THURBER ENGINEERING LTD.

MEMORANDUM

To: Christopher Schueler, P.Eng.
AECOM

Date: April 15, 2016

From: Murray Anderson, P.Eng.
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(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-4406-20

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN NORRIS WHITNEY BRIDGE REHABILITATION (SITE 28-28)

1 INTRODUCTION

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing Norris Whitney Bridge on Highway 62 in the Geographic Township of Ameliasburgh – Municipality of Belleville and Prince Edward County, Ontario. It also presents preliminary geotechnical recommendations for use in assessment of the existing foundations at the site. It is noted that the proposed structural alternatives for the rehabilitation are not yet defined.

The recommendations provided in this memorandum are for planning, structure evaluation and preliminary design purposes only. Additional investigation and analysis may be required in any subsequent detail design phase of the project.

The following reference numbers apply to this site:

- Current W.P. 4082-13-01
- Site No. 28-28
- GEOCRES No. 31C-135
- Historic W.P. 134-74-01

2 SITE DESCRIPTION

The Norris Whitney Bridge carries Highway 62 over the Bay of Quinte between Belleville at the north end and the hamlet of Rossmore at the south end. The RFP information indicates that the existing bridge was constructed in 1982 and consists of an 11 span steel plate I-girder bridge with a total length of 881 m (58 m + 9 x 85 m + 58 m). The bridge has an overall deck width of 12 m and accommodates two lanes of traffic and a pedestrian sidewalk.

The General Layout drawing for the existing bridge (Contract No. 80-34) indicates that the top of pavement elevation on the bridge rises from Elev. 86.9 at the north abutment to Elev. 100.3 at the highest point, and then falls to Elev. 79.2 at the south abutment. The highest point is located between the fourth and fifth piers from the north end, and road grade at this location is approximately 25.6 m above the water level indicated on the drawing (Elev. 74.89).

The natural terrain in the vicinity of the bridge is generally flat. In the GEOCRES report, the depth of water in the bay was reported to be typically 3.0 to 4.5 m, locally about 10.7 m within the former main navigation channel adjacent to the south shore. A discontinuous causeway from a former crossing alignment runs parallel to the west side of the existing bridge.

3 SUBSURFACE CONDITIONS

The site is located across the boundary of two physiographic regions known as the Napanee Plain on the north shore and the Prince Edward Peninsula on the south shore. The Napanee Plain is a flat to undulating limestone plain largely stripped of overburden. The Prince Edward Peninsula is a low plateau of limestone projecting into the eastern part of Lake Ontario. Geologic maps indicate that the bedrock consists of interbedded limestone and shale of the Verulam Formation.

A site investigation was completed by the Ministry of Transportation and Communications between May 24 and June 17, 1977 for the proposed construction of the current high level bridge. A total of 21 boreholes and eight dynamic cone penetration tests (DCPTs) were advanced to depths ranging from 0.5 to 20.7 m during the investigation. The boreholes were terminated upon refusal on probable bedrock or after recovery of at least one run of bedrock core. The available GEOCRES files are attached in Appendix A.

The soil conditions encountered in the boreholes were variable as a result of dredge and fill operations previously carried out in the area. Up to 4.6 m of mixed fill was encountered in boreholes drilled on the north shore, and up to 13.9 m of organic clay and/or sandy gravel fill (related to the former causeway crossing) was encountered on the bay bottom, typically increasing in thickness to the south. These materials were not present adjacent to the south shore where dredging was carried out in the former main navigation channel.

The fill and organic soils on the north half of the site are underlain by a 3.4 to 4.0 m thick layer of native sand to sandy gravel, overlying a 2.7 to 4.0 m thick layer of clay to clayey silt. A 0.6 to 5.5 m thick layer of glacial till comprising sand and gravel with various proportions of silt and clay was encountered below all of the above soils along essentially the full alignment.

Standard Penetration Test (SPT) N-values recorded in the various units and the corresponding consistency/relative density of the layers are summarized below:

Soil Unit	SPT N-values (blows/0.3 m)	Relative Density	Consistency
Fill	2 to 120, typically 2 to 32	Very loose to dense	-
Organic Clay	0 to 19, typically 1 to 10	-	Very soft to stiff
Sand to Sandy Gravel	5 to 53	Loose to very dense	-
Clay to Clayey Silt	2 to 24	-	Soft to very stiff
Glacial Till	15 to 80 (for 75mm), locally 3 to 9	Compact to very dense	Very stiff to hard

The undrained shear strength of the organic clay measured by field and laboratory vane testing ranged from 1.4 to 46 kPa. Similar testing of the clay and clayey silt yielded undrained shear strengths of 37 to 135 kPa.

Bedrock or refusal on probable bedrock was encountered below the organic clay, sandy gravel and glacial till in all but four of the boreholes at depths of 0.5 to 19.0 m below the ground or water surface. In general, the bedrock surface falls from approximate Elev. 76.7 (0.5 m depth) some 50 m south of the south abutment, to Elev. 55.9 (19.0 m depth) approximately 230 m north of the south abutment, and then rises to Elev. 68.4 (8.1 m depth) approximately 165 m north of the north abutment. The inferred bedrock elevations at the south and north abutments are approximate Elev. 69.5 and 65.5, respectively.

Bedrock was proven by recovering up to 1.5 m of rock core in 11 of the boreholes. The bedrock was described as sound limestone with shaly sections. RQD values ranged from 50% to 97%, indicating a fair to excellent quality rock.

Groundwater levels at about 0.3 and 0.9 m depth (Elev. 75.0) were observed in two boreholes drilled on land. The water level in the bay was at Elev. 74.9 at the time of the fieldwork. The water was typically 3.0 to 4.5 m deep at the borehole locations in the bay, locally about 10.7 m within the former main navigation channel adjacent to the south shore.

4 SITE OBSERVATIONS

Foundations engineering staff from Thurber visited the site to observe conditions related to the geotechnical performance.

No obvious signs of settlement or distress of the existing bridge foundations were observed.

The approach embankments appeared to be stable, with no obvious signs of instability or bulging. Rock protection is provided at the abutments along the shoreline. Possible erosion was noted at a drain outlet on the east side of the north approach embankment.

Photographs of the structure and the approaches are attached in Appendix B.

5 EXISTING FOUNDATIONS

The archive design drawings indicate that the bridge is supported on steel pile foundations bearing on bedrock. The layout of the piles is as follows:

- The south abutment is supported on nine HP 310x110 piles driven to bedrock. The front row of piles contains five piles battered forward at 1H:3.5V, and the hind row contains two inner piles inclined forward at 1H:3.5V and two outer piles inclined backwards at 1H:8V.

- Each pier is supported by a pile cap carried on 10 to 14 composite piles comprising three HP 310x110 piles enclosed in steel tube piles (1219 mm outside diameter by 12.7 mm thick) filled with tremie concrete after installation of the H-piles. The H-piles are socketed at least 500 mm into sound bedrock in a 1050 to 1200 mm diameter socket filled with tremie concrete. The tube piles are driven to bedrock. The perimeter composite piles are inclined outwards at a batter of 1H:4.5V, and the two interior piles, where present, are vertical.

- The north abutment is supported on ten HP 310x110 piles driven to bedrock. The front row of piles contains six piles battered forward at 1H:3V, and the hind row contains two inner piles inclined forward at 1H:3V and two outer piles inclined backwards at 1H:8V. Additionally, a vertical H-pile is driven to bedrock under each wing wall.

An underwater inspection of the pile caps was carried out in 2007 by Harmer Podolak Engineering Consultants Inc. The inspection involved visual examination of the submerged portion of the concrete pile caps at the piers, between the water line and the bay bottom. At four of the piers, the top 0.6 to 5.1 m of the tube piles below the pile caps was accessible for examination. In general, the pile caps were assessed to be in good condition with light to medium scaling and localized spalls. The exposed tube piles were considered to be in good condition with light surface rust. One perforation was noted on the north side of a pile at the sixth pier from the south.

6 ASSESSMENT OF EXISTING FOUNDATIONS

The archive information and site observations indicate that the existing bridge is founded on piles to bedrock. The foundations appear to be performing satisfactorily, and it can be assumed that the foundations will continue to perform satisfactorily in the future provided they are structurally sound.

The RFP document suggests that the required rehabilitation work will consist of concrete deck repairs, waterproofing, paving, expansion joint replacement, substructure repairs, and barrier wall repair/replacement. In this case, no appreciable increase in the loading is anticipated.

If a significant (greater than 10%) increase in loading on the foundations due to rehabilitation of the bridge is subsequently planned, it will be necessary to carry out further assessment and possibly site investigation and field testing to support the preparation of foundation design recommendations.

