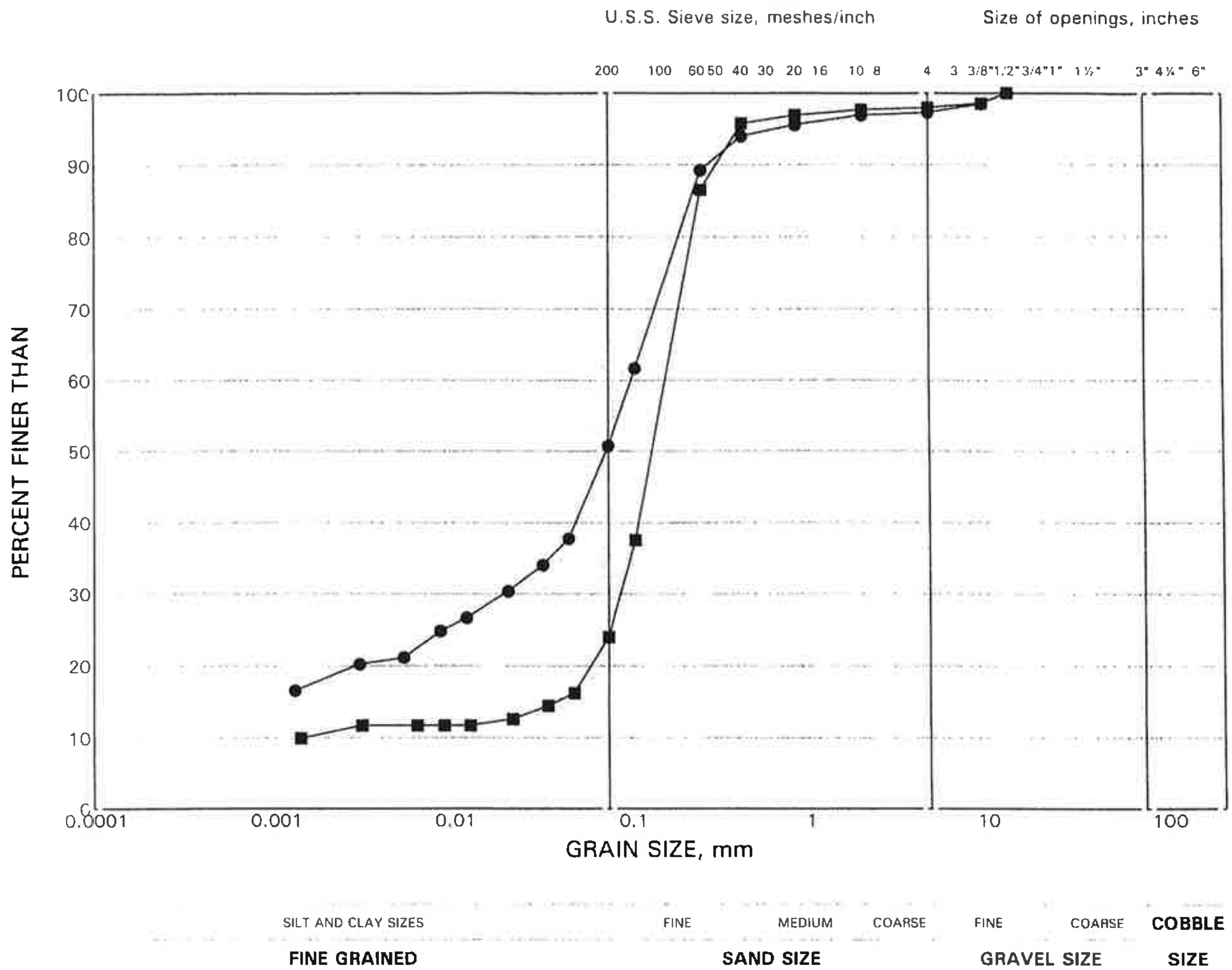
PLASTICITY CHART
Surficial Clayey Silt

FIG No.3

Project No. 001-1141F

GRAIN SIZE DISTRIBUTION TEST RESULTS
Surficial Sand and Silt to Silty Sand

FIGURE 4



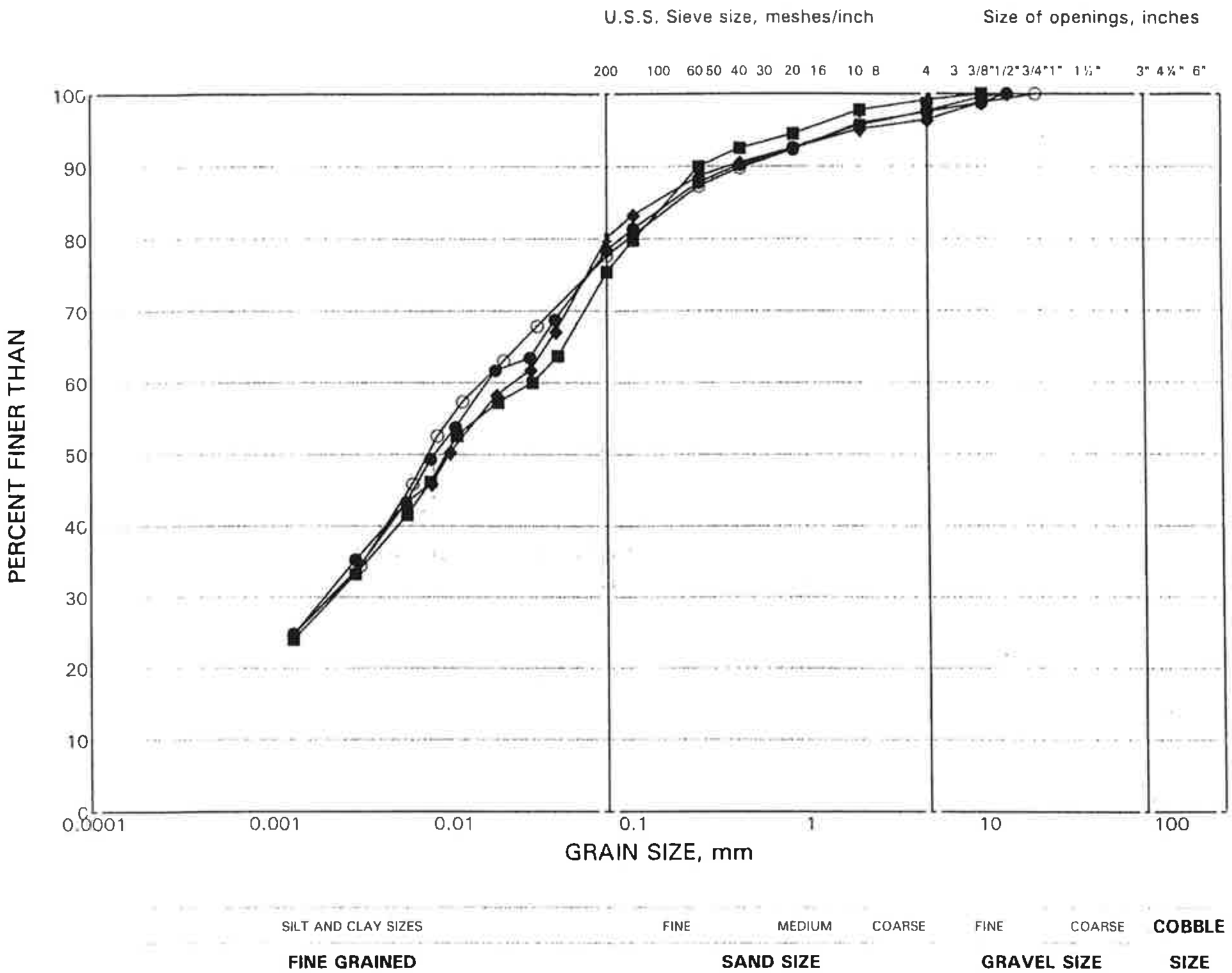
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B1	3A	178.2
■	B3	3	184.2

