THURBER ENGINEERING LTD.

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
GRAND RIVER BRIDGE REPLACEMENT
HIGHWAY 401
REGIONAL MUNICIPALITY OF WATERLOO, ONTARIO
G.W.P. 3080-12-00, Site No. 33-141

Geocres Number: 40P8-246

Report to

WSP

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January 18, 2018
File: 11373

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PART 1: FACTUAL INFORMATION

1.0 INTRODUCTION

This report presents a description of the subsurface conditions anticipated at the location of the bridge carrying Highway 401 over the Grand River in Kitchener, Ontario. The description is based solely on borehole information presented in the following report:

> Subsurface Exploration and Foundations for Waterloo Township Bridge No. 12, County of Waterloo, Ontario. Geocres No. 40P08-023. Universal Geotechnique Limited, November 1958.

The purpose of the report is to present existing borehole logs, a borehole location plan, a stratigraphic profile, and a written description of the subsurface conditions at the site.

Thurber completed the report as a sub-consultant to WSP who are completing the preliminary design of a replacement bridge under the Ministry of Transportation Ontario (MTO) Agreement Number 3015-E-0013.

2.0 SITE DESCRIPTION

The existing Grand River Bridge consists of twin six-span structures, each with a total length of 237.1 m and width of 14.7 m. The span lengths are 39.3 m at the abutments and 39.6 m between piers. Each bridge accommodates three lanes of Highway 401 traffic. The clearance between the twin structures is 4.2 m. The bridges are supported on spread footings, and the foundation units are aligned on a 25° skew.

Road grades on Highway 401 rise from west to east on the structures, from approximate Elev. 280.3 to 287.1 on the eastbound bridge, and from approximate Elev. 280.7 to 287.4 on the westbound bridge. The Grand River channel bed is near Elev. 272.0 under the crossing. The west approach is located on an approximate 5 m high fill embankment constructed within the wide river floodplain, and the east approach is located within an approximate 8 to 9 m deep earth cut excavated into the east valley wall.

Photographs of the site are presented in Appendix A.

The site is located at the south end of Kitchener near the boundary with Cambridge to the south, and approximately 600 m west of King Street East. The adjacent lands comprise a golf course on both sides of the west approach, an established residential subdivision to the north of the east approach, and a residential subdivision under development to the south of the east approach.

The study area is located within the Waterloo Hills physiographic region, an area of sandy hills kames, kame moraines, and ridges of sandy till, with outwash sands occupying the intervening hollows. The Grand River spillway system adjoins the hilly region, within which sand and gravelly alluvial materials are present. Bedrock lies at relatively shallow depth below the spillway (3 to 8 m at the bridge site) and consists of dolomite and shale of the Salina formation.

3.0 DESCRIPTION OF SUBSURFACE CONDITIONS

The field investigation carried out in 1958 for the existing bridge structure consisted of exploratory boreholes at ten locations and electrical resistivity soundings at seven locations along the proposed bridge alignments. One borehole (Borehole 21 at the east approach) was terminated above bedrock at 10.5 m depth, and the remaining boreholes (Boreholes 9, 10, 12, 13/13A, and 16 to 21) were terminated within bedrock at depths of 5.9 to 14.3 m. The geophysical soundings were extended to depths of 9.1 to 10.0 m (RS 8, and RS 11 to 15), locally 24.7 m at the east approach (RS 21).

Reference is made to the Borehole Logs and resistivity sounding records from the previous investigation, reproduced in Appendix B, for details of the encountered soil stratigraphy. The borehole locations, along with a stratigraphic profile prepared from the existing data, are shown on the "Borehole Locations and Soil Strata" drawing in Appendix D. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Log of Borehole Sheets governs any interpretation of the site conditions.

In general terms, the subsurface stratigraphy encountered in the boreholes drilled within the river floodplain consisted of alluvial sand, gravel and boulders, locally overlain by peat and underlain by clay, overlying poor quality bedrock encountered at depths of 2.7 to 8.2 m in the boreholes. Locally above the east valley slope, the stratigraphy comprised sand over sandy clay

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underlain by clayey gravelly sand to the borehole termination depth. More detailed descriptions of the individual strata are presented below.

3.1 Organic Deposits

A 1.6 to 2.4 m thick layer of peat was encountered in two boreholes (Boreholes 9 and 10) drilled adjacent to the east bank of the river. The peat was described as very soft and dark brown to black. A further 0.7 m of very loose, grey to dark grey silt with organic matter was encountered below the peat in Borehole 9. The lower boundary of the organic materials was encountered at depths of 2.3 and 2.4 m (Elev. 272.3 and 272.4).

A 150 to 1200 mm thick layer of topsoil, identified as loam to sandy loam on the borehole logs, was encountered in four boreholes (Boreholes 17 to 20) drilled on the floodplain to the west of the river, and in one borehole (Borehole 21) drilled above the east valley slope.

3.2 Sand, Gravel and Boulders

Alluvial materials consisting primarily of sand, gravel and boulders were encountered below the river channel and organic deposits in the floodplain. The total thickness of these materials ranged from 0.8 to 6.7 m, generally increasing towards the west.

SPT 'N' values obtained in the sand and gravel typically ranged from 54 blows for 0.3 m to 60 blows for 0.075 m of penetration, indicating a very dense condition, and may also reflect the presence of cobbles and boulders. One 'N' value of 28 blows per 0.3 m (compact) was obtained in Borehole 17.

3.3 Sand

Sand was encountered below the topsoil in Borehole 21 drilled above the east valley slope. The sand layer was 4.6 m thick, with a lower boundary at 4.9 m depth (Elev. 294.4). Additional sand layers were encountered within/below the sandy clay layer underlying the upper sand deposit in this borehole, at depths of 8.7 and 9.9 m. The sand deposits varied from fine grained sand, to fine to coarse sand with gravel, to clayey gravelly sand at the base of the borehole. Borehole 21 was terminated in the clayey gravelly sand at 10.5 m depth (Elev. 288.8).

SPT 'N' values recorded in the sand deposits varied from 12 to 53 blows per 0.3 m penetration, indicating a compact to very dense relative density. An 'N' value of 102 blows per 0.3 m was obtained in the sand at the base of the borehole.

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3.4 Clay

Clay units were encountered below the sand and gravel deposits as follows:

- Sandy clay with gravel (to gravelly) was encountered below the alluvial deposits in Boreholes 9 and 10 at the east river bank and in Borehole 20 at the west abutment. The sandy clay was generally described as hard, with SPT 'N' values of 69 blows per 0.3 m to 74 blows per 0.15 m. The clay layer was approximately 0.9 to 3.6 m thick.

- A layer described as hard grey clay with fragments of shale (probable badly weathered shale) was encountered between the alluvial deposits and the underlying bedrock in Boreholes 17 to 19. This layer was 0.5 to 1.9 m thick.

- A 3.8 m thick layer of sandy clay with gravel (to gravelly) and boulders was encountered below the sand layer in Borehole 21 drilled above the east valley slope. A second layer, 0.5 m thick, was encountered within sand interbeds near 9.5 m depth. The sandy clay was described as very stiff to hard, with SPT 'N' values of 15 blows per 0.15 m to 96 blows per 0.18 m.

3.5 Bedrock

Bedrock was encountered below the sandy clay and sand and gravel deposits in all boreholes except Borehole 21. The bedrock was proven by coring. The depths and elevations at which bedrock was encountered are summarized in Table 3.1.

Table 3.1 – Bedrock Depths and Elevations

Borehole	Location	Bedrock Surface	
		Depth (m)	Elevation
9	East riverbank	4.9	269.8
10	East riverbank	5.0	269.8
12	River channel	3.2	268.4
13	River channel	3.4	268.2
13A	River channel	2.7	268.9
16	West riverbank	4.6	267.1
17	West floodplain	8.2	266.3
18	West floodplain	7.7	266.7
19	West floodplain	7.6	267.3
20	West floodplain	6.6	268.1

The bedrock surface elevation is subject to interpretation as a result of the highly weathered nature of the rock. Of note, the 0.5 to 1.9 m of clay identified above the bedrock surface in Boreholes 17 to 19 may represent completely weathered shale bedrock. In addition, the upper 1.7 m thick zone identified at the bedrock surface in Borehole 20, described as fragments of limestone and calcareous shale, may represent re-deposited material.