7 EXCAVATION AND ROADWAY PROTECTION

If the selected rehabilitation strategy requires excavation in the approach fills behind the abutments, it is recommended that site investigation and field testing be carried out in each approach fill in order to characterize the fill and bedrock, and to select parameters for the design of roadway protection. One borehole within each approach fill and within the probable extent of excavation is considered to be appropriate. The boreholes should extend for the full depth of fill or to twice the depth of excavation, whichever is the greater, or terminate on bedrock if encountered within this depth.

8 CLOSURE

The factual subsurface information used in the preparation of this memorandum was taken from the report by The Ministry of Transportation and Communications titled “Feasibility Foundation Investigation Report for Bay of Quinte Crossing at Belleville, Hwy. 14, District 8, Kingston”, WP 134-74-01, Site 28-28, undated.

The memorandum was prepared by Mr. Murray Anderson, P.Eng., Senior Foundations Engineer and was reviewed by Mr. Alastair Gorman, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

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Review Principal, Designated MTO Contact

Attachments

Appendix A

GEOCRES Report, Correspondence, and Archive Drawings

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31C-135

DIST. 8 REGION

W.P. No. 134-74-01

CONT. No. 80-34

W. O. No.

STR. SITE No. 28-28

HWY. No. 14

LOCATION Bay of Quinte, Belleville

No OF PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

ENGINEERING MATERIALS OFFICE
SOIL MECHANICS SECTION

WP 134-74-01 DIST 8

HWY 14 STR SITE 28-28

Bay of Quinte Crossing
at Belleville

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FEASIBILITY
FOUNDATION INVESTIGATION REPORT

For

Bay of Quinte Crossing at Belleville
Hwy. 14, District 8, Kingston
W.P. 134-74-01, Site 28-28

INTRODUCTION

The Ministry of Transportation and Communications has proposed to replace the existing Hwy. 14 crossing of the Bay of Quinte. The crossing is presently accomplished by means of a causeway and two steel through truss type structures, one structure being a swing bridge for the purpose of accommodating marine traffic on the waterway. A route location study resulted in the decision to build a high-level structure, east of, and parallel to the existing crossing.

Because of the large scope of the project, the Soil Mechanics Section was requested to carry out a feasibility study to determine the subsurface conditions along the line of the proposed crossing. The request was contained in a memorandum dated March 22, 1977 from Mr. T.C. Kingsland, Kingston Regional Structural Planning Engineer. An investigation was subsequently carried out to establish the subsoil, bedrock and groundwater conditions existing at the site.

The pertinent factual data were provided immediately after completion of the fieldwork. In addition, recommendations were also provided verbally. This report contains the factual data obtained from the field and laboratory work, together with recommendations pertaining to the design and construction of the structure and associated approach fills.

FIELD INVESTIGATION

The fieldwork was carried out during the period of May 24, 1977 to June 17, 1977. A total of 21 boreholes, 8 accompanied by dynamic cone penetration tests, were put down to depths of 67 feet below the bay water surface. The borings on the bay were put down by means of diamond drilling techniques using NX casing operating from a drum-floating raft. The remaining boreholes were advanced by means of a muskeg vehicle equipped with hollow stem continuous flight augers. Bedrock was proven by obtaining up to 5 feet of BXL size rock core.

SITE DESCRIPTION: PAST AND PRESENT CONDITIONS

The proposed crossing on the Bay of Quinte will connect Zwick's Park in the Town of Belleville with the Hamlet of Rossmore in Prince Edward County (Refer to Photo 1 in the Appendix).

The north approach will extend through generally flat terrain immediately east of existing Hwy. 14 within Zwick's Park (Refer to Photo 2 in the Appendix). Maps of the area prior to 1890 show the area now occupied by Zwick's Park to be largely open water except for an island. This piece of land, about 4 acres in area, was named Zwick's Island and now comprises the southwestern portion of Zwick's Park. The island was joined to the mainland by means of a causeway as part of the original 1890 Bay of Quinte Bridge. It is believed that this causeway resulted in the development of the swamp areas as shown in maps as early as the 1920's. In the 1960's the swamp area was used for sanitary landfill. The western portion has been sodded and extensively developed into a park for day users. The municipality of Belleville is presently placing clean landfill east of Hwy. 14 with the intention of developing it for recreational uses.

On the Prince Edward County side the approaches will extend partially through residential areas within the Hamlet of Rossmore (Refer to Photo 3 in the Appendix).

At the location of the proposed crossing, the Bay of Quinte is some 3000 feet wide. The depth of the main channel, at the swing bridge location, is up to 35 feet deep. Elsewhere along the proposed crossing, the bay is 10 to 15 feet deep.

The existing crossing from Zwick's Park to Rossmore, some 2750 feet, is accomplished by means of an earth causeway and two steel through truss type structures. The main structure adjacent to the Rossmore side is composed of one swing span and three fixed spans (128' fixed, 200' swing, 165' fixed and 128' fixed); and about 1000 feet north of the swing span, is a two fixed span structure (99', 108').

The original structure completed in 1891, was composed of about seventeen spans between Zwick's Island and Rossmore. The present causeway was constructed in the 1920's by placing dredged material between the piers. The original superstructure was removed and the causeway was brought up to final grade leaving the original piers in place. These piers are constructed of stone, timber and concrete cribwork and are believed to be supported on timber piles. The causeway was completed by placing 10-20 ton armour stone on the side slopes for protection against wave action (Refer to Photo 5 in the Appendix).

GEOLOGY

The site borders the physiographic regions of the "Napanee Plains" and the "Prince Edward Peninsula". These regions are characterized by a thin veneer of glacial drift underlain by generally flat to undulating limestone of the Trenton-Black River Formation. At this site, limestone and shale bedrock outcrops appear on the Prince Edward County shore in the vicinity of existing Hwy. 14 (Refer to Photo 4 in the Appendix).

SUBSOIL DESCRIPTION

General

Subsoil across the site is quite variable as a result of dredging and landfill operations carried out in this area in the past. The parent subsoil consists of 11 to 13 feet of sandy gravel or medium to coarse sand underlain by 9 to 13 feet of clay or clayey silt which in turn overlies a 5 to 18 foot thick deposit of glacial till. The glacial till is underlain by limestone bedrock. On the Prince Edward County shore subsoil consists of 2 to 11 feet of sandy gravel overlying bedrock. In Zwick's Park the parent subsoil is overlain by up to 15 feet of fill material, whereas, within the bay the parent subsoil is overlain by a thin veneer up to 9 feet thick of very soft organic clay.

East of the existing causeway from the Prince Edward County side to about 1000 feet north of the shoreline, the parent subsoil beneath the bay has been dredged, in some locations down to the bedrock surface. In a few locations, up to 45 feet of organic clay has been recently deposited within the dredged areas. However, within the main channel the current has kept the dredged areas relatively free of organic clay deposits.

The locations and elevations of the borings, together with a stratigraphical profile and sections inferred from borehole data, are shown on Drawing No. 1347401-A.

A brief description of the various subsoil and bedrock types encountered and the groundwater conditions are presented in the paragraphs to follow.

Fill Material

Fill material was encountered in all borings put down in the Zwick's Park area and in the existing causeway. Fill material was also encountered in two boreholes (B.H.'s 17 & 20) put down in the bay adjacent to the causeway.

Fill material in Zwick's Park: The fill material encountered in Zwick's Park is estimated to be 3 to 15 feet thick. This fill material varies in composition from a sandy gravel to a sand with silt and inclusions and/or pockets of clayey silt. Typical grain size distribution curves for the fill material are shown in envelope form on Figure 1 of the Appendix. The results of Atterberg Limit testing on representative samples from the cohesive zones of clayey silt are plotted on the Plasticity Chart,Figure 2. The Atterberg Limits indicate that the clayey silt pockets are inorganic and of low plasticity. This fill material also contains inclusions of wood chips and organics. In these areas the organic content was found to be as high as 7% by weight.

Standard Penetration testing carried out in the fill material in Zwick's Park gave a range of 'N' values of 2 to 32 blows per foot indicating this fill has undergone slight to moderate compaction.

Fill material in the causeway: The fill material within the causeway and adjacent to it was found to be up to 41 feet deep. The composition of this fill material is a gravelly sand with a trace of silt. The result of grain size distribution testing is shown in an envelope form on Figure 1. This fill material contains a trace of shells and wood chips in isolated zones. The organic content in these zones was found to be as high as 13% by weight; however, this high organic content is attributed to the presence of wood chips and is not indicative of the deposit as a whole.

The range of Standard Penetration Test 'N' values for this fill material is 8 to 92 blows per foot, indicating that the material has been subject to a non-uniform compactive effort.