GRAIN SIZE DISTRIBUTION TEST RESULTS

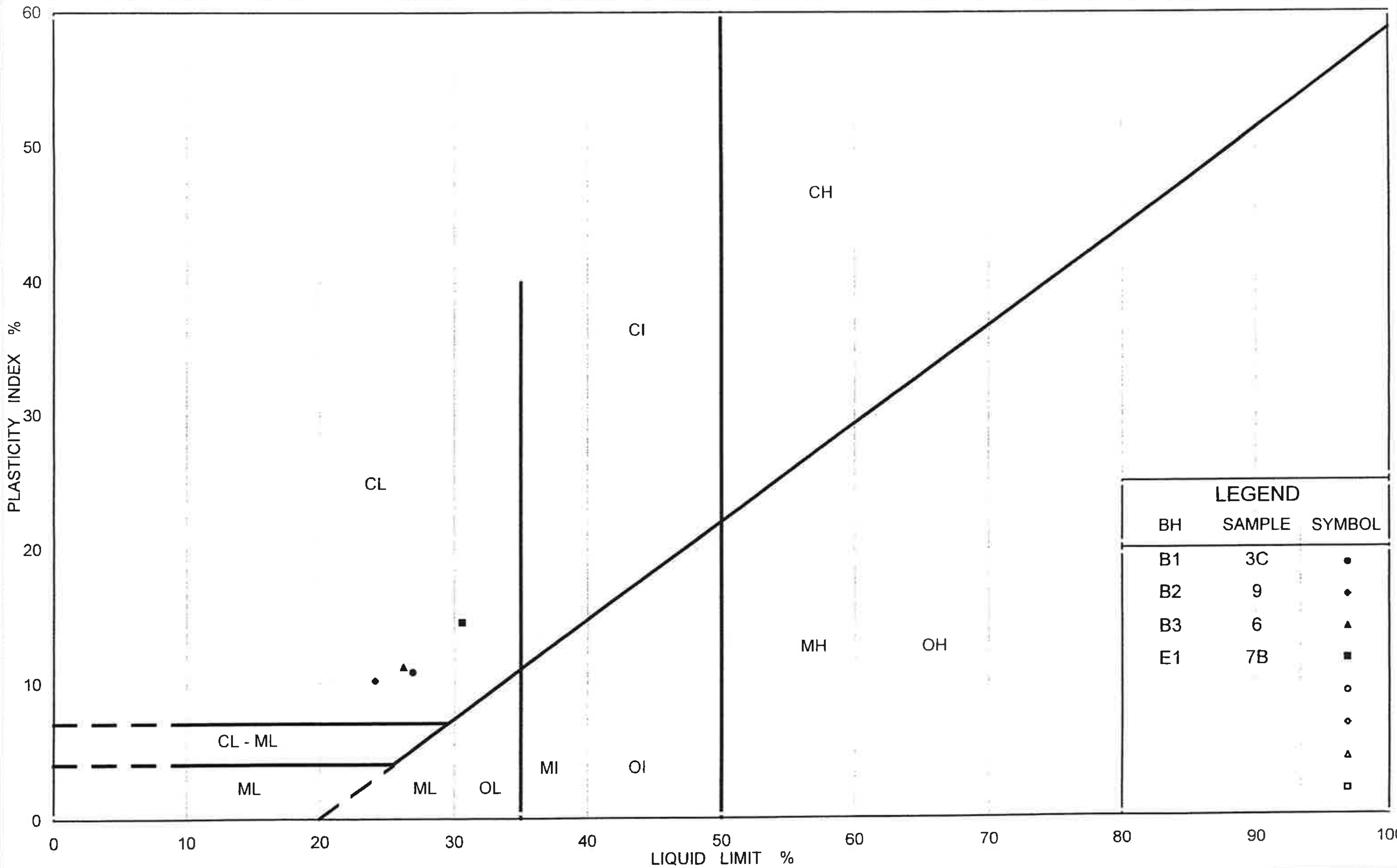
Clayey Silt Till

FIGURE 5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B1	3C	177.5
■	B2	9	179.8
◆	B3	6	182.0
○	E1	7B	177.8