The bedrock consists of interbeds of grey shale, calcareous shale, argillaceous limestone and limestone, with isolated interbeds of buff dolostone. In general, the bedrock is highly weathered, with some shale beds completely weathered to resemble a hard clay. Solution cavities up to 13 mm in diameter were observed in selected cores of limestone and dolomite, and seams of anhydrite and gypsum were noted in cores from Boreholes 13A and 20. Core recovery varied widely from 11% to 92%. RQD values were not recorded.

3.6 Groundwater Conditions

The groundwater conditions recorded during drilling are summarized in the following table.

Table 3.2 - Groundwater Observations

Borehole	Approximate Water Level (m)		Comment
	Depth	Elevation	
9	-	-	Not documented
10	9.2 0.4 ags	265.7 275.2	Initial measurement Artesian in bedrock
12	0.8 ags	272.3	Artesian in bedrock
13	-	-	Not documented
13A	0.7 ags	272.3	Artesian in bedrock
16	0.4 ags	272.1	Artesian in bedrock
17	-	-	Not documented
18	2.4	272.0	
19	2.3	272.6	
20	2.1	272.6	
21	-	-	Not documented

* ags = above ground surface

The above water level measurements are short-term observations and seasonal fluctuations of the groundwater level and the water level in the river are to be expected. In particular, the water levels may be higher after the spring snowmelt or after periods of heavy rainfall.

The water level in the Grand River at the time of investigation (October 1958) was approximate Elev. 272.0 m. The depth of water was reported to be about 0.3 to 0.9 m.

3.7 Geophysical Survey

Electrical resistivity soundings were carried out at the locations of Boreholes 12, 13 and 21 (designated RS 12, 13 and 21), and at four additional locations (RS 8, 11, 14 and 15). The bedrock depths and elevations interpreted from the soundings are summarized in Table 3.3.

Table 3.3 – Bedrock Depths and Elevations Interpreted from Soundings

Resistivity Sounding	Location	Estimated Bedrock Surface	
		Depth (m)	Elevation
RS 8	Above east slope	> 10.0	< 277.4
RS 11	River channel	3.2	268.4
RS 12	River channel	2.7	269.3
RS 13	River channel	2.3	269.8
RS 14	River channel	2.7	269.3
RS 15	West riverbank	2.3	269.8
RS 21	Above east slope	> 24.7	< 274.8

The bedrock surface elevation is subject to interpretation as a result of the highly weathered nature of the rock, the presence of cobbles and boulders in the overlying deposits, and intrinsic limitations of the geophysical survey methods employed.

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4.0 MISCELLANEOUS

The description of subsurface conditions at the site is based solely on borehole information presented in the following report:

> Subsurface Exploration and Foundations for Waterloo Township Bridge No. 12, County of Waterloo, Ontario. Geocres No. 40P08-023. Universal Geotechnique Limited, November 1958.

Thurber provides no warranty and does not accept responsibility for the accuracy or reliability of the information presented on the borehole logs prepared by others. Additional boreholes will be required during detailed design to confirm the subsurface conditions at the locations of the structure foundation units and bridge approaches.

Interpretation of the field data and preparation of the report were performed by Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

5.0 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents preliminary geotechnical recommendations to assist selection and preliminary design of the foundation system for the replacement of the Grand River Bridge carrying Highway 401.

Replacement of the existing six-span twin bridge structure with a new four-span structure is proposed. The proposed bridge will have a total length of 225 m between abutments, approximately 12 m shorter than the existing structure, and the new abutments will be placed inside of the existing abutments. The span lengths for the new bridge will range from 40.0 to 75.0 m.

The width of the structures will be increased from 14.7 m each (three lanes) to 30.3 m for the westbound structure and 23.3 m for the eastbound structure. The clearance between structures will be 2.0 m. The existing 25° skew of the abutments and piers will be maintained. Proposed road grades on Highway 401 will be near existing grades.

The discussion and recommendations presented in this report are based on the information provided by WSP and on the factual data obtained during the previous investigation on site.

The interpretation and recommendations are intended for the use of the design consultant and the Ministry of Transportation (MTO), and shall not be relied upon by any other parties including the construction contractor, or used for any purposes other than development of the project design. Comments on construction methodology and equipment, where presented, are provided only to highlight those aspects that could affect the design of the project. Contractors must make their own assessment of the factual information presented in Part 1 of the report, and the implications on equipment selection, construction methodology, and scheduling.

The report references the Canadian Highway Bridge Design Code published in December 2014 (CHBDC 2014) by the CSA Group. In accordance with the CHBDC 2014, a consequence classification of "typical consequence" and a degree of site and prediction model understanding of "typical understanding" have been assumed.

6.0 PRELIMINARY FOUNDATION DESIGN

In general terms, the subsurface stratigraphy encountered in the boreholes drilled within the river floodplain consisted of alluvial sand, gravel and boulders, locally overlain by peat and underlain by clay, overlying poor quality bedrock encountered at depths of 2.7 to 8.2 m in the boreholes. Locally above the east valley slope, the stratigraphy comprised sand over sandy clay underlain by clayey gravelly sand to the borehole termination depth.

The water level in the Grand River at the time of investigation (October 1958) was at approximate Elev. 272.0 m. The depth of water was reported to be about 0.3 to 0.9 m. Artesian head rising to about 0.4 to 0.8 m above the ground surface was observed in four boreholes.

Based on the subsurface conditions at the site, consideration was given to supporting the bridge using the following foundation types:

- Spread footings on native soil or bedrock
- Driven steel H-piles
- Socketed H-Piles
- Drilled-in Pipe Piles
- Drilled shafts (caissons)
- Micropiles

A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix C. Preliminary recommendations for feasible foundation alternatives are presented in the following sections. A foundation scheme preferred from a foundations perspective is then recommended.

6.1 Spread Footings on Native Soil

Supporting the piers and west abutment on spread footings constructed on the floodplain and river bed deposits is not recommended in view of the potential for future scour and undermining of the footings. Limited information is available on the density of the deposits overlying the bedrock, and therefore the support capability of these materials cannot be assessed at the current time. Further, the native soils may have been disturbed during construction of the existing bridge foundations. Therefore, recommendations for spread footings have not been developed for these foundation units.

The use of spread footings could be considered for the east abutment. The borehole drilled at the east abutment (Borehole 21) was terminated at Elev. 288.8, approximately 2 m above existing road grade, in very dense clayey gravelly sand. Assuming the very dense conditions continue below the exploration depth, preliminary design of spread footings founded on the very dense native soils at or below the existing footing level may be carried out using factored geotechnical resistances of 600 kPa and 400 kPa at factored ULS and SLS, respectively.

The depth of footing embedment and geotechnical resistances must be reviewed during detailed design with consideration of the proximity of the footing to the slope face. The resistance values may need to be reduced subject to soil type, footing position and slope angle.

The resistance values are for a minimum 2 m wide footing subjected to vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance values used in design must be reduced in accordance with the CHBDC Clauses 6.10.2 to 6.10.4.

The geotechnical resistances at SLS are based on an estimated settlement not exceeding 25 mm. This settlement should be essentially complete by the end of construction.

The lateral resistance developed along the base of concrete footings founded on the very dense sand may be computed using an ultimate friction coefficient of 0.5.

6.2 Spread Footings on Bedrock

Extending spread footings down to bear on the underlying bedrock, as per the existing bridge foundations, may be considered for support of the piers and west abutment. Supporting the east abutment on spread footings on bedrock is not considered feasible in view of the significant inferred depth to bedrock.