Organic Clay

This material comprises the bay bottom, being generally a thin veneer up to 9 feet thick covering the parent subsoil. In some areas where the bay bottom has been dredged for causeway fill, the bay has filled up to 46 feet of organic clay within these dredged areas. In one boring in Zwick's Park area a deposit of organic clay about 7 feet thick was encountered immediately below the fill. The material in this deposit is black, being generally plastic and composed of organic clay. Where the very deep deposits of organic clay were encountered in the bay bottom, the organic material contains appreciable amounts of silt and sand. The organic content of the deposit as determined by laboratory testing ranges from 3 to 26% by weight.

The results of laboratory and field testing are summarized below:

Moisture Content, Bulk Density and Atterberg Limits

		Range	Average
Natural Moisture Content	(W%)	34-280	167
Liquid Limit	(W_L%)	55- 96	74
Plastic Limit	(W_p%)	29- 85	65
Plasticity Index	(I_p%)	11- 26	17

Undrained Shear Strength Su

		Range	Sensitivity
Laboratory Vane Tests	(psf)	50-490	3
Field Vane Tests	(psf)	30-960	3

The Atterberg Limits indicate that the material is organic and of high plasticity. The natural moisture content generally decreases with depth while the undrained shear strength generally increases with depth. The undrained shear strength indicates that the deposit has a very soft to firm consistency.

Sandy Gravel to Sand

This granular deposit was encountered immediately below the fill in Zwick's Park below the organic clay deposit of the bay bottom south of the park and also immediately below the ground surface on the Prince Edward County side. On the Prince Edward County side this deposit ranges in thickness from 2 to 11 feet. Elsewhere, the thickness of this deposit varies from 11 to 13 feet. This granular stratum is composed of sandy gravel or medium to coarse sand. The results of grain size distribution testing performed on representative samples from this stratum are summarized in envelope form on Figure 3.

Standard Penetration testing gave 'N' values ranging from 4 to 53 blows per foot, generally increasing with depth. Based on these values the deposits are estimated to have a compact to very dense relative density.

Clay

This stratum was encountered in three borings (B.H. #4, 7 & 8) put down in Zwick's Park and also in three borings put down in the bay east of the causeway and north of the fixed span structure (B.H. #10, 11 & 13). This cohesive

deposit was found beneath the stratum of sandy gravel to sand and also in some locations beneath the deposit of organic clay. The thickness of the deposit is estimated to be between 9 and 13 feet. The deposit is composed of clay which is somewhat fissured and laminated. In two locations (B.H. #4 & 7) part of this deposit was found to have random layers of clayey silt.

The results of laboratory and field testing on representative samples taken from this stratum are summarized below.

Natural Moisture Content, Atterberg Limits and Bulk Density

		Clay		Clayey Silt Layers	
		Range	Average	Range	Average
Natural Moisture Content	(W%)	42-72	56	30-44	36
Liquid Limit	(W_L%)	58-80	69	22-35	31
Plastic Limit	(W_p%)	19-24	23	14-24	18
Plasticity Index	(I_p%)	39-54	46	8-17	12
Bulk Density	(γPCF)	97-109	104		

Undrained Shear Strengths (Su. P.S.F.)

	Range	Sensitivity
Field Vane Tests	800->2400	3-8
Laboratory Vane Tests	765-2830	2-4
Laboratory Unconfined Tests	825-1940	
Laboratory Quick Triaxial Tests	1740-1880	

Consolidation Tests (3 tests)

Initial Void Ratio	e_o	1.2-2.0
Coefficient of Consolidation	c_c	0.4-1.4
Degree of Preconsolidation	$P'_c - P'_o$ (PSF)	3200-6600

The results of the Atterberg Limit testing are plotted on the Plasticity Chart, Figure 4. The Atterberg Limit testing indicates that the clay deposit is generally inorganic and of high plasticity, whereas the layers of clayey silt are inorganic and of low plasticity. The Natural Moisture Content is generally between the Plastic Limit and the Liquid Limit. The consolidation testing gave a range of preconsolidation pressure of 3200 to 6600 P.S.F. in excess of the existing effective overburden pressure.

The undrained shear strength as measured by laboratory and in situ testing ranges from greater than 2400 P.S.F. to 760 P.S.F. decreasing with depth. The sensitivity as measured by vane testing (both laboratory and field testing) indicates that in general the deposit is slightly to moderately sensitive to remoulding. Furthermore, the undrained shear strengths indicate that the consistency of the deposit varies from very stiff in the upper portion changing to firm, generally decreasing with depth.

Clayey Silt

This deposit was encountered in two locations; one in Zwick's Park (B.H. #5) immediately below the sandy gravel to sand deposit and one in the bay (B.H.#12) immediately below the sandy gravel deposit. The thickness of this deposit is estimated to be 9 feet thick. The material in this stratum is clayey silt and a trace of sand with random silt and sand seams. The results of the laboratory and field testing are summarized as follows:

Moisture Content and Atterberg Limits

		Range	Average
Natural Moisture Content	(W%)	22-44	33
Liquid Limit	(W_L%)	33-36	34
Plastic Limit	(W_p%)	15-23	18
Plasticity Index	(I_p%)	10-21	16

Undrained Shear Strength (Su P.S.F.)

	Range	Sensitivity
Field Vane Tests	1000-1600	2-5

The results of the Atterberg Limit testing are shown on Figure 5; the testings indicate that the clayey silt deposit is inorganic and of low plasticity. In general, the testing shows that the natural moisture content is slightly above or below the liquid limit.

Standard Penetration testing gave 'N' values ranging from 13 to 24 blows per foot. Based on these 'N' values,and together with the in situ vane testing, the deposit is estimated to have a stiff to very stiff consistency.

Glacial Till

A deposit of glacial till up to 18 feet thick was encountered in all borings except in the area of the south bank and also in areas where the parent subsoil has been completely dredged. The composition of the glacial till varies widely across the site. Beneath Zwick's Park and adjacent to it, the till is cohesive being a heterogeneous mixture of clayey silt, with sand and gravel. Elsewhere, the till deposit is granular and composed of a heterogeneous mixture of sand, gravel with some silt and clay. In some locations the glacial drift was found to contain occasional cobbles and boulders in the lower portion of the deposit. The results of laboratory testing on representative samples from this deposit are shown on the Plasiticity Chart, Figure 6 and on the Grain Size Distribution Envelope, Fig.7. The Atterberg Limits indicate that the cohesive glacial drift has an inorganic matrix of low plasticity.

The range of 'N' values from the Standard Penetration testings in this deposit is 15 blows per foot to 80 blows for 3 inches. The cohesive glacial till is estimated to have a firm to hard consistency based on 'N' values. Similarly, the relative density of the granular till is estimated to have a compact to very dense relative density that in general increases with depth.

Groundwater Conditions

Observations on the groundwater level were carried out during the fieldwork by measuring in the open boreholes. The measurements place the groundwater table at a depth of 3 feet below the existing ground surface which corresponds to elevation 246. During the time of the field investigation the water level in the bay fluctuated only slightly from elevation 245.6 to elevation 245.8.

DISCUSSION AND RECOMMENDATIONS

The Ministry of Transportation and Communications has proposed to replace the existing crossing of Hwy. 14 and the Bay of Quinte with a new two-lane high level structure about 100 feet east of and parallel to the existing crossing.

A feasibility study was initiated to assess the foundation requirements for the high level bridge and related approaches by carrying out a preliminary subsurface investigation. The high level structure will be required to have a minimum vertical navigational clearance of 90 feet and a minimum depth of channel of 13 feet. The main navigation channel will be shifted toward the centre of the bay. The alignment of the north approach will meet the existing conditions some 1000 feet north of Zwick's Park shoreline, whereas about 500 feet south of the Rossmore shoreline, the alignment of the south approach will match existing. The existing structures are to be removed, however, the extent of the causeway removal is yet to be reconciled. The grades of the proposed structure are restricted to a maximum of 5%.

The number of spans and span details are as yet to be decided. Furthermore, the locations of the piers and abutments will depend to a large degree upon the extent of the approach fills which are in turn affected by the feasibility and economics of constructing and maintaining the fill slopes. Because of the importance of the approach fills at the feasibility stage in the planning process, this aspect will be discussed first.

Approach Embankments

North approach - Based on the preliminary profile grade as established by the Regional Planning and Design Office, the heights of profile grade above the following existing conditions at the north approach are anticipated.

Height of Profile Grade Above Average Ground Surface	up to 55 feet
Height of Profile Grade Above Bay Bottom	60 - 105 feet

Because of the generally flat terrain, fill heights will not vary significantly in the transverse direction. Longitudinally, the fill height will depend upon the profile grade at that location and upon the sloping nature of the bay bottom. However, the slope of the bay bottom at the north approach is relatively insignificant, being about 2%.