Ministry of Transportation
Ontario

PLASTICITY CHART
Clayey Silt Till

FIG No.6

Project No. 001-1141F

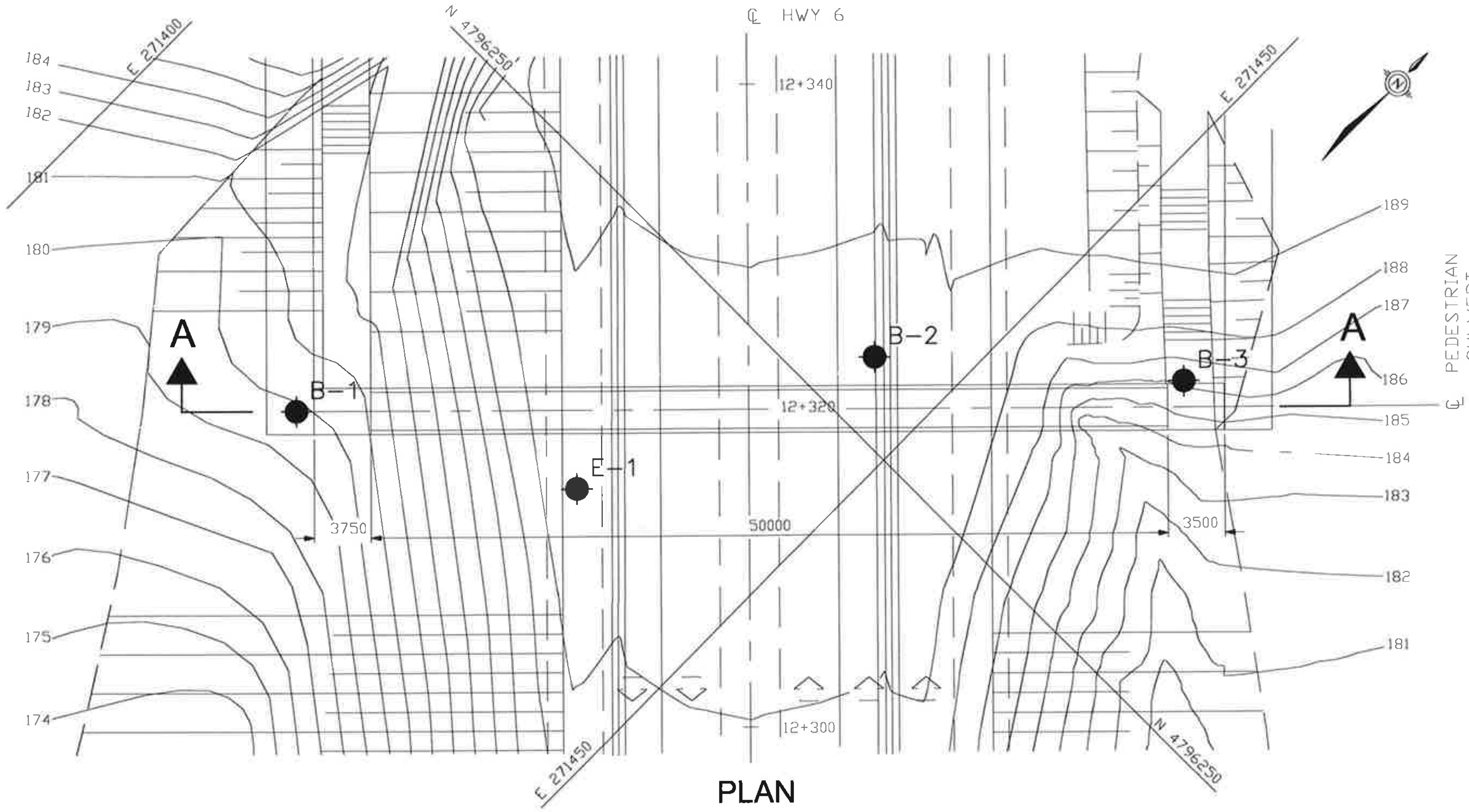
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 19-95-07

BRUCE TRAIL TUNNEL
STATION 12+320, HWY 6
BOREHOLE LOCATIONS & SOIL STRATA

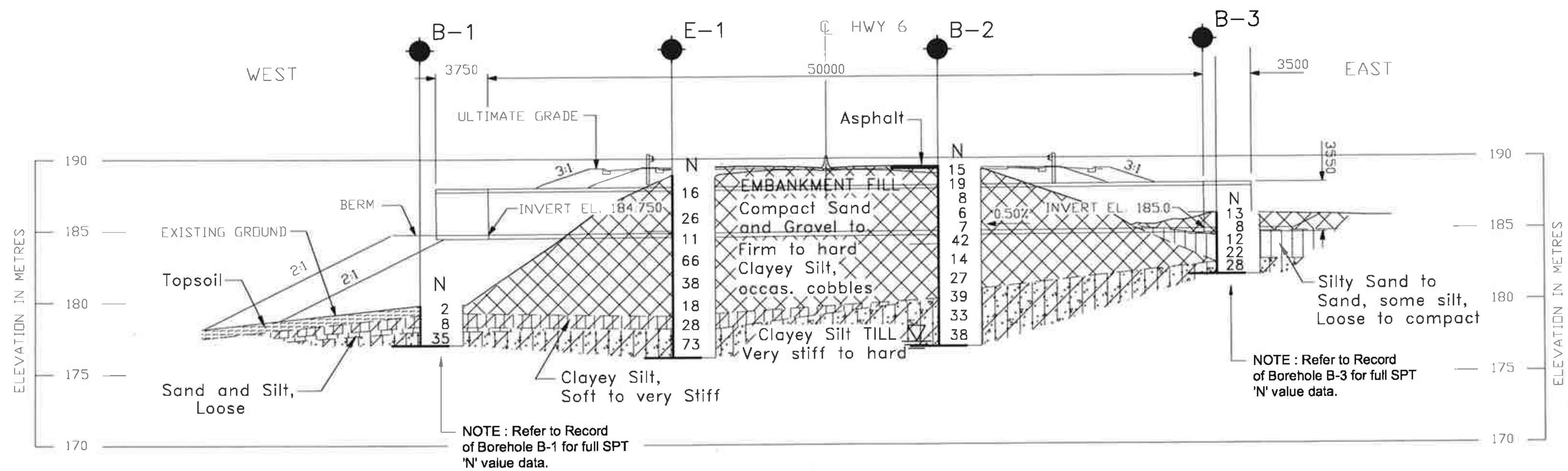
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

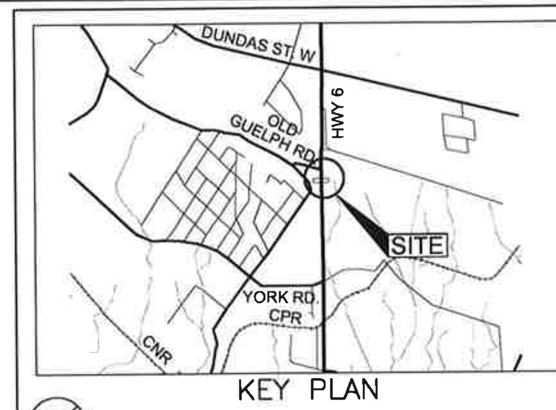
SCALE 5 0 5 10 METRES



SECTION A-A, ALONG CENTRELINE OF PROPOSED PEDESTRIAN TUNNEL

SCALE 5 0 5 10 METRES

SCALE 5 0 5 10 m



KEY PLAN

LEGEND

- Borehole – Current 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 in piezometer
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	EASTING
B-1	179.8	4796226.2	271421.8
B-2	189.4	4796254.2	271445.1
B-3	186.1	4796266.8	271459.9
E-1	188.9	4796235.1	271437.6

NOTES

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

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

The complete 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

General Arrangement file was provided in digital format by URS Canada Inc., File name "2002-04/underpass.dwg"

LICENSED PROFESSIONAL ENGINEER L.C. COYNE JULY 22, 2005 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER F.J. HEFFERNAN JULY 22, 2005 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No.