The bedrock is highly weathered, of poor quality, and contains gypsum layers and solution cavities. In consideration of the varied quality of the bedrock, the GEOCRES report recommended that footings be founded at least 0.9 m below the bedrock surface and designed using an allowable bearing capacity (working stress design) of 6 tons/sq.ft. (575 kPa).

For evaluation of the design concept, it is recommended that a similar embedment depth (minimum 0.9 m below the bedrock surface) be employed and relatively low geotechnical resistance values of 750 kPa and 500 kPa at factored ULS and factored SLS be assumed for preliminary design of footings on bedrock.

For preliminary estimation purposes, the following average bedrock depths and elevations may be assumed at the foundation units, based on the existing borehole information:

Table 6.1 – Approximate Bedrock Depths and Elevations

Foundation Unit	Reference Boreholes	Approximate Bedrock Surface	
		Depth (m)	Elevation
West Abutment	19, 20	7.0	267.7
Pier 1	17, 18	8.0	266.5
Pier 2	13, 13A, RS13, RS14, RS15, 16	3.0	268.8
Pier 3	9, 10, RS11, 12, RS12	4.0	269.2

The lateral resistance developed along the base of concrete footings founded on the weathered bedrock may be computed using an ultimate friction coefficient of 0.5.

Excavation for footing construction will require cofferdam installation and dewatering. Installation of sheet pile cofferdams may be problematic depending upon the relative density of the river bed deposits and the frequency/size of boulders. Dewatering equipment may need to handle significant flow volumes in view of the permeable nature of the weathered bedrock and overlying granular deposits.

6.3 Driven Steel H-Pile Foundations

Driving of steel H-piles to bedrock at the piers and west abutment is expected to be problematic due to the presence of cobbles, boulders and rock fragments in the river bed deposits overlying the bedrock. Depending upon the relative frequency and size of boulders in these deposits, the piles may encounter refusal above the bedrock surface or be damaged during driving. Further, the pile length may be inadequate due to the locally shallow depth to rock.

Similarly, driving of piles at the east abutment is expected to require pre-augering to enable the piles to penetrate a sufficient depth into the very dense till soils to achieve lateral fixity.

In view of these concerns, the use of driven steel H-piles is not recommended to support the replacement bridge, and this option has not been developed further.

6.4 Socketed H-Pile Foundations

Consideration may be given to socketing steel H-piles into bedrock to support the structure. Installation of the piles would involve augering to the bedrock surface, augering and/or coring as required to form a minimum 1.5 m deep socket into the bedrock, inserting the pile, and grouting the annular space in the socket with concrete. The actual length of socket will need to be determined by additional investigation and coring. A socket diameter approximately 200 mm larger than the largest dimension (corner to corner) of the pile will be required.

For preliminary design purposes, a factored geotechnical resistance of 1,600 kN per pile is recommended for steel HP 310x110 piles socketed at least 1.5 m into bedrock. The SLS resistance will not govern design. Downdrag on the piles is not an issue at this site.

The approximate depths to the bedrock surface anticipated at the piers and west abutment are summarized in Table 6.1. The elevation of the bedrock surface was not investigated at the east abutment.

Socketing operations may be difficult and significantly impacted by the presence of boulders and rock fragments in the river bed deposits as well as the highly weathered condition of the underlying bedrock. A temporary liner may be required to support the auger hole in the cohesionless river bed deposits. Pile cap construction will require cofferdam installation and dewatering.

6.5 Drilled-in Pipe Piles

The replacement bridge may be supported on drilled-in steel pipe piles socketed into bedrock and filled with concrete. This option requires a rock cutting shoe at the tip of the pipe pile and involves installing the piles using a rotational method such as the Symmetrix concentric drilling system.

The use of concrete-filled pipe piles is advantageous when socketed into very strong bedrock such that the capacity will be dictated by the structural resistance of the composite pile section, and will not be governed by the geotechnical resistance of the bedrock. The axial geotechnical resistances recommended for preliminary assessment of several pipe pipe sections (concrete-filled) drilled into bedrock are presented in Table 6.2. The SLS resistance will not govern design.

Table 6.2 – Preliminary Axial Resistances for Drilled-in Pipe Pile Design

Pipe Pile Section		Factored Axial Resistance at ULS (kN)
Outer Diameter (mm)	Wall Thickness (mm)	
324	9.5	2,000
457	12.7	4,000
610	12.7	6,300
762	15.6	9,700

The preliminary axial resistance values assume a steel yield strength of 245 MPa and a concrete compressive strength of 30 MPa. The computed capacity includes a reduction factor of 75% as per Clause 6.11.4.4 of the OHBDC. The structural resistance of the pile must be reviewed by the structural designer. The resistance may need to be further reduced to account for section loss resulting from potential corrosion of the piles.

To penetrate the upper highly weathered zone of the bedrock at this site and achieve a resistance value practical for design, it will be necessary to socket the piles at least 5 m into the bedrock to found on sound limestone and dolostone. The length of socket will need to be confirmed by additional investigation and coring.

The Contractor's drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles, boulders or rock fragments in the overburden soils. Care must be exercised while drilling into the bedrock; the drilling methodology must be capable of advancing the pile without disturbing or fracturing the bedrock at the base of the pile.

6.6 Drilled Shafts (Caissons)

The use of caissons socketed into bedrock may be considered. However, socketing operations may be difficult and significantly impacted by the presence of boulders and rock fragments in the river bed deposits as well as by the highly weathered condition of the underlying bedrock. A temporary liner may be required to support the auger hole in the cohesionless river bed deposits. Further, dewatering of the caisson may not be possible and tremie methods may be required to place the concrete.

Based on the existing coring information, it is recommended that preliminary caisson design be based on a minimum 5 m socket length below the bedrock surface to penetrate the upper highly weathered zone of the bedrock and found on sound limestone and dolostone. The length of socket will need to be confirmed by additional investigation and coring during detailed design.

The axial geotechnical resistances recommended for preliminary design of caissons socketed 5 m into bedrock are presented in Table 6.3. The SLS resistance will not govern design.

Table 6.3 – Axial Resistances for Preliminary Caisson Design

Caisson Diameter (m)	Socket Length (m)	Factored Axial Resistance at ULS (kN)
0.9	5.0	5,000
1.2	5.0	7,000
1.5	5.0	9,500
1.8	5.0	12,000

The preferred base elevations and recommended axial resistances will need to be reviewed and modified subject to investigation during detailed design.

Downdrag on the caissons is not considered to be an issue at this site.

6.7 Micropiles

Micropiles socketed into the bedrock may be considered. In view of their smaller diameter, micropile installation may be less impacted by the bouldery material and bedrock conditions than larger diameter caissons, however production would still be slowed. The length of the micropiles will need to be determined by additional investigation and coring. Pile cap construction will require cofferdam installation and dewatering.

MTO should be consulted to determine their willingness to consider the use of micropiles to support a large bridge structure carrying a major highway.

6.8 Recommended Foundation

From a geotechnical perspective, the preferred foundation option to support the piers and west abutment of the replacement bridge comprises spread footings constructed on bedrock. However, considering the depth to bedrock, the need for extensive cofferdam construction and dewatering, as well as potential environmental concerns related to construction in the river, the use of drilled-in pipe piles or caissons may expedite construction and minimize site impacts.

The preferred foundation system at the east abutment consists of spread footings founded on very dense till forming the east valley slope. However, the use of augered caissons founded in the till or on bedrock should be considered depending upon the position of the new abutment relative to the existing bridge foundations and the face of the river valley slope.

7.0 FROST COVER

The depth of frost penetration at this site is 1.4 m. The base of footings or pile caps must be provided with a minimum of 1.4 m of earth cover as protection against frost action.

8.0 ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the abutments should consist of free-draining granular material conforming to OPS Granular A or B Type II specifications. The granular material should be placed to the extents shown in OPSD 803.010.