Subsoil at the north approach generally consists of up to 15 feet of fill material overlying 10 to 13 feet of firm to stiff clay or hard clayey silt which in turn overlies 2 to 4 feet of hard glacial till, followed by limestone bedrock. In one location (B.H. #5), a 7 ft. thick pocket of soft organic clay is sandwiched between the lower clayey silt stratum and the 7 ft. overlying fill material. In another location (B.H. #8) 11 feet of dense sand to sandy gravel is encountered between the lower clay stratum and a 4 foot thick deposit of fill material. Subsoil beneath the bay bottom is somewhat less competent, being a surficial veneer of very soft organic clay up to 9 feet thick overlying 10 to 13 feet of compact to dense sandy gravel which in turn overlies 10 feet of firm to stiff clay. The clay in turn is overlying a compact to very dense glacial till.

The fill material is heterogeneous in composition. In some zones it is composed of competent granular fill material but in other areas it is composed of sanitary landfill or topsoil. This fill material will be detrimental to the stability and performance of the approaches and it is, therefore, recommended that the fill material and organic clay be removed entirely within the plan limits of the proposed embankment. Backfill placed underwater should be composed of Granular 'A' to prevent segregation of material. Fill material for the remainder of the approaches should be of acceptable granular material placed and compacted according to current MTC standards.

Stability analysis in terms of total stress have been carried out to determine the stability of fills immediately after construction. In this method of analysis, stability is governed by undrained shear strength properties of the foundation and fill materials. The following data and values were used in carrying out the stability analysis.

Fill Material	γ (pcf)	ϕ^o	Su (psf)
(Tension Cracks 5')			
Granular Material	130	30	0

The subsoil condition beneath the bay and beneath the land are somewhat different, subsoil beneath the land being slightly more competent. For this reason two sets of subsoil data were considered in the analysis. The subsoil conditions also assume that the unacceptable fill material and organic clay will be removed entirely within the plan limits of the embankment and replaced by a granular type of acceptable fill material.

Subsoil Conditions Beneath the Bay (Water Elevation 246)

Elevation (Feet)	γ(PCF)	γ'(PCF)	ϕ^o	Su (PSF)
240-225	130	68	30°	0
225-220	100	38	0	1000
220-215	100	38	0	750
Below 215	140	78	35	0

Subsoil Conditions Beneath Land

Elevation (Feet)	γ(PCF)	γ'(PCF)	ϕ^o	Su (PSF)
250-235	130	68	30	0
235-225	100	38	0	1500
Below 225	140	78	35	0

The longitudinal stability of the embankments will depend upon the geometry of the forward slope of the embankment, as well as the position and location of the structure's abutment. Furthermore, the longitudinal stability will depend on the location of the toe of the slope since subsoil is not as competent beneath the bay bottom as beneath the land.

The following are recommendations based on the above analysis. They are discussed according to three categories, depending upon the location of the toe of the slope with regard to the shoreline.

Case A: The toe of the slope will not extend within 20 feet of the shoreline.

- Fills up to 40 feet will be stable with forward and side slopes of 2:1.
- Fills up to 50 feet with 20 foot long counterbalancing berms at mid-height on both the forward and side slopes of the embankment would also be stable with slopes of 2:1.
- Fills up to 60 feet with 40 foot long counterbalancing berms at mid-height on both the forward and side slopes of the embankment would be stable with 2:1 slopes.

These recommendations are summarized in Figure 9, together with the critical slip circle and assumed subsoil conditions for 40 foot and 50 foot fill heights.

Case B: The toe of the slope is located within 20 feet of the shoreline but will not extend into the bay.

- Fills up to 30 feet above the average ground surface (assumed to be at elevation 250±) will be stable with forward and side slopes of 2:1.
- Fills up to 45 feet above the average ground surface will require 35 foot long berms at mid-height in both forward and transverse direction of the approaches.

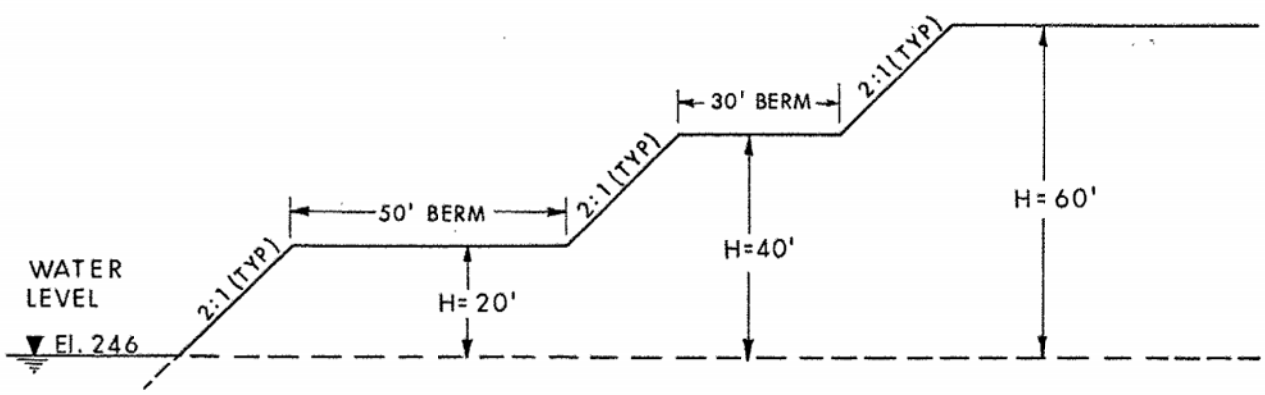
- Fills up to 55 feet above the average ground surface will be stable with 60 foot long counterbalancing berms at mid-height on both the forward and side slopes of the embankment with slopes not steeper than 2:1.

  These recommendations are shown on Figure 10, together with the critical slip circles and the assumed subsoil stratigraphy.

Case C: The toe of the slope extends past the shoreline into the bay.

- Fill heights up to 20 feet above the water line will be stable with side slopes of 2:1.
- Fills up to 40 feet above the waterline with 50 foot long counterbalancing berms at midheight on both the forward and side slopes of the embankment would be stable with 2:1 slopes.
- Fill heights of up to 60 feet above the water line would require the following berm configuration for stability against deep seated rotational failure (also see sketch below):

  berm at 1/3 height 50 feet long
  berm at 2/3 height 30 feet long
  all side slopes 2:1 maximum



Case C

A minimum of 50 foot transitition taper should be provided between the different geometrical configuration, i.e. between a Case C and Case A or between a Case A and Case B condition.

Due to the presence of the underlying compressible clay stratum, fill will undergo settlements as a result of the consolidation of the clay deposit. To estimate settlements the stress distribution was computed by the Purdue Method and consolidation characteristics of the clay deposit were based on three laboratory consolidations tests. An estimate of the field e-log p curve was made from the laboratory curve by means of a graphical procedure after Schmertmann, 1953. The calculations indicate that a 40 foot fill with side slopes of 2:1 and no berms will undergo a settlement of approximately 5-6 inches; furthermore, it is estimated that 90% of the settlement will occur within 4 months after construction. Calculations were also carried out for a 60 foot fill with 40 feet mid-height berms and slopes of 2:1. The expected settlement for this fill is 7-9 inches, 90% of which would occur within 4 months after completion of the fill. The above magnitudes of settlement are applicable to fill heights located on the land. The consolidation testing indicated that the clay stratum beneath the bay would undergo about 2-3 times the settlement of clay stratum beneath land subject to the same loading conditions.

If settlements of such magnitudes are detrimental to the performance of the approaches and the pavement, the fills should be constructed and left in place for 4-6 months prior to paving. It is calculated that this preloading period would allow about 90% of the settlement to occur.

South approach: Based on the preliminary profile grade the height of fills above the following existing conditions at the south approach are anticipated.