HWY.		PROJECT NO. 001-1141F	DIST.
SUBM'D. LCC	CHKD. LCC	DATE: APRIL 2005	SITE:
DRAWN: PS	CHKD. LCC	APPD. ASP	DWG. 1

THURBER

Appendix B

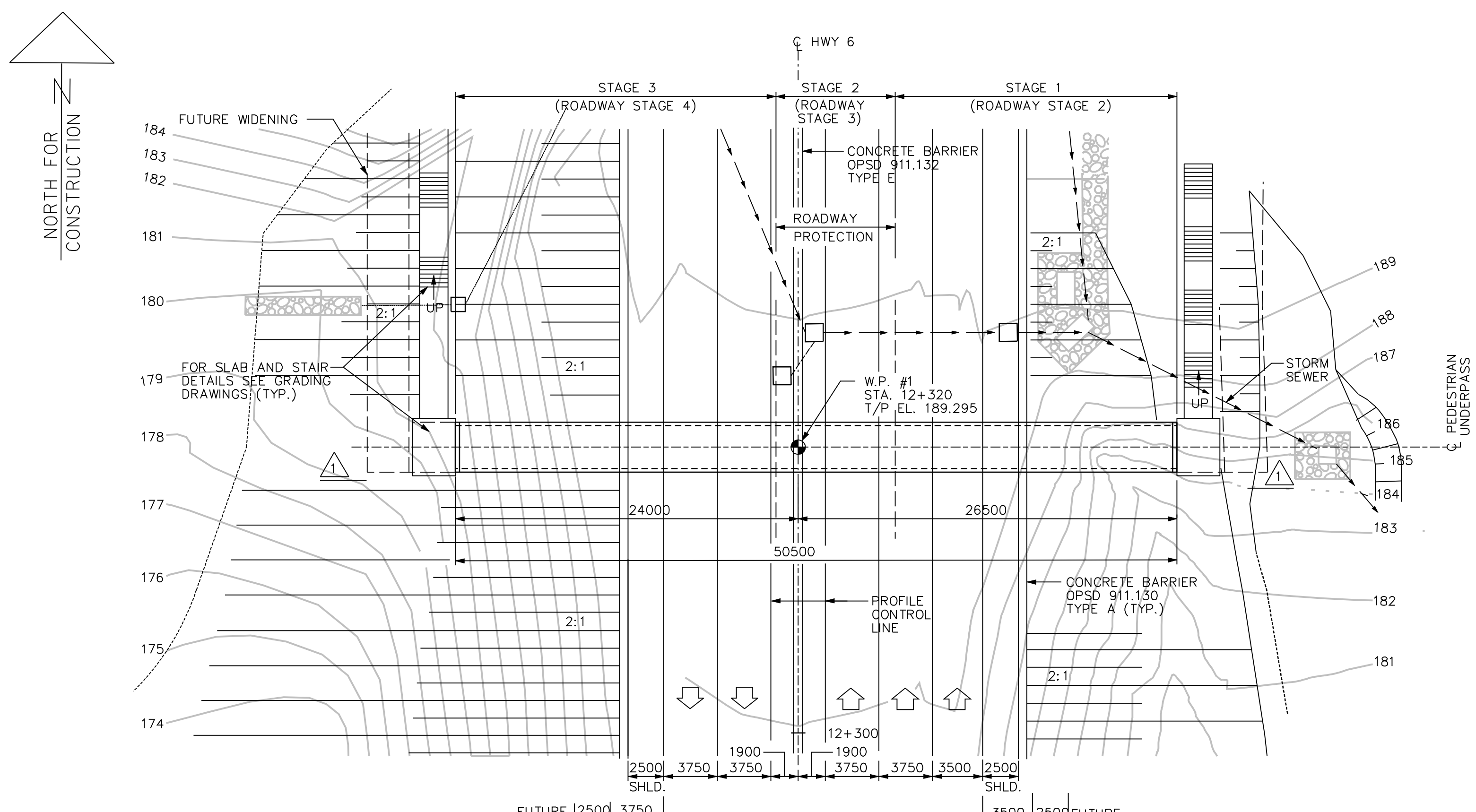
Archive Drawings of Existing Pedestrian Underpass (Construction)

HWY	6	
CONT No	2005–2019	
WP No	19–95–07	
BRUCE TRAIL PEDESTRIAN UNDERPASS GENERAL ARRANGEMENT		SHEET 275

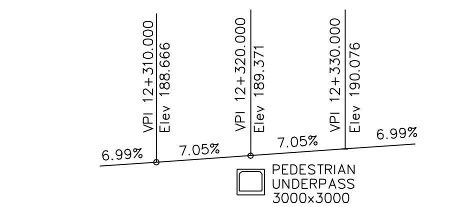
URS

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



PLAN
1:250



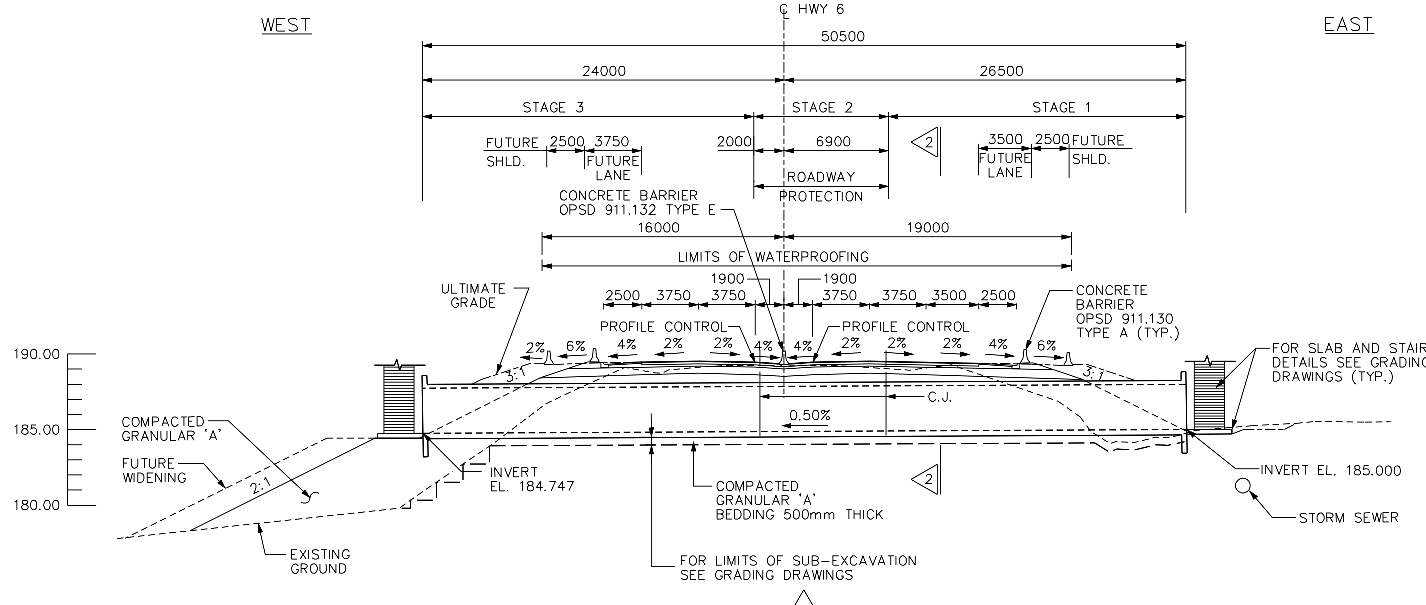
PROFILE CONTROL ALONG HWY 6
N.T.S.