Heavy compaction equipment should not be used adjacent to the abutment walls. Compaction should be carried out in accordance with OPSS 501.

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

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

Where:
- p = horizontal earth pressure on the wall at depth h (kPa)
- K = earth pressure coefficient (see table below)
- γ = 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)

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

The parameters in the table correspond to full mobilization of active and passive earth pressures, and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC.

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Table 8.1 – Lateral Earth Pressure Coefficients

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2$ kN/m^3	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.39*	0.31	0.47*
At-rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

* For wing walls.

In accordance with Clause 6.12.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

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 preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B, Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves towards the soil mass.

The design of the abutment walls must incorporate measures such as weep holes and/or subdrains to permit drainage of the backfill and avoid the potential build-up of hydrostatic pressures behind the walls.

9.0 EMBANKMENT SLOPES

The west bridge approach is located on an approximate 5 m high fill embankment constructed within the wide river floodplain, and the east approach is located within an approximate 8 to 9 m deep earth cut excavated into the east valley wall. Widening of the highway on the new bridge approaches will require widening of both the west approach embankment and the east cut section.

Embankment widening should be carried out in accordance with OPSS.PROV 206. Materials used to construct the embankment widening should comprise granular materials or Select Subgrade Material (SSM) in compliance with OPSS.PROV 1010, earth borrow as per OPSS 212, or on-site inorganic materials subject to geotechnical approval. Where new embankment fill is placed against the existing embankment slopes of the west approach, the existing fill slope must be benched in accordance with OPSD 208.010.

The embankment slopes are expected to be stable with side slopes inclined no steeper than 2H:1V. Mid-height berms comprising 2 m wide benches should be incorporated along the length of embankments with heights exceeding 8 m. Settlement of the embankments due to compression of the foundation subgrade is generally expected to be less than 25 mm, provided all peat and organic materials are removed from the embankment footprint.

Permanent roadway cuts along the east approach are expected to be stable with side slopes constructed no steeper than 2H:1V in the native soils. Flatter slopes may be required where loose soils or groundwater seepage (from perched zones or the regional groundwater table) is encountered. Earth cut slopes greater than 6 m high should be provided with a 2 m wide mid-height berm.

Embankment slopes must be provided with erosion protection in accordance with OPSS.PROV 804. Typically, rock protection should be provided over all surfaces with which river flow is likely to be in contact. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion. Surface water should be directed away from the embankment slopes and conveyed down the slope in appropriately designed drainage channels or storm sewers.

10.0 EXCAVATION AND GROUNDWATER CONTROL

All excavation must be carried out in accordance with OPSS 902 and the Occupational Health and Safety Act (OHSA). For the purposes of assessing excavation slope requirements in compliance with the OHSA, the alluvial river channel deposits are classified as Type 3 soils above the water level and Type 4 below. The underlying dense to very dense/hard soils above the east valley slope are classified as Type 2 soil.

Where temporary excavations cannot be constructed with inclined slopes due to space limitations, roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

Excavation for footing or pile cap construction within the river flood plain will require cofferdam installation and dewatering. Installation of sheet pile cofferdams may be problematic depending upon the relative density of the river bed deposits and the frequency/size of boulders. The sheet piles should be driven to bedrock if possible.

Groundwater control measures must be implemented during construction and prior to excavating below the river water level. Dewatering equipment may need to handle significant flow volumes in view of the permeable nature of the weathered bedrock and overlying granular deposits. Well-points installed around the proposed excavation in conjunction with interlocking steel sheet piling cutoff around the foundation excavation may provide a suitable system. A dewatering specialist should be consulted to provide input on the required dewatering system.

11.0 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Staging of the bridge replacements must be carried out in a manner that minimizes the potential for disturbance of functioning bridge foundations adjacent to the work area. The GA drawing for the existing bridge indicates that the piers and west abutment are supported on bedrock and the east abutment is supported on footings on native soil above the east valley slope. Foundation installation activities must consider the potential for disturbance to the subgrade on which these foundations are constructed.

- The existing footings at the west abutment may interfere with new footing construction or caisson/pipe pile installation. It is recommended that the existing abutment and footings be removed prior to installation of new foundations at this location, or that the new abutment be positioned to avoid the existing foundations.

- The river valley deposits are locally very dense and contain boulders and rock fragments. In addition, the soils underlying the east valley slope are very dense. These conditions may have a significant impact on excavation activities, pile driving, socket or caisson augering, and sheet pile installation. The feasibility of the various construction activities and measures to mitigate the impact will need to be addressed during detailed design.

- Temporary steel liners may be required during construction to support the caisson or socket hole sidewalls in the cohesionless valley base deposits.

- At the pier and west abutment locations within the river flood plain, excavation for foundation construction will require dewatering and excavation support systems such as sheet pile cofferdams. In view of the highly weathered nature of the bedrock, significant inflow of water may be experienced even with sheet piles driven to the bedrock surface.

- An artesian groundwater condition was identified locally in the bedrock. The artesian condition should be further investigated and the impact on foundation construction evaluated.

- Given the occurrence of gypsum inclusions in the bedrock, sulphate resistant concrete should be used for all foundation elements where applicable. The potential for corrosion of steel piles must also be taken into consideration.

Further subsurface investigation, analysis and design must be carried out during detailed design to confirm the soil and bedrock conditions at the location of the structure foundation elements and approaches.

12.0 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
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Jan 18/18
PROVINCE OF ONTARIO

Murray R. Anderson, P.Eng.
Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
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Dr. P.K. Chatterji, P.Eng.
Review Principal

THURBER

Appendix A

Site Photographs

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Photograph 1 – South side of bridge looking west from east abutment

Photograph 2 – East approach looking east from south end of east abutment

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Photograph 3 – North side of bridge looking west from east bank of river

Photograph 4 – West pier and abutment, north side of bridge

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Photograph 5 – North side of bridge looking east from west river bank

Photograph 6 – South side of bridge looking east from west river bank

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Appendix B

Record of Borehole Sheets from Previous Investigation

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 9 DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	900.92			Zero			
Very soft dark brown to black somewhat silty PEAT.			● 1			2	Moist.
do Very loose grey to dark grey SILT with organic matter.			● 2	5'3"		Zero	Moist. Medium dry strength.
				7'6"			
Dense grey to brown sand, gravel and some clay.			● 3			92	Wet.
				13'0"			
Hard brown gravelly very sandy CLAY with fine to medium generally subangular gravel.			● 4			64(6")	Damp. High dry strength.
				15'11"			
Grey limestone with argillaceous bands and some small solution cavities. Joints parallel to 30° to core length.							15'11" to 19'2" core recovery 61%
do Some joints iron stained. Solution cavities from 20'3" to 21'3".							19'2" to 23'9" core recovery 60%
Grey argillaceous limestone with thin interbeds of dark grey and greenish grey shale.				24'5"			23'9" to 24'5" core recovery 85%
		End of Borehole					

SCALE: 1" = 5'0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

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SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 10 DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	901.65			Zero			
Very soft black to dark brown PEAT.			• 1			1	Moist.
				8'0"			
Dense grey brown SAND and GRAVEL, little clay.			• 2	10'9"		64	Moist.
Hard brown very sandy CLAY with fine to medium subangular gravel.			• 3			74(6")	Damp. High dry strength.
Grey limestone with some small solution cavities. Jointing parallel to core length. Solution cavities zone at about 19'0" to 20'0".				16'6"			16'6" to 20'6" core recovery 70% 20'6" to 21'3" core recovery 35%
Grey badly weathered shale. Thin beds of limestone.				21'3"			21'3" to 23'3" core recovery 47%
Grey limestone. Thin layer of badly weathered shale.				23'3"			23'3" to 24'6" core recovery 65%
Grey argillaceous limestone exhibits fine bedding and some jointing.				24'6"			24'6" to 27'6" core recovery 61%
Dark grey shale, lower 4" badly weathered.				27'6"			27'6" to 28'3" core recovery 78%
Grey limestone with interbeds of buff dolomite with zones of small solution cavities. Some jointing parallel to core length. Sound rock.				28'3"	Free Water		28'3" to 33'0" core recovery 45%
				33'0"			Artesian pressure water rose to elevation 902.8' approximately.
		End of Borehole					

SCALE: 1" = 5'0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

FORM G-1A 500 UNITED STATIONERY CO.