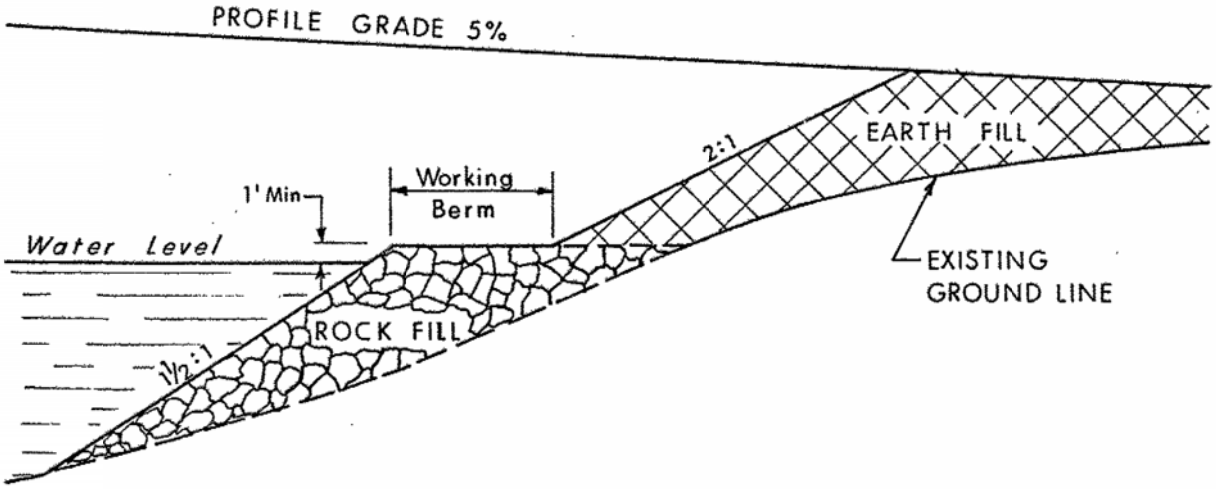
Height of Profile Grade Above Average Ground Surface up to 15 feet
Height of Profile Grade Above Bay Bottom at Pier Stop up to 62 feet

Again, generally flat terrain is prevalent at the south approach and the fill heights will not vary appreciably in the transverse directions. However, due to the sloping nature of the bay bottom and the 5% profile grade, fill height will vary considerably in the longitudinal direction, increasing as the approaches extend outward from shore.

Subsoil at the south approach is comprised of 2 to 11 feet of loose to compact sand to sandy gravel overlying limestone bedrock.

The subsoil is such that the anticipated fill heights will be stable with respect to deep seated rotational failure with forward and side slopes of 2 horizontal to 1 vertical. If the forward slopes extend into the bay the embankment should be constructed by placing rockfill to extend to a height of one

foot above the water level with side and forward slopes of 1½ to 1. The fill may then be completed by the placing of earthfill with a slope of 2 to 1. It may be advantageous for construction to provide a 10 foot berm between the crest of the rockfill slope and the toe of the earthfill slope. See sketch below.



Furthermore, the earthfill should be protected against wave action by rip-rapping to an elevation as per hydrological requirements.

Structure Foundations

As mentioned earlier, details of the spans and pier locations are as yet not finalized. However, one proposal put forward is that the structure be comprised of 12 spans of equal length (250'), the centre span to cross the relocated main channel where the present 2 fixed-span structure is located. This centre span is required to have a minimum vertical navigational clearance of 90 feet above the water level. Highway grades are to be limited to a maximum of 5%.

At this stage in the design it is felt that detailed recommendations concerning the construction and design of the structure foundations is not warranted. Only concepts or alternatives will be presented at this stage and further elaboration or clarification will be provided by this office as required.

Because of the widely differing subsoil conditions, foundation requirements for the southern and northern portion will be discussed separately.

Southern portion: On the southern portion subsoil consists of up to 46 feet thick deposits of very soft organic clay underlain by up to 5 feet of very dense glacial till overlying bedrock. Within the main channel bedrock is exposed at the bay bottom at a depth of about 35 feet.

Within the southern portion of the proposed line the subsoil conditions are such that virtually no lateral support will be provided to the structure foundations. The structure foundations must provide the sufficient and adequate internal lateral rigidity to be considered in this particular area. The following alternatives are put forward for consideration.

The structure may be supported on large diameter concrete caissons socketted into the bedrock surface. Construction would require that caissons be provided with a permanent liner. The bedrock conditions are such that foundations may be designed for a maximum allowable load of 30 t.s.f. The sizing of the caissons would be based on the slenderness ratio. To reduce the dewatering problems it may be advantageous to extend the caissons to the underside of the deck and in this manner construct the pile caps to serve also as the pier caps.

Alternatively, the foundation may be accomplished by constructing cofferdams and supporting the structure foundation directly on the bedrock surface. In this manner the structure foundation would have to be brought up by means of mass or reinforced concrete. For this scheme proposed, an extensive dewatering scheme would be essential for construction purposes.

Alternatively, the structure may be supported on steel tubular piles keyed into the bedrock surface to provide sufficient lateral resistance. Tubular piles should be sized according to the slenderness ratio.

Northern portion: Subsoil conditions on the northern portion are somewhat more competent. Subsoil consists of a thin veneer of very soft organic clay up to 9 feet thick overlying 11 to 13 feet of sand or sandy gravel followed by 9 to 13 feet of clay which in turn overlies 5 to 18 feet of glacial till. The glacial till overlies limestone bedrock. The following alternatives are provided for consideration.

The structure foundation may be founded on steel 'H' piles, steel tube piles, or concrete caissons founded on the bedrock surface. Piles constructed in this fashion may be designed for maximum allowable load, i.e. 100 tons/pile for a 12 BP 74 steel 'H' pile. Again, concrete caissons may be used but for construction purposes it will be necessary to use a permanent liner.

General considerations: The bedrock depths are quite variable across the stratigraphical profile of the proposed line, being from 33 to 63 feet below the water level. At this stage in the feasibility planning where the type of structure foundation has yet to be decided, there does not appear to be any advantage in the saving of pile lengths, etc., in shifting the alignment slightly to the east or west. However, depending on the type of foundation chosen for the southern portion, it may be advantageous to shift the proposed line to avoid the causeway fill, thus simplifying the construction of the cofferdams. Conversely, if piles are chosen it may be advantageous to shift the alignment toward the causeway fill to take advantage of the lateral support offered to the piles by the causeway fill.

MISCELLANEOUS

The fieldwork was supervised by Mr. M. MacLean, Project Engineer, and Mr. J. White, Student Engineer, using equipment owned and operated by Atcost Soil Drilling Inc., Concord, Ontario.

This report was written by Mr. M. MacLean with the assistance of Miss Y. Jamani, Student Engineer, and was reviewed by Mr. M. Devata, Supervising Engineer.

M. MacLean, P. Eng.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
M. D. MacLEAN
PROVINCE OF ONTARIO

M. Devata, P. Eng.
Supervising Engineer

MD/MM/gs
November, 1977

APPENDIX

Ministry of Transportation and Communications Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

W P 134-74-01 LOCATION Sta 435+83 o/s 104' Rt ℄ Exist. Hwy. 14 ORIGINATED BY JW

DIST 8 HWY 14 BOREHOLE TYPE Solid Stem Augers COMPILED BY SC

DATUM Geodetic DATE May 25, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE 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 Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL (WATER CONTENT (%))	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
253.1	Ground Surface										
251.6	Sandy Gravel					*					
1.5	Refusal to Augering Probable Bedrock End of Borehole * Note: Water Level Not Established						250				

RECORD OF BOREHOLE No 2

W P 134-74-01 LOCATION Sta 436+56 o/s 105' Rt ℄ Exist. Hwy. 14 ORIGINATED BY JW

DIST 8 HWY 14 BOREHOLE TYPE Hollow Stem Augers & BXL Rock Coring COMPILED BY SC

DATUM Geodetic DATE May 25, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE 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 Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL (WATER CONTENT (%) 20 40 60)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
246.5	Ground Surface										
0.0	Sandy Gravel Some Silt Trace Clay Trace Shells Loose to Compact		1	SS	12	▼			○		36 55 5 4
			2	SS	9		240		○		50 31 15 4
			3	SS	5				○		33 30 29 8
236.0	Boulders										
10.5	Limestone Bedrock Sound		4	RC BXL	REC 95%						RQD 86%
			5	RC BXL	REC 100%		230				RQD 80%
	Shale Bed		6	RC BXL	REC 93%						RQD 70%
220.2											
26.3	End of Borehole										

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 (%) STRAIN AT FAILURE / 10

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 4

W P 134-74-01 LOCATION Sta 469+29 °/s 178' Rt ℄ Exist. Hwy. 14 ORIGINATED BY JW

DIST 8 HWY 14 BOREHOLE TYPE Hollow Stem Augers, BXL Rock Coring Dynamic Cone Test COMPILED BY SC