GENERAL NOTES

1. CLASS OF CONCRETE TO BE 30 MPa.
2. CLEAR COVER TO REINFORCING STEEL:
   BOTTOM OF TOP SLAB 40±10 FOR SLABS < 300 THICK
   50±10 FOR SLABS > 300 THICK
   BOTTOM OF BOTTOM SLAB 100±25
   REMAINDER 70±20 UNLESS NOTED OTHERWISE.
3. REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED.

CONSTRUCTION NOTES

1. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH SIDES OF CULVERT KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.
2. NO CONCRETE SHALL BE PLACED UNTIL THE DEPTH OF THE EXCAVATION AND THE CHARACTER OF THE FOUNDATION HAVE BEEN APPROVED BY THE ENGINEER.
3. SUPPORTS FOR REINFORCING STEEL SHALL BE AS PER OPSD 3922.000 AND OPSD 3923.000 ON FORMED SURFACES. ON NON-FORMED SURFACES, CONCRETE BLOCKS (MIN. 20 MPa) SHALL BE USED.
4. FOR CONSTRUCTION STAGING AND ROADWAY PROTECTION SEE GRADING DRAWINGS.



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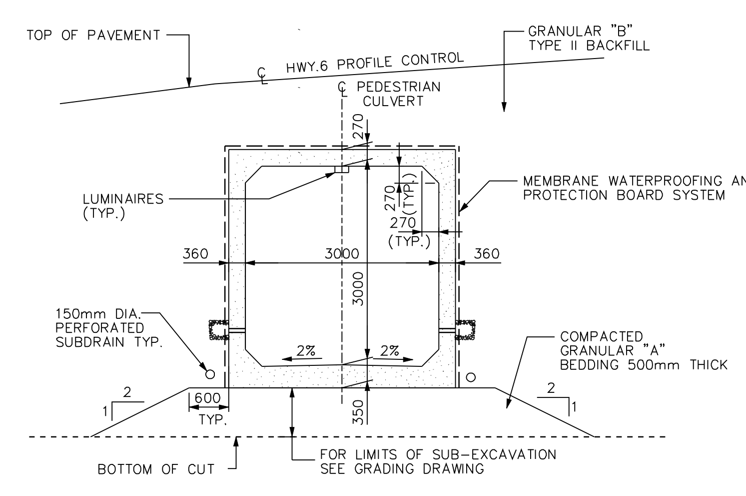


2
N.T.S

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS AND SOIL STRATA
3. REINFORCING DETAILS

LIST OF ABBREVIATIONS

TYP.	TYPICAL
C.J.	CONSTRUCTION JOINT
N.T.S.	NOT TO SCALE

APPLICABLE STANDARD DRAWINGS

OPSD 208.010	BENCHING OF EARTH SLOPES
OPSD 803.010	BACKFILL AND COVER FOR CONCRETE CULVERTS
OPSD 4601.000	LOCATION OF SITE NUMBER AND DATE FIGURES
OPSD 4670.000	TYPICAL JOINT DETAILS

LICENSED PROFESSIONAL ENGINEER N. WALKER PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER R.S. REEL 20/07/05 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L. KOZACHUK 20/7/25 PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN N.W.	CHK R.S.R.	CODE CSA-S6-00	LOAD CL 625-ONT	DATE	JULY 2005
DRAWN V.A.	CHK N.C.W.	SITE 36-515/C		DWG	1

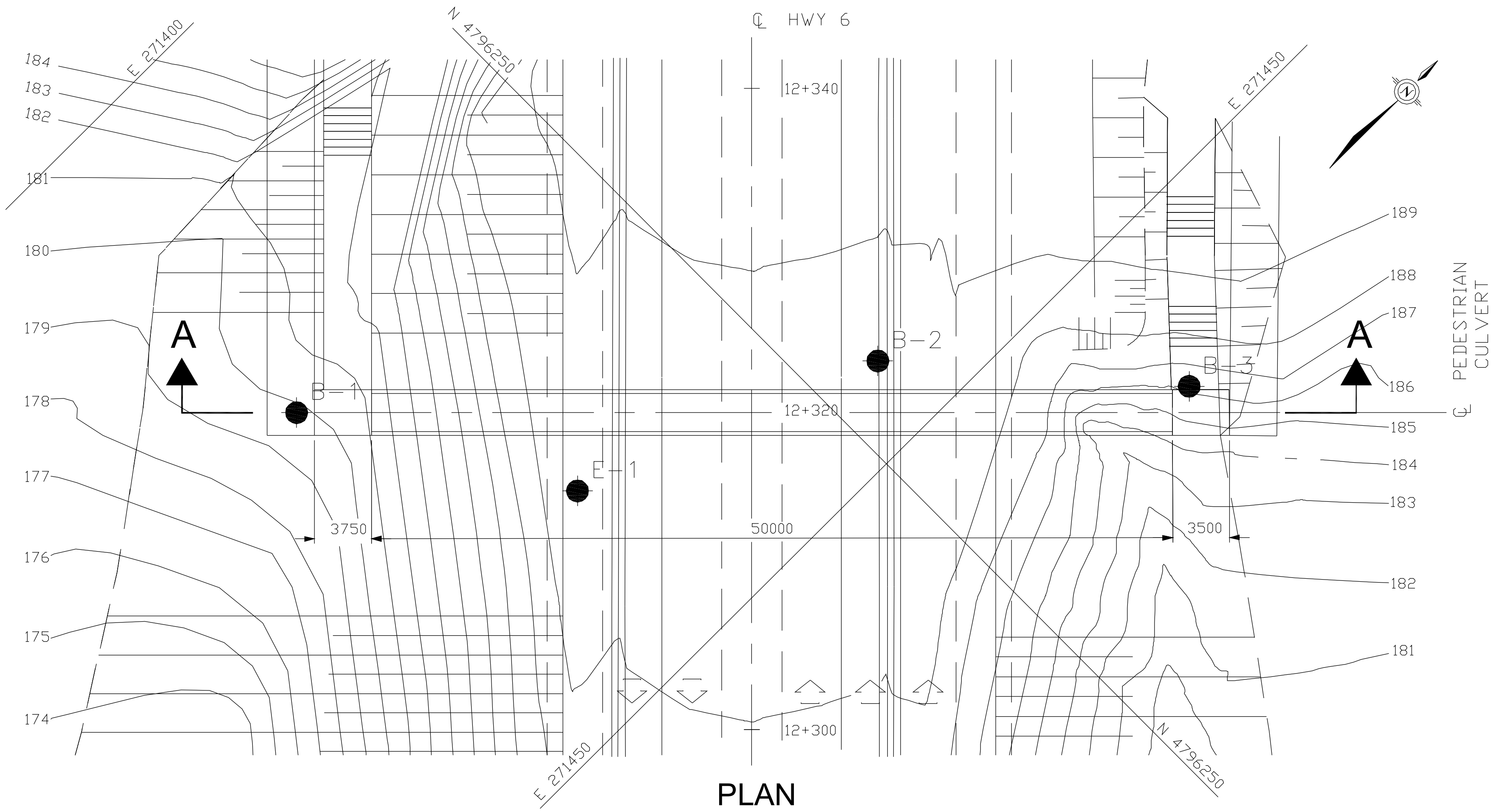
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2005-2019
WP No. 19-95-07