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SOIL MECHANICS LABORATORY

BOREHOLE LOG

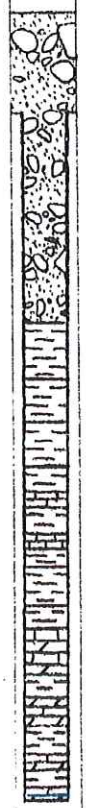
PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 12 DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

FORM G-1A 500



DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	890.8			Zero			BX Core Drilling
Sand, gravel and boulders.							1'4" to 3'4" core recovery 25%
do							3'4" to 9'4" core recovery 48%
do							9'4" to 11'7" core recovery 33%
Badly weathered grey shale.				10'4"			11'7" to 13'7" core recovery 57%
Grey calcareous shale rather broken.				12'1"			
Grey calcareous shale with badly weathered layers.				13'7"			
Grey calcareous shale with interbeds of limestone zones of small solution cavities.				15'0"			15'0" to 17'10" core recovery 66%
Badly weathered shale.				17'10"			
Grey calcareous shale.				18'6"			18'6" to 19'9" core recovery 45%
Badly weathered shale.				19'9"			
Buff dolomite with interbeds of calcareous shale zones of solution cavities. Some jointing parallel to core length. Sound rock.				20'6"			20'6" to 23'0" core recovery 83%
							23'0" to 26'2" core recovery 76%
				26'2"			Artesian pressure encountered in bedrock. Water rose to elevation 893.5' approximately
		End of Borehole					

SCALE: 1" = 5'0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

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SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 13 DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	891.17			Zero			BX Core Drilling
Sand, gravel and boulders.							
				11'2"			11'2" to 13'2" core recovery 33%
							13'2" to 14'0" core recovery 60%
Bedrock.							14'0" to 19'6" core recovery 30%
				19'6"			
		End of Borehole					

SCALE: 1" = 5'0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

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SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 13A DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	+891.2			Zero			BX Core Drilling
Same as BH. 13.							
Badly weathered shale with occasional layer of argillaceous limestone up to 2" thick.			● 1	9'0"		74	Damp.
			● 2			104	do
do			● 3			120	do
do			● 4			103(8")	do
do			● 5			120	do
Grey argillaceous limestone exhibits thin bedding.				14'6"			14'6" to 17'0" core recovery 70%
				16'6"			
Badly weathered grey shale.			○ 6	18'2"		39	Moist. 18'2" to 19'4" No core recovery
Grey calcareous shale. Jointing parallel to core length.							18'6" to 20'9" core recovery 59%
							20'9" to 22'9" core recovery 62%
Badly weathered shale.				22'6"			
Grey shale interbedded with anhydrite.				22'9"			22'9" to 25'9" core recovery 20%
Grey somewhat argillaceous limestone with interbeds of buff dolomite and zones of small solution cavities. Occasional joints parallel to core length. Sound rock.				26'9"			26'6" to 30'6" core recovery 81%
				30'6"			Artesian pressure encountered in bedrock. Water rose to elevation 893.5' approximately
		End of Borehole					

SCALE: 1" = 5'0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

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SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 16 DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	891.4			Zero			BX Core Drilling
Brown sand, gravel and boulders generally subrounded.							1'0" to 5'0" Wash Sample
Sand, gravel and boulders.							5'0" to 10'0" Wash Sample
do							10'0" to 14'6" Wash Sample
Fine to coarse SAND.				14'6"			
Fragments of grey limestone.				15'0"			15'0" to 16'0" Wash Sample
Grey limestone with interbeds of calcareous shale. Small solution cavities. Jointing parallel to core length.				16'0"			16'0" to 19'0" core recovery 53%
Badly weathered shale.				19'3"			
Grey argillaceous limestone with joints parallel to 15° to core length.				19'9"			19'9" to 25'6" core recovery 30%
Grey limestone with joints parallel to core length, and some solution cavities.							25'6" to 29'0" core recovery 37%
				29'0"			Artesian pressure encountered in bedrock. Water rose to elevation 892.8' approximately.
		End of Borehole					

SCALE: 1" = 5'0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

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BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 17 DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING -

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Brown sandy loam with gravel and organic matter.	900.57			Zero			
Loose dark brown sandy loam with some organic matter.			• 1	1'6"		4	Damp. Low dry strength.
Firm brown SAND with fine to medium generally subrounded gravel.			• 2	4'0"		28	Moist. Low dry strength.
Firm to dense fine to coarse SAND and fine to large GRAVEL. Little silt and clay. Gravel generally subrounded.			• 3			54	Wet. High N due to large gravel.
Firm to dense SAND and fine to large GRAVEL subangular to subrounded.			• 4			58	do
Firm to dense fine to coarse SAND and fine to large GRAVEL. Little silt and clay. Gravel subangular to subrounded.			• 5			58	do
do			• 6	24'3"		70	Moist. Medium dry strength
Hard grey clay with fragments of shale. Probably badly weathered shale.				27'0"			
Weathered grey shale.				27'6"			27'6" to 28'7" core recovery 85%
Grey somewhat calcareous laminated shale.				28'7"			28'7" to 31'0" core recovery 77%
Grey argillaceous limestone. Some solution cavities. Jointing parallel to core length.				31'0"			31'0" to 35'5" core recovery 82%
Grey to dark grey somewhat calcareous shale, exhibits lamination. Jointing parallel to core length.				35'5"			35'5" to 40'3" core recovery 52%
Grey argillaceous limestone with thin layers of shale. Some solution cavities. Joints parallel to core length.				40'3"			40'3" to 45'0" core recovery 45%
Grey to dark grey laminated shale. Occasional joints parallel to core length.				45'0"			
		End of Borehole					

SCALE: 1" = 5'0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

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SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways, Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 18 DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Brown loam with organic matter.	900.27			Zero 0'6"			
Loose brown silty SAND with some organic matter.			• 1			4	Damp. Low dry strength.
				5'0"			
Brown fine to coarse SAND and GRAVEL, some boulders. Gravel generally subrounded.				Free Water			
Firm to dense fine to coarse SAND with fine to large gravel. Gravel generally subrounded.			• 2			60(3")	Wet.
Sand, gravel and boulders. Little clay.							10'6" to 15'0" Wash Sample
do							15'0" to 19'0" Wash Sample
				19'0"			
Hard grey clay with fragments of calcareous shale. Badly weathered shale.			• 3			60(4")	Damp. High dry strength.
Grey argillaceous limestone with small solution cavities. Some jointing.				25'4"			25'4" to 27'4" core recovery 92%
Grey argillaceous limestone interbedded with dark grey laminated shale. Some jointing parallel to core length.				27'6"			27'4" to 31'4" core recovery 82%
do							31'4" to 35'2" core recovery 90%
				35'2"			
		End of Borehole					

SCALE: 1" = 5'0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

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SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 19 DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Brown sandy loam with organic matter.	902.13			Zero			
Brown silty sand with traces of organic matter.				1'6"			BX Core Drilling
Sand, gravel and boulders.							Wash Sample
				Free Water			6'3" to 7'3" core recovery 30%
do							10'0" to 12'0" Wash Sample
Sand, gravel and boulders with some clay.							15'0" to 17'0" Wash Sample
Brown sand and fine gravel.							18'0" to 19'0" Wash Sample
Grey clay with fragments of rock. Badly weathered shale.				25'0"			25'0" to 26'6" core recovery 22%
Grey limestone, extensive small solution cavities from 26'9" to 27'3". Occasional jointing.				26'6"			26'6" to 28'4" core recovery 90%
Interbedded grey argillaceous limestone and dark grey laminated shale.				28'4"			28'6" to 32'0" core recovery 56%
Grey sometimes argillaceous limestone with some irregular jointing and occasional small solution cavities.				32'0"			32'0" to 42'9" core recovery 50%
Interbedded grey argillaceous limestone and dark grey shale. Some jointing at 45° to core length. Sound rock.				42'9"			42'9" to 45'0" core recovery 75%
				45'0"			
		End of Borehole					