DATUM Geodetic DATE May 27, 1977 CHECKED BY

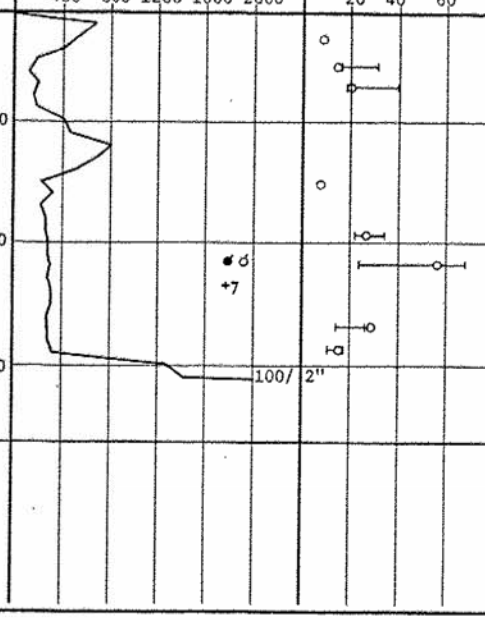
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ PCF	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
248.9	Ground Surface							
0.0	Fill: Sand and Gravel with Inclusions and Layers of Clayey Silt	1	SS	17	*		Om 3.7%	44 31 18 7
		2	SS	4			Om 2.4%	27 40 19 14
		3	SS	PH			Om 7.0%	21 32 26 21
		4	SS	2				
	Trace of Organic Matter With Cobbles and Boulders	5	SS	14		240	Om 0.7%	
233.9		6	SS	11				64 34 (2)
15.0	With Sand	7	SS	8		230		
	Clay Stiff Laminated and Fissured	8	TW	PH			104	
	With Random Layers of Clayey Silt	9	TW	PM			98	
220.9								
28.0 / 218.8	Glacial Till Hard	10	SS	42		220		12 28 37 23
30.1	Limestone Bedrock	11	RC BXL	REC 100%				RQD 60%
213.7	Sound							
35.2	End of Borehole							

* Note: Water Level Not Established

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH PSF: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 400 800 1200 1600 2000

PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60



RECORD OF BOREHOLE No 5

W P 134-74-01 LOCATION Sta 471+83 o/s 60' RT ℄ Exist. Hwy. 14 ORIGINATED BY JW

DIST 8 HWY 14 BOREHOLE TYPE Hollow Stem Augers & Dynamic Cone Test COMPILED BY SC

DATUM Geodetic DATE May 26, 1977 CHECKED BY

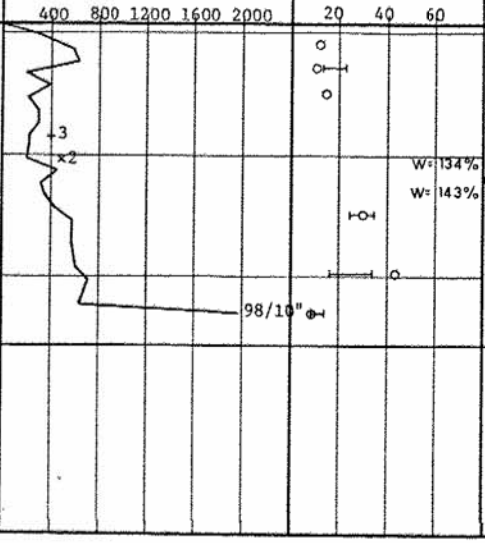
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ PCF	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
250.7	Ground Surface							
0.0	Fill: Sand and Gravel with Inclusions and Layers of Clayey Silt	1	SS	22	*	250		38 34 19 9
		2	SS	3				
243.7		3	SS	14				41 43 10 6
7.0	Organic Clay	4	SS	1				
		5	TW	PH		240	W= 134%	78
236.7	Soft	6	TW	PH			W= 143%	Om 7.5% 98
14.0	Clayey Silt with Random Silt and Sand Seams Hard	7	SS	24				
		8	SS	16				
227.7		9	SS	13		230		
23.0	Glacial Till Het. Mixture Clayey Silt Sand and gravel Hard	10	SS	80/ 3"				11 38 36 15
224.3		11	SS	20/ 3"				
26.4	Refusal to Augering Probable Bedrock End of Borehole							

* Note: Water Level Not Established

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH PSF: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 400 800 1200 1600 2000

PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60



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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 7

W P 134-74-01 LOCATION Sta 467+06 o/s 92' RT ℄ Exist. Hwy. 14 ORIGINATED BY JW
DIST 8 HWY 14 BOREHOLE TYPE Hollow Stem Augers, BXL Rock Coring & Dynamic Cone Test COMPILED BY SC
DATUM Geodetic DATE May 30, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH PSF (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (400 800 1200 1600 2000)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ PCF	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
249.1	Ground Surface										
0.0	Fill: Sand and Gravel some Silt, Trace of Clay Organic Material in upper 2 feet		1	SS	12					Om 4.2%	46 25 19 10
			2	SS	32						
238.1	Cobbles at 3',5',&10'		3	SS	31		240				33 52 13 2
11.0	Sand Medium to Coarse Trace of Shells Compact		4	SS	15						
			5	SS	28						
			6	SS	27						2 93 (5)
							230				
227.1			7	SS	17						3 87 (10)
22.0	Clay with Random Layers of Clayey Silt		8	SS	8						0 4 24 72
			9	SS	3						
218.6	Firm to Stiff						220	+5 +3			
30.5 / 216.3	Glacial Till Hard		10	TW	PH			100/0"		139	
32.8	Limestone Bedrock		11	RC BXL	REC 100%						
210.8	Sound										RQD 63%
38.3	End of Borehole										

RECORD OF BOREHOLE No 8

W P 134-74-01 LOCATION Sta 469+56 o/s 52' Rt. ℄ Exist.Hwy. 14 ORIGINATED BY JW
DIST 8 HWY 14 BOREHOLE TYPE Hollow Stem Augers & Dynamic Cone Test COMPILED BY SC
DATUM Geodetic DATE May 31, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH PSF (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (400 800 1200 1600 2000)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ PCF	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
249.3	Ground Surface										
0.0	Fill: Sand & Clayey Silt Some Organics					*					
245.8			1	SS	5					Om 7.1%	7 54 29 10
3.5	Sandy gravel		2	SS	44						60 30 7 3
	Sand medium to Coarse		3	SS	38		240				
234.3	Dense		4	SS	16/6"						6 87 (7)
15.0	Clay Laminated & Fissured Stiff		5	SS	8						
			6	TW	PH		230			109	
			7	TW	PH			+4 >+2400		107	
224.2											
25.1 / 220.6	Glacial Till Hard Het. Mix Clayey Silt, Sand and Gravel		8	SS	55			135/	9"		19 25 27 29
28.7	Refusal to Augering Probable Bedrock End of Borehole										
	*Note: Water level not Established										

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications, Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 10

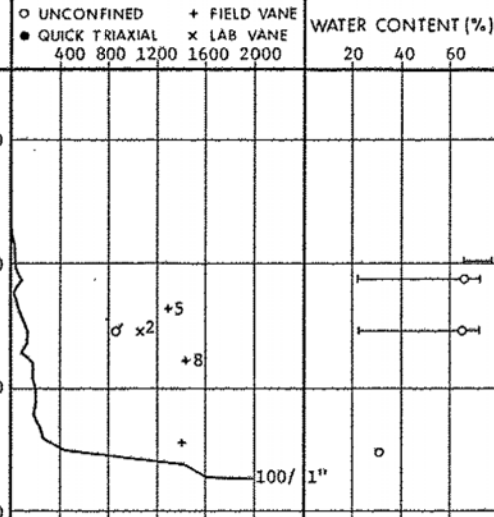
W P 134-74-01 LOCATION Sta. 466+00 o/s 205' RT. ℄ Exist.Hwy. 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX Casing, BXL Rock Coring & Dynamic Cone Test COMPILED BY SC

DATUM Geodetic DATE May 26, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.8	Water Surface				
0.0	Water				
237.8	Bay Bottom				
8.0	Organic Clay V. Soft				
229.3		1	SS	10	W=159%
16.3	Clay Fissured and Laminated Stiff	2	SS	2	
		3	TW	PM	e_o = 2.05 C_c = 1.45
		4	TW	PM	
30.0					
213.3	Glacial Till Hard	5	SS	76	4 64 8 24
32.5	Sound Limestone Bedrock With Shaly Sections	6	RC BXL	REC 97%	RQD 70%
208.3					
37.5	End of Borehole				

Dynamic cone penetration resistance plot (20 40 60 80 100); shear strength PSF (o unconfined, ● quick triaxial, + field vane, x lab vane) 400 800 1200 1600 2000; ground water conditions; elevation scale 240, 230, 220, 210; plastic limit W_p, natural moisture content W, liquid limit W_L; water content (%) 20 40 60; unit weight γ PCF



RECORD OF BOREHOLE No 11

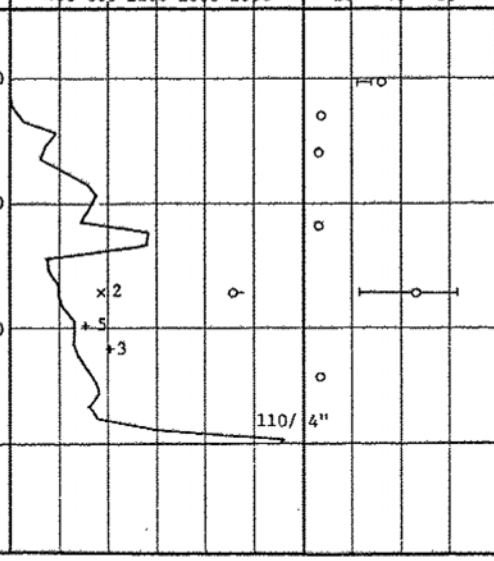
W P 134-74-01 LOCATION Sta 463+10 o/s 130' RT ℄ Exist. Hwy 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX Casing, & Dynamic Cone Test COMPILED BY SC