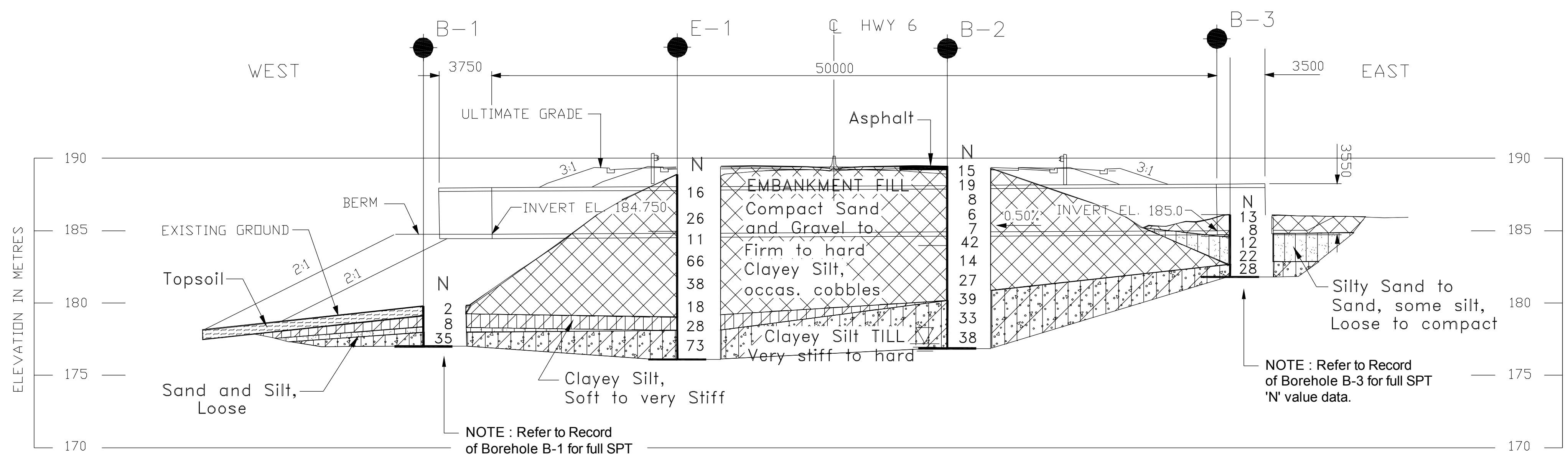
BRUCE TRAIL TUNNEL
STATION 12+320, HWY 6
BOREHOLE LOCATIONS & SOIL STRATA

SHEET 276

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



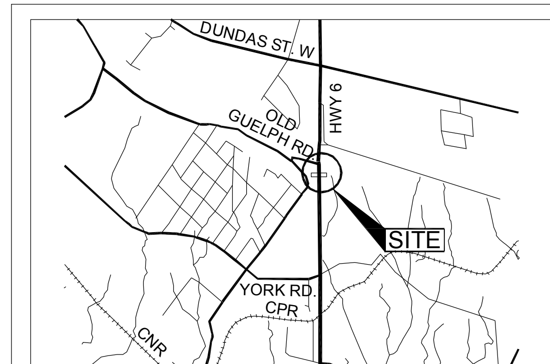
PLAN



SECTION A-A, ALONG CENTRELINE OF PROPOSED PEDESTRIAN TUNNEL

SCALE 5 0 5 10 METRES

SCALE 5 0 5 10 m



KEY PLAN

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		NORTHING	EASTING
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REGISTERED PROFESSIONAL ENGINEER F.J. HEFFERNAN JULY 22, 2005 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No.