SCALE: 1" = 5'0" • DISTURBED SAMPLE ▌UNDISTURBED SAMPLE

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SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

BOREHOLE NO. BH. 20 DIAMETER 2-1/2" CASING BX & AX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

FORM G-1A 500

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	901.03			Zero			
Brown loam with organic matter.				1'0"			
Brown silty SAND. Traces of organic matter.							
							BX Core Drilling
Sand, gravel and boulders.				Free Water			7'0" to 10'0" Wash Sample
Brown very sandy CLAY with fine to large subangular gravel.							10'0" to 14'0" Wash Sample
			● 1			69	Damp. High dry strength. High N due to gravel.
do							19'0" to 21'7" core recovery 4"
Fragments of limestone and calcareous shale. Traces of grey clay.							21'7" to 23'7" core recovery 33%
Fragments of grey limestone and dark grey shale, probably badly weathered bedrock.				27'0"			27'0" to 28'6" Wash Sample
				28'6"			28'6" to 32'0" No core recovery
Fragments of limestone, calcareous shale, some clay, probably badly weathered rock.							32'0" to 35'6" core recovery 33%
Grey argillaceous limestone with solution cavities and some jointing parallel to core length.				36'0"			36'0" to 39'3" core recovery 60%
Light grey limestone containing numerous layers of anhydrite.				39'3"			39'3" to 47'0" core recovery 11%
				47'0"			
		End of Borehole					

SCALE: 1" = 5'0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Waterloo Township Bridge No. 12 W.P. 155-58 ORDER NO. T.340/58

CLIENT Department of Highways, Ontario (Consulting Engineers, Messrs. Laughlin, Wyllie & Ufnal)

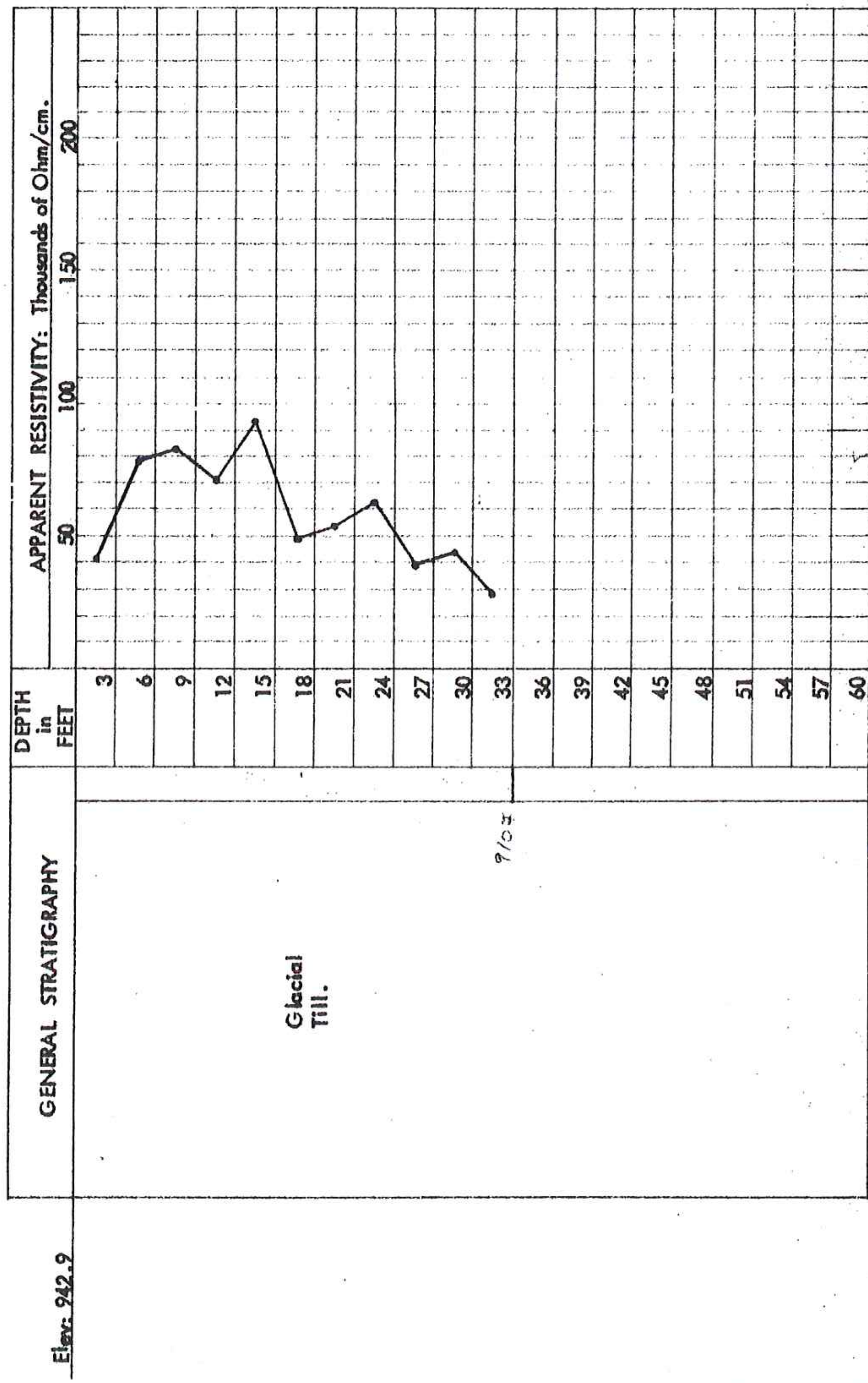
BOREHOLE NO. BH. 21 DIAMETER 2-1/2" CASING BX

BOREHOLE LOCATION See Sketch INCLINATION Vertical BEARING ---

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Brown sandy loam with organic matter.	± 982.0			Zero			
Brown SAND with some gravel.				1'0"			
Firm brown generally fine SAND with occasional subrounded gravel.			● 1	2'0"		12	Damp. No dry strength.
do			● 2			21	do
Dense brown fine to coarse SAND with fine gravel subangular to subrounded.			● 3			53	Moist. No dry strength.
							BX Core Drilling
do			● 4			52	do
Very stiff to hard sandy CLAY with fine to medium subangular gravel.			● 5	16'0"	966.0	15(6")	Damp. High dry strength.
Hard brown sandy gravelly CLAY. Gravel subangular, fine to large. Some boulders.			● 6			96(7")	do
			● 7			72	do
do			● 8			88	do
Dense brown fine to coarse SAND with fine to large subangular to subrounded gravel.			● 9	28'6"		48	Damp. No dry strength.
Hard brown very sandy CLAY with gravel.				31'0"			
Very dense brown clayey gravelly SAND. Gravel subangular to subrounded.			● 10	32'6" 34'6"		102	Damp. Medium to high dry strength.
		End of Borehole					