DATUM Geodetic DATE May 27, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.8	Water Surface				
0.0	Water				
240.5	Bay Bottom				
5.3					
238.8	Organic Clay Very Soft	1	SS	13	Om 4.0%
7.0	Sandy Gravel Trace of Silt Compact to Dense	2	SS	20	67 22 9 2
		3	SS	25	
		4	SS	34	58 32 (10)
		5	SS	42	
		6	SS	49	50 41 (9)
225.8					
20.0	Clay Fissured Laminated Firm to Stiff	7	SS	16	
		8	TW	PH	109
		9	SS	6	
215.8					
30.0	Glacial Till Het. Mix: of Sand and Gravel some Clayey Silt Compact to Very Dense	10	SS	15	39 31 20 10
210.5		11	SS	170	
35.3	Refusal to Driving Casing Probable Bedrock End of Borehole				

Dynamic cone penetration resistance plot (20 40 60 80 100); shear strength PSF (o unconfined, ● quick triaxial, + field vane, x lab vane) 400 800 1200 1600 2000; ground water conditions; elevation scale 240, 230, 220; plastic limit W_p, natural moisture content W, liquid limit W_L; water content (%) 20 40 60; unit weight γ PCF



$+^3$, x^5 : Numbers refer to Sensitivity

20 15 ◇ 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 12

W P 134-74-01 LOCATION Sta 460+79 o/s 243' RT. ℄ Exist. Hwy. 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX Casing, BXL Rock Coring, & Dynamic Cone Test COMPILED BY SC

DATUM Geodetic DATE May 31, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.8	Water Surface							
0.0	Water							
237.8	Bay Bottom							
8.0 / 235.8	Organic Clay Very Soft							
10.0	Sandy Gravel Trace of Silt Compact to Dense	1	SS	29			Om 3.0%	
		2	SS	27				58 24 13 5
225.3		3	SS	53				41 46 (13)
20.5	Clayey Silt Trace of Sand Firm to Stiff	4	TW	PM				
216.3								
29.5	Glacial Till Het. Mixture Sand, Silt, Trace of Clay Compact	5	SS	19				
		6	SS	26				
206.8								
206.0	Limestone Bedrock	7	RC	REC 100%				
39.8	End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)

SHEAR STRENGTH PSF
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
400 800 1200 1600 2000

PLASTIC LIMIT Wp — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%) 20 40 60

ELEVATION SCALE: 240, 230, 220, 210

+2 +5 100/3"

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

20 / 15 ⊹ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 13

W P 134-74-01 LOCATION Sta 457+81 o/s 80' RT ℄ Exist. Hwy. 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX casing, BXL Rock Coring & Dynamic Cone Test COMPILED BY SC

DATUM Geodetic DATE June 2, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.8	Water Surface				
0.0	Water				
236.5	Bay Bottom				
9.3	Organic Clay				
230.0	Very Soft	1	SS	9	W=161.5%
15.8	Sandy Gravel Trace of Silt Compact	2	SS	34	57 34 8 1
		3	SS	15	
219.0		4	SS	14	
26.8	Clay Fissured Stiff	5	TW	PM	97
207.8		6	SS	5	
38.0	Glacial Till Het. Mix of Sand and Gravel Trace of Clayey Silt Compact to Very Dense	7	SS	30	
		8	SS	55	
		9	SS	146	
191.3					
54.5	Sound Limestone Bedrock With Shaly Sections	10	RC BXL	REC 100%	RQD 97%
186.0					
59.8	End of Borehole				

GROUND WATER CONDITIONS

ELEVATION SCALE: 240, 230, 220, 210, 200, 190

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH PSF: O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE; 400 800 1200 1600 2000

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ PCF

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

15 ⟡ 5 (%) STRAIN AT FAILURE (20, 10)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 14

W P 134-74-01 LOCATION Sta 464+58 o/s 196' RT ℄ Exist. Hwy. 14 ORIGINATED BY MM
DIST 8 HWY 14 BOREHOLE TYPE Continuous Vane Tests COMPILED BY SC
DATUM Geodetic DATE June 6, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH PSF (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (400 800 1200 1600 2000)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ PCF	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.8	Water Surface					▼					
0.0	Water										
238.0	Bay Bottom						240				
7.8	Organic Clay Very Soft		1	TP	PM			×2 +8 +2 +2	W= 270% WL= 87% Wp= 67%	64	
229.0							230				
16.8	Refusal to Pushing Vane End of Borehole										

RECORD OF BOREHOLE No 15

W P 134-74-01 LOCATION Sta 461+14 o/s 20'LT ℄ Exist. Hwy. 14 ORIGINATED BY MM
DIST 8 HWY 14 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SC
DATUM Geodetic DATE June 6, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
252.0	Ground Surface										
0.0	Fill Sand, Gravel and Cobbles					*	250				
246.6											
5.4	Refusal to Augering End of Borehole										
	* Note: Groundwater Not encountered										

RECORD OF BOREHOLE No 16

W P 134-74-01 LOCATION Sta 467+10 o/s 242' RT. ℄ Exist. Hwy. 14 ORIGINATED BY MM
DIST 8 HWY 14 BOREHOLE TYPE Continuous Vane Test COMPILED BY SC
DATUM Geodetic DATE June 6, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH PSF (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (400 800 1200 1600 2000)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ PCF	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.8	Water Surface					▼					
0.0	Water										
239.5	Bay Bottom						240				
6.3	Organic Clay Very Soft to Soft		1	TP	PM			×2 +2 +3 +3	W= 282% WL= 92% Wp= 77%	69	
232.0											
13.8	Refusal to Pushing Vane End of Borehole						230				

+3, ×5 : Numbers refer to Sensitivity

20 / 15 ⊕ 5 (%) STRAIN AT FAILURE / 10

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 17

W P 134-74-01 LOCATION Sta 453+52 o/s 84' Rt ℄ Exist. Hwy. 14 ORIGINATED BY MM
DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX Casing & BXL Rock Coring COMPILED BY SC
DATUM Geodetic DATE June 7, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.8	Water Surface				
0.0	Water				
235.0	Bay Bottom				
10.8 / 232.3	Organic Clay Very Soft	1	SS	12	Om 4.0%
13.5	Fill	2	SS	40	50 36 9 5
	Sandy Gravel Trace of Silt	3	SS	7	58 39 (3)
		4	SS	13	
		5	SS	31	36 62 (2)
207.8		6	SS	26	
38.0 / 202.8	Organic Clay Hard	7	SS	19	W=102%
43.0	Glacial Till Het. Mix Sand, Gravel Some silt Trace of Clay Dense Boulders	8	SS	41	14 62 18 6
192.5		9	SS	100/0"	
		10	RC	100%	
53.3	Sound Limestone Bedrock With Shaly Sections	11	RC	50%	
187.5		12	RC BXL	REC 100%	RQD 90%
58.3	End of Borehole				

ELEVATION SCALE: 240, 230, 220, 210, 200, 190

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE; 400 800 1200 1600 2000

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ PCF

GROUND WATER CONDITIONS

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

15 20 10 5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 18

W P 134-74-01 LOCATION Sta 452+48 o/s 18' Lt ℄ Exist. Hwy. 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Solid Stem Augers COMPILED BY SC

DATUM Geodetic DATE June 7, 1977 CHECKED BY CP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (o UNCONFINED, • QUICK TRIAXIAL, + FIELD VANE, x LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						GR SA SI CL
256.0	Ground Surface										
0.0 *	Fill		1	SS	24	**					
			1	SS	24						
	Sand and Gravel with Cobbles to Boulders Very Dense						250				
	Compact						240				
	With Cobbles to Boulders V. Dense Compact						230				
226.0											
30.0	End of Borehole										

* Description inferred from nature of auger operation and material on augers

** Note: GROUNDWATER NOT ESTABLISHED

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to Sensitivity

15 20 10 5 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
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HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 19

W P 134-74-01 LOCATION Sta 447+28 o/s 145' Rt ℄ Exist. Hwy. 14 ORIGINATED BY MM
DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX Casing&BXL Rock Coring COMPILED BY SC
DATUM Geodetic DATE June 13, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH PSF (DYNAMIC CONE PENETRATION RESISTANCE PLOT)	WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.8	Water Surface										
0.0	Water						240				
234.5	Bay Bottom										
11.3	Organic Clay With Sand		1	SS	1/	24"	230		W = 280%	Om 26%	
			2	SS	own wt.			+2 +2 +2			
	V. Soft to Firm						220				
			3	SS	2				W = 181%		
								8 +3			
			4	SS	1		210				
								+3 +2			
			5	SS	2				W = 155%		
							200	+3 +2			
189.0							190				
56.8	Glacial Till Het. Mix of Clayey Silt with Sand and Gravel Hard		6	SS	3				H O		0 5 83 12
185.5			7	SS	100/6"						
60.3	Limestone Bedrock Sound		8	RC BXL	REC 100%						RQD 80%
180.5											
65.3											

Shear strength: o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE. Plastic limit W_p, natural moisture content W, liquid limit W_l.