HWY.	PROJECT NO. 001-1141F		DIST.
SUBM'D. LCC	CHKD. LCC	DATE: JULY 2005	SITE:
DRAWN: PS	CHKD. LCC	APPD. ASP	DWG. 2

PLOT DATE: July 26, 2005
FILENAME: T:\Projects\2000\001-1141F\RAPRIL2005\276.dwg

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No 2005–2019
WP No 19–95–07

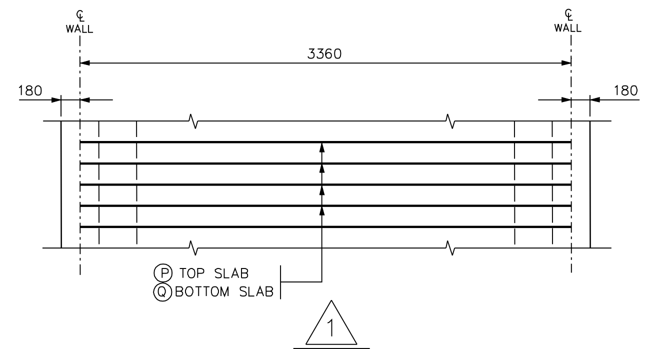
BRUCE TRAIL PEDESTRIAN UNDERPASS

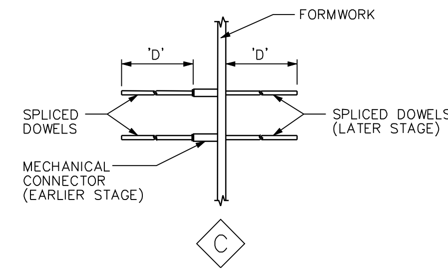
CULVERT DETAILS

SHEET 277

URS

QUANTITIES			
ITEM	WALLS & SLABS	RETAINING WALL	TOTAL
VOLUME OF CONCRETE cubic metres	235	–	235

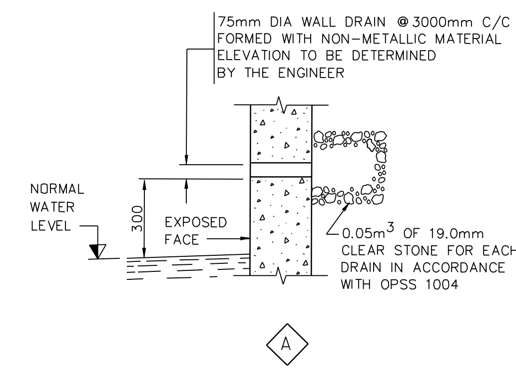


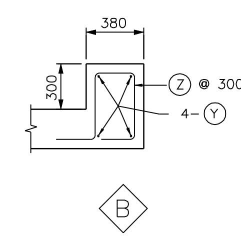


TYPICAL MECHANICAL SPLICE DETAIL

NOTE:
1. SIZE OF MECHANICAL SPLICE TO MATCH REBAR SIZES.
2. DOWELS TO BE HOOKED FOR DEVELOPMENT WHERE REQUIRED.

DIMENSION 'D'	
15M SPLICED DOWELS	650 (TOP) 650 (BOTTOM)





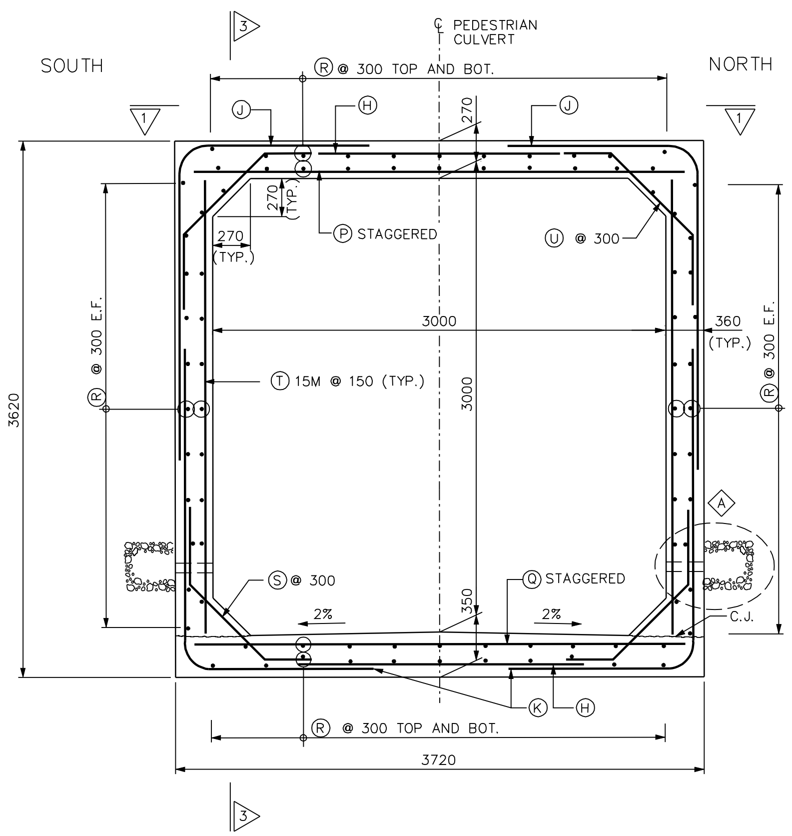
HEADER WALL

MARK	BAR SIZE	C/C	DETAILS	REMARKS
(H)	15M	300	STRAIGHT	TOP OF TOP SLAB & BOTTOM OF BOTTOM SLAB
(J)	20M	150	1820, 1960, R=180	J BARS ALTERNATE WITH K BARS
(K)	20M	150	1820, 1960, R=180	K BARS ALTERNATE WITH J BARS
(P) ••	20M	150	STRAIGHT	BOTTOM OF TOP SLAB STAGGERED
(Q) ••	20M	150	STRAIGHT	TOP OF BOTTOM SLAB STAGGERED
(R)	15M	300	STRAIGHT	LONGITUDINAL MIN. LAP SPLICE = 500
(S)	15M	300	720, 45°, 45°, 180, 180	BOTTOM HAUNCHES
(T)	15M	150	STRAIGHT	INSIDE FACE OF WALLS
(U)	15M	300	720, 45°, 45°, 180, 180	TOP HAUNCHES
(L)	15M	300	500, 1080	DOWELS TO APRON WALLS
(M)	15M	450	STRAIGHT	APRON WALL
(Y)	15M	–	STRAIGHT	HEADER WALL
(Z)	15M	300	460, 225, 190, 230	HEADER WALL

NOTES: – All dimensions shown to centre line of bar
– • represents vertical dimension
– •• c/c spacing given at midspan