SCALE: 1" = 5'0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

FORM G-3 500
UNITED STATIONERY CO. Limited



PROJECT Waterloo Township Bridge No. 12

TITLE Electrical Resistivity Sounding: RS. 8

DRG. NO. 11 ORDER NO. T.340/58

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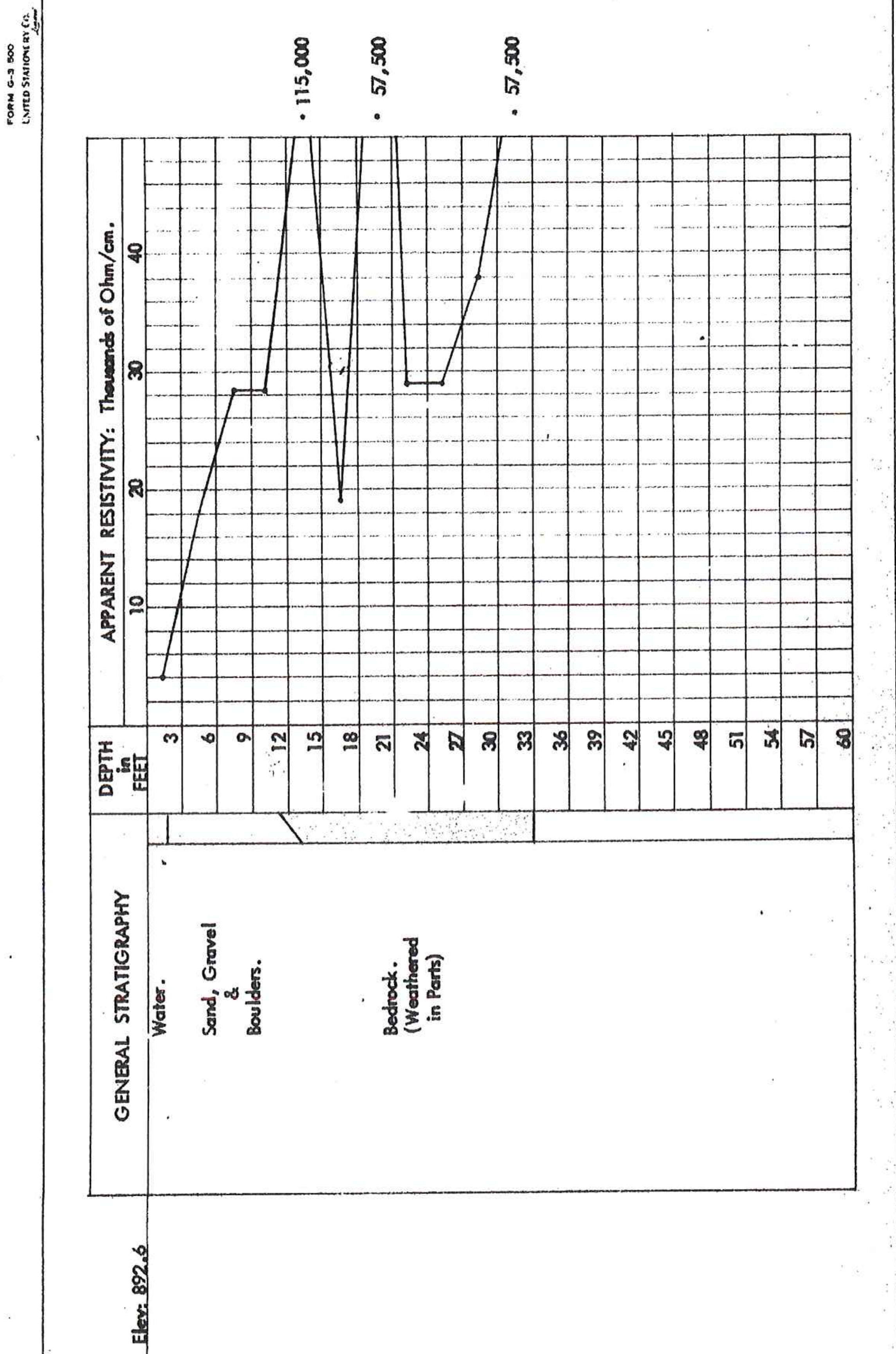

PROJECT Waterloo Township Bridge No. 12

TITLE Electrical Resistivity Sounding: RS. 11

DRG. NO. 12 ORDER NO. T.340/58

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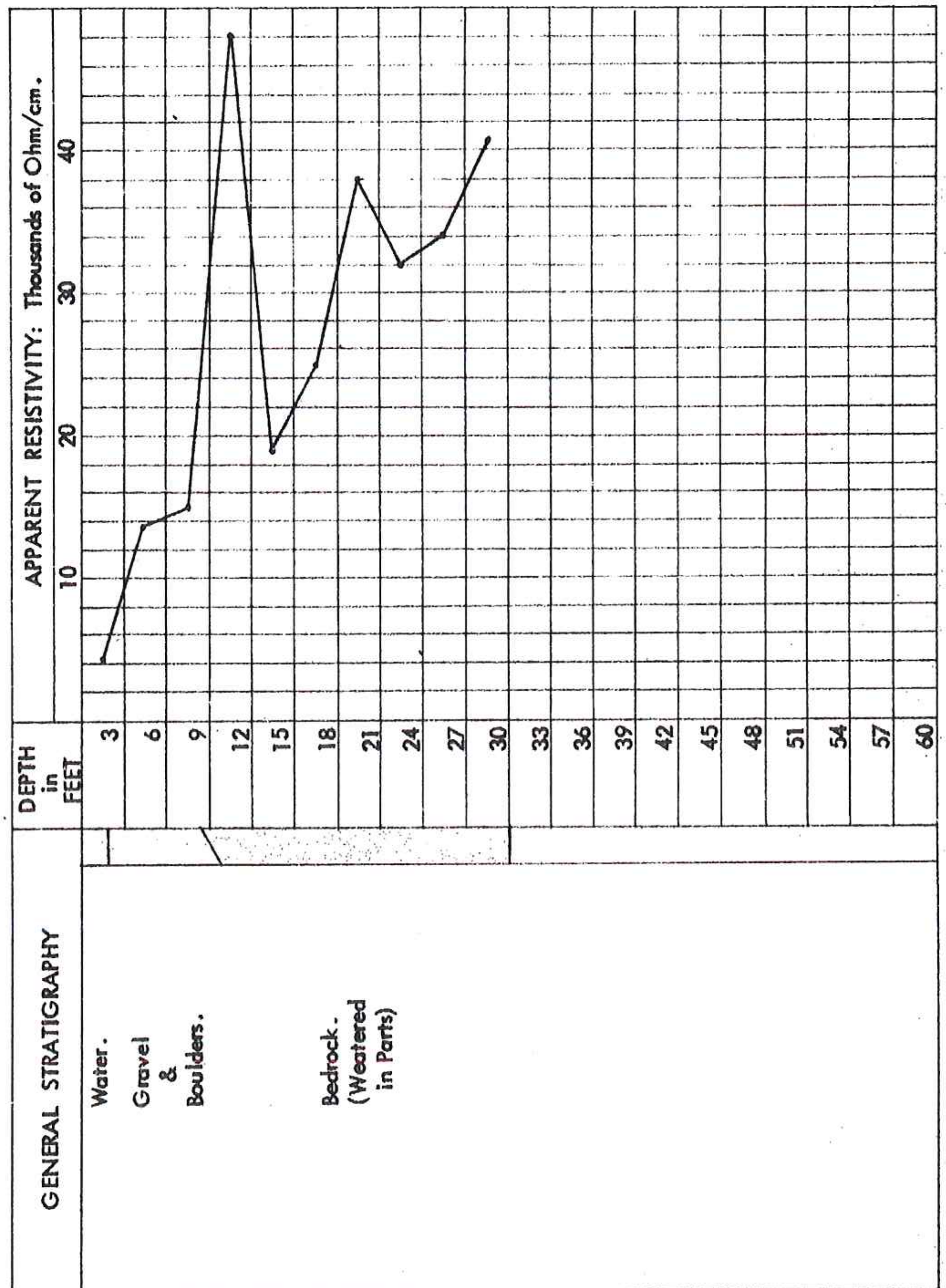
FORM G-3 500
UNITED STATIONERY Co. Limited