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

20 15 ⊹ 5 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 20

W P 134-74-01 LOCATION Sta 444+84 o/s 90' RT. ℄ Exist.Hwy. 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX Casing, BXL Rock Coring COMPILED BY SC

DATUM Geodetic DATE June 4, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.6	Water Surface									
0.0	Water					240				
234.3	Bay Bottom									
11.3	Fill	1	SS	2				o	Om.3.0%	
	Sandy Gravel	2	SS	8		230				
	Trace of silt with Shells with decayed and undecayed Wood Chips to Elev. 210.0'	3	SS	48				o	Om.0.5%	11 86 (3)
		4	SS	92		220				
		5	SS	18					Om.13.1%	25 60 (15)
						210				
		6	SS	41				o		53 42 (5)
						200				
193.6		7	SS	120				o		20 72 (8)
52.0	Glacial Till Het. Mix. of sand, gravel, trace silt with cobbles and boulders up to 1"thick	8	RC	100%						
		9	RC	REC	0%	190				
183.3	Very Dense	10	RC	REC	25%					
62.3	Sound Limestone Bedrock	11	RC BXL	REC 94%		180				RQD 80%
177.8										
67.8	End of Borehole									

+3, x5 : Numbers refer to Sensitivity

15 20 10 5 (%) STRAIN AT FAILURE

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Ministry of Transportation and Communications Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 21

W P 134-74-01 LOCATION Sta 450+20 o/s 170' RT. ℄ Exist. Hwy. 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX Casing & BXL Rock Coring COMPILED BY SC

DATUM Geodetic DATE June 15, 1977 CHECKED BY CF

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH PSF (400 800 1200 1600 2000)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ PCF	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.6	Water Surface										
0.0	Water						240				
230.9	Bay Bottom										
14.7	Organic Clay with sand						230				
	Very Soft to Firm		1	SS	2		220		W = 107%		
			2	SS	1/	24"	210	+2 +2 +2			
198.3							200				
47.3	Glacial Till Clayey Silt with Sand Hard		3	SS	25				⊢ o		0 17 71 12
193.0	Limestone Bedrock Sound		4	RC	REC 100%						RQD 50%
191.3											
54.3	End of Borehole										

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

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

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OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 22

W P 134-74-01 LOCATION Sta 437+92 o/s 96' RT ℄ Exist. Hwy. 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX casing, BX Rock Coring COMPILED BY SC

DATUM Geodetic DATE June 15, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (o UNCONFINED, • QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L (WATER CONTENT (%))	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.6	Water Surface										
0.0	Water						240				
							230				
226.3	Bay Bottom										
224.7	Organic Clay V. Soft										
222.8	Glacial Till Loose		1	SS	9						15 42 35 8
22.8	Sound Limestone Bedrock		2	RC BXL	REC 100%		220				RQD 80%
217.6	With Shaly Sections										
28.0	End of Borehole										

RECORD OF BOREHOLE No 23

W P 134-74-01 LOCATION Sta 439+24 o/s 73' RT. ℄ Exist. Hwy. 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX Casing COMPILED BY SC

DATUM Geodetic DATE June 15, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (o UNCONFINED, • QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L (WATER CONTENT (%))	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.6	Water Surface										
0.0	Water						240				
							230				
							220				
211.3	Bottom of Bay										
34.3	Refusal to Driving Casing Probable Bedrock End of Borehole										

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

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

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HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 24

W P 134-74-01 LOCATION Sta 441+92 o/s 108' RT. ℄ Exist. Hwy. 14 ORIGINATED BY MM

DIST 8 HWY 14 BOREHOLE TYPE Washboring with NX Casing COMPILED BY SC

DATUM Geodetic DATE June 16, 1977 CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (o UNCONFINED, • QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
245.6	Water Surface										
0.0	Water						240				
							230				
							220				
215.3	Bay Bottom										
30.3	Organic Clay With Sand V. Soft						210				
			1	SS	1				W= 264%		
							200				
195.3											
50.3	Refusal to Driving Casing Probable Bedrock End of Borehole										

+3, ×5 : Numbers refer to Sensitivity

15 ⊕ 5 (20, 10) (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. _____ SHEET NO. _____

PROPERTY: W.P. 134-74-01
LOCATION: Hwy. 14, Belleville, Ontario
LATITUDE:
DEPARTURE:
BEARING:

DIP: 90°
TOTAL FOOTAGE:

ELEV. COLLAR:
DATUM:
DATE STARTED:
DATE COMPLETED:
DRILLED BY:
LOGGED BY:

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER	% Shale		REMARKS
		HOLE #7				
33'0"	38'3"	Limestone, med. grey colour, med. texture, hard, thinly bedded, shaly sections, horizontal breakage throughout length of core.		1%		Trenton formation RQD 45%
		HOLE #2				
13'0"	20'9"	Limestone, med. grey colour, fine, med. & coarse texture		2%		Trenton formation RQD 80%
20'9"	26'3"	Limestone, med. grey colour, med. texture, hard, thinly bedded, shaly sections, broken core, 23' - 24' shale		15%		RQD 30%
		HOLE #21				
52'6"	55'0"	Limestone, med. grey colour, med. texture, hard, thinly bedded, shaly sections, horizontal breakage throughout length of core. Boulders 52'6" - 53'4"		0.5%		RQD 53'4" to 55'0" - 15%
		HOLE #4				
30'1"	35'2"	Limestone, med. grey colour, coarse to med. texture.		2%		RQD 45%

DATE OF EXAMINATION: June 28, 1977

B.K. Glassford

08-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. _____ SHEET NO. _____

PROPERTY: W.P. 134-74-01
LOCATION: Hwy. 14, Belleville, Ontario
LATITUDE
DEPARTURE
BEARING

DIP: 90°
TOTAL FOOTAGE

ELEV. COLLAR
DATUM
DATE STARTED
DATE COMPLETED
DRILLED BY
LOGGED BY

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER	% Shale		REMARKS
		HOLE #13				
55'3"	60'6"	Limestone, med. grey colour, med. texture, hard, thinly bedded, shaly sections, horizontal breakage throughout length of core.		3%		RQD 35%
		HOLE #10				
33'4"	38'4"	Limestone, med. grey colour, med. texture, hard, thinly bedded, shaly sections, horizontal breakage throughout length of core.		1%		RQD 35%

DATE OF EXAMINATION June 28, 1977

B.K. Glassford

08-MT-113

Ministry of Transportation and Communications
Ontario

DIAMOND DRILL RECORD

HOLE NO. _____ SHEET NO. _____

PROPERTY: W.P. 134-74-01
LOCATION: Hwy. 14, Belleville, Ontario
LATITUDE:
DEPARTURE:
BEARING:

DIP: 90°
TOTAL FOOTAGE:

ELEV. COLLAR:
DATUM:
DATE STARTED:
DATE COMPLETED:
DRILLED BY:
LOGGED BY:

FOOTAGE FROM	FOOTAGE TO	FORMATION	SAMPLE NUMBER	% Shale		REMARKS
		HOLE #22				
23'6"	28'9"	Limestone, med. grey colour, med. texture, hard, thinly bedded, shaly sections, horizontal breakage throughout length of core.		3%		Trenton formation RQD 18%
		HOLE #19				
61'0"	66'0"	Limestone, med. grey colour, med. texture, hard, thinly bedded, shaly sections, horizontal breakage throughout length of core.		1%		Trenton formation RQD 20%
		HOLE #20				
55'3"	68'6"	Limestone, med. grey colour, med. to fine texture, ground and lost core 55'3" to 63'0".		0.5%		Trenton formation RQD 63' to 68'6" - 80%
		HOLE #17				
50'5"	51'0"	boulders		1%		Trenton formation
51'0"	54'0"	ground and lost core				RQD 54'0" - 59'0" - 45%
54'0"	59'0"	Limestone, med. grey colour, med. texture, hard, thinly bedded, shaly sections, horizontal breakage throughout length of core.				

DATE OF EXAMINATION: June 28, 1977

B.K. Glassford

OB-MT-113