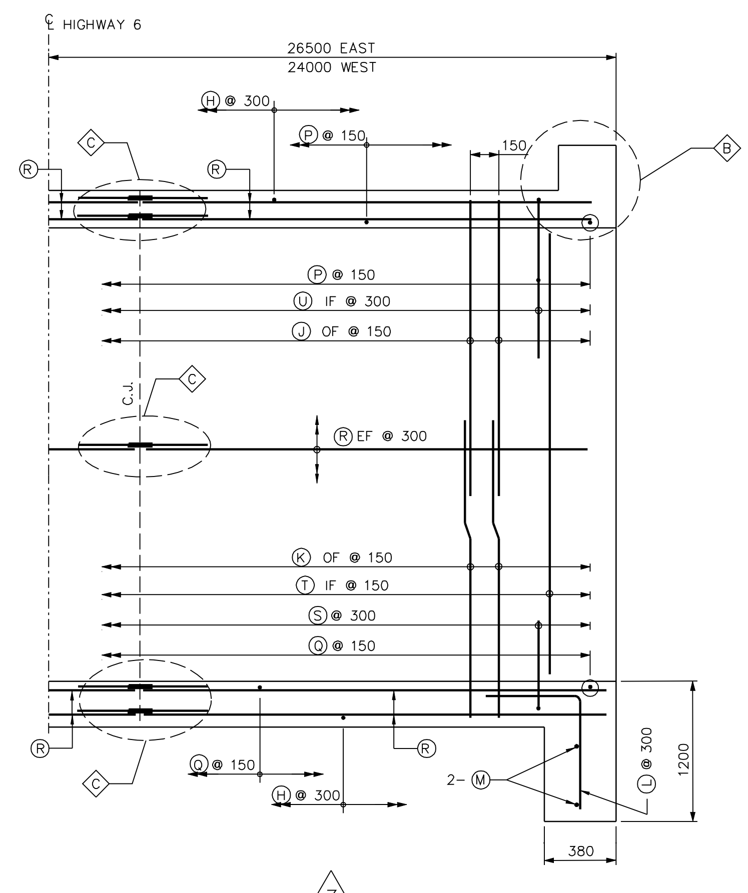
LIST OF ABBREVIATIONS

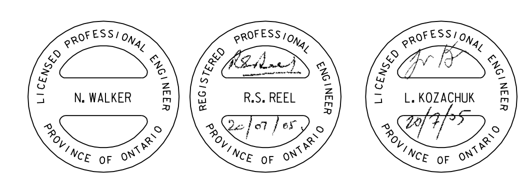
TYP.	TYPICAL
ALT.	ALTERNATE
E.F.	EACH FACE
C.J.	CONSTRUCTION JOINT
N.T.S.	NOT TO SCALE
U/N	UNLESS NOTED
BOT.	BOTTOM



TYPICAL CULVERT SECTION

HEADER WALL DETAILS SHOWN IN (B)





DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

(MODIFIED)

STANDARD DRAWING JUNE 2002	SS114–2
RIGID FRAME BOX CULVERT	

REVISIONS

DATE	BY	DESCRIPTION

DESIGN N.W.	CHK R.S.R.	CODE CSA–S6–00	LOAD CL 625–ONT	DATE JULY 2005
DRAWN V.A.	CHK N.C.W.	SITE 36–515/C		DWG 3

Appendix C

Selected Site Photographs

THURBER

Photo 1- Bruce Trail Pedestrian Underpass, west side
March 27, 2022

THURBER

Photo 2- Bruce Trail Pedestrian Underpass, west side.
March 27, 2022

Photo 3- Bruce Trail Pedestrian Underpass, northwest side
March 27, 2022

Photo 4- Bruce Trail Pedestrian Underpass, southwest side
March 27, 2022

THURBER

Photo 5- Bruce Trail Pedestrian Underpass, east side
March 27, 2022

Photo 6- Bruce Trail Pedestrian Underpass, northeast side
March 27, 2022

Photo 7- Bruce Trail Pedestrian Underpass, southeast side
March 27, 2022

THURBER

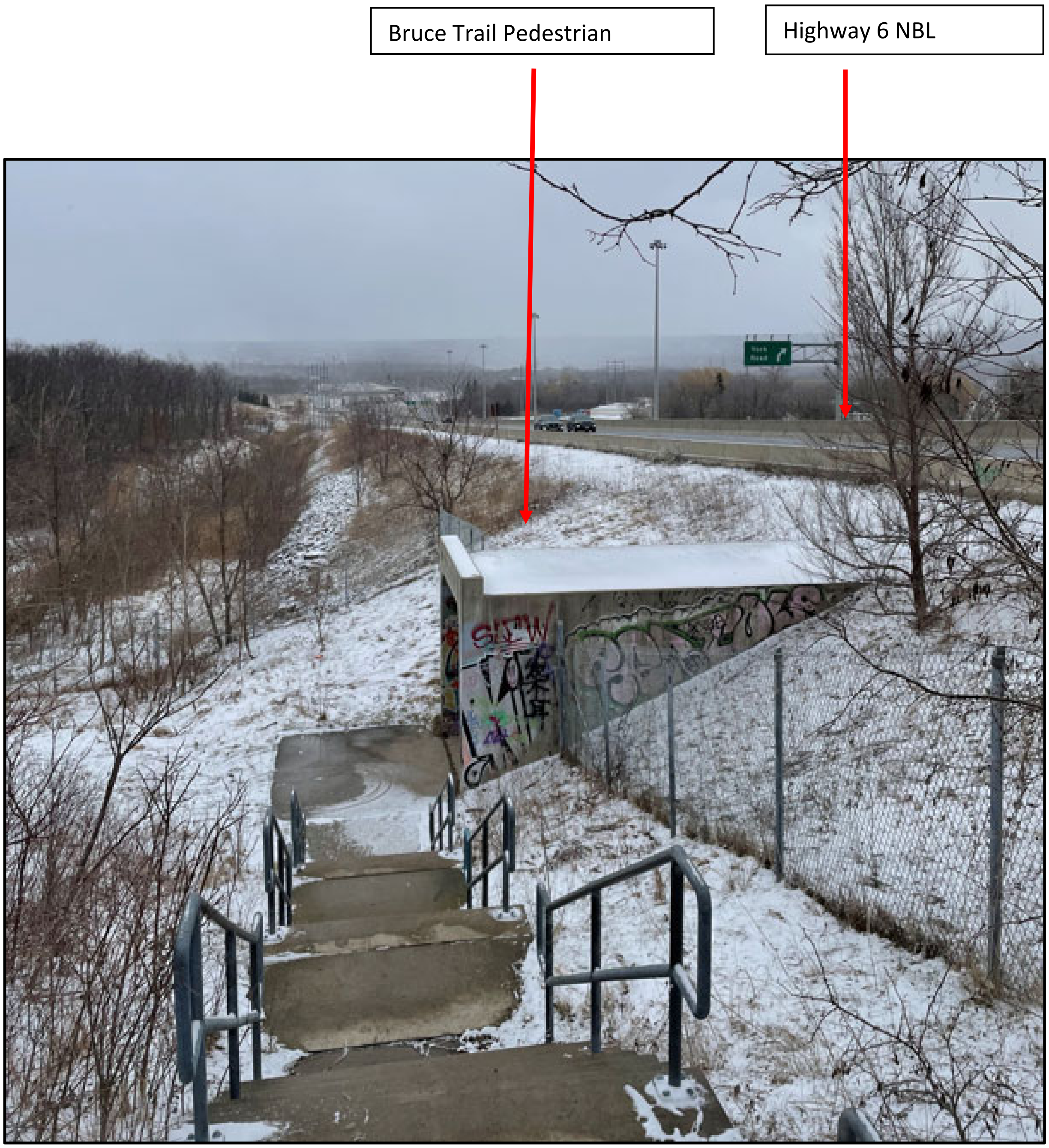


Photo 8- Bruce Trail Pedestrian Underpass, northeast side
March 27, 2022