Elev: 892.6

PROJECT Waterloo Township Bridge No. 12

TITLE Electrical Resistivity Sounding: RS. 12

DRG. NO. 13 ORDER NO. T.340/58

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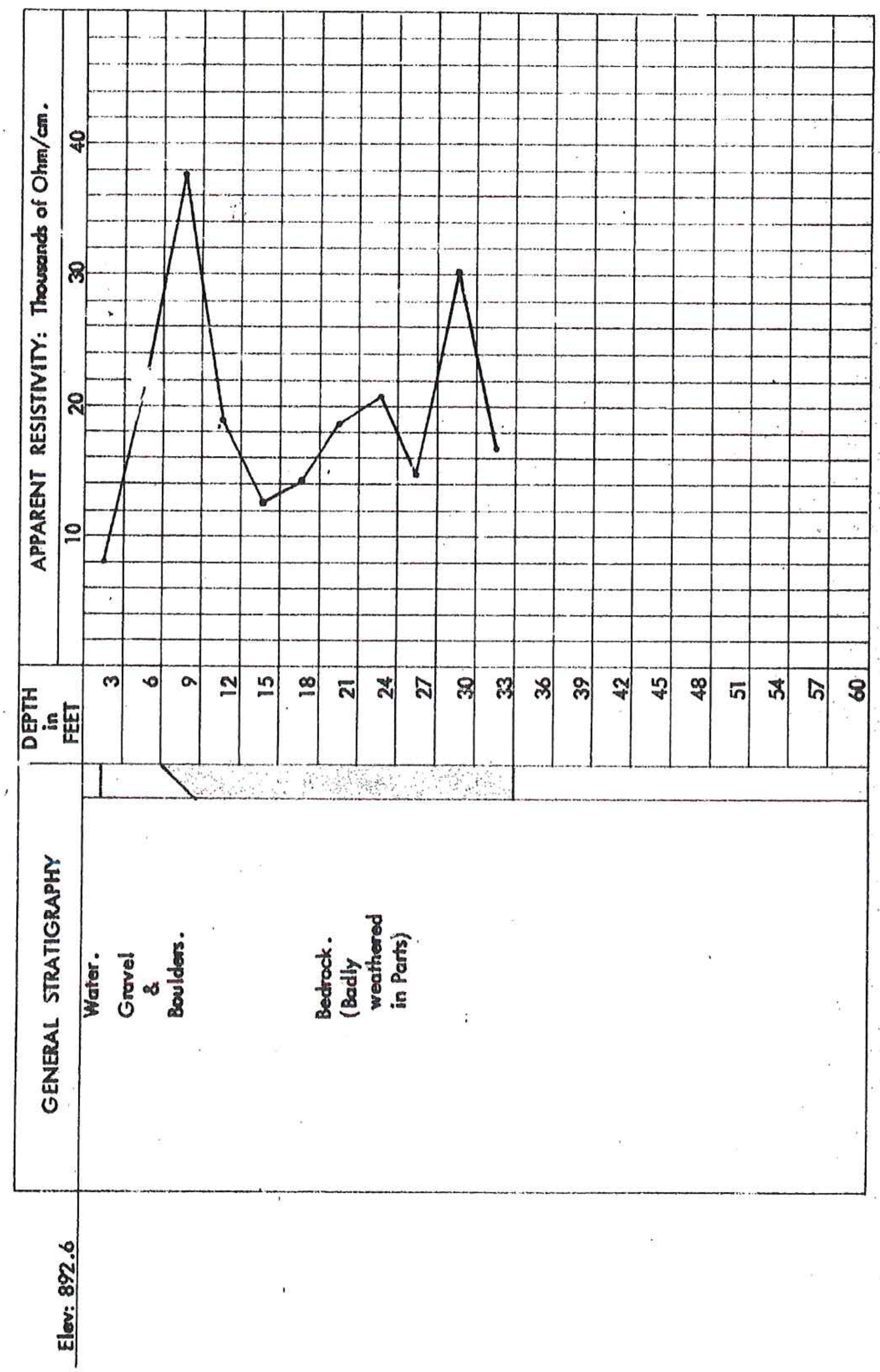
GEOTECHNIQUE
LIMITED

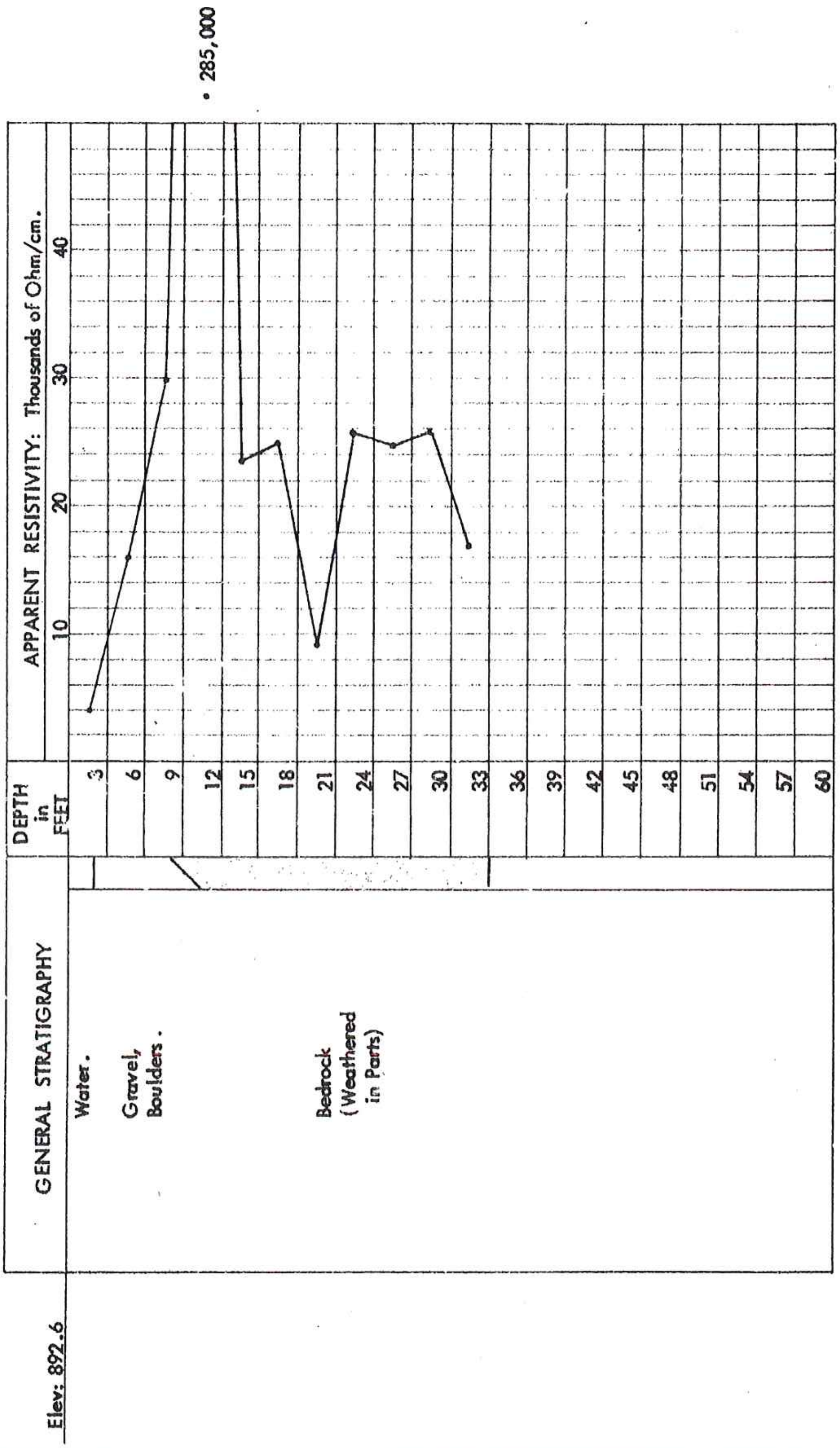


PROJECT Waterloo Township Bridge No. 12

TITLE Electrical Resistivity Sounding: RS. 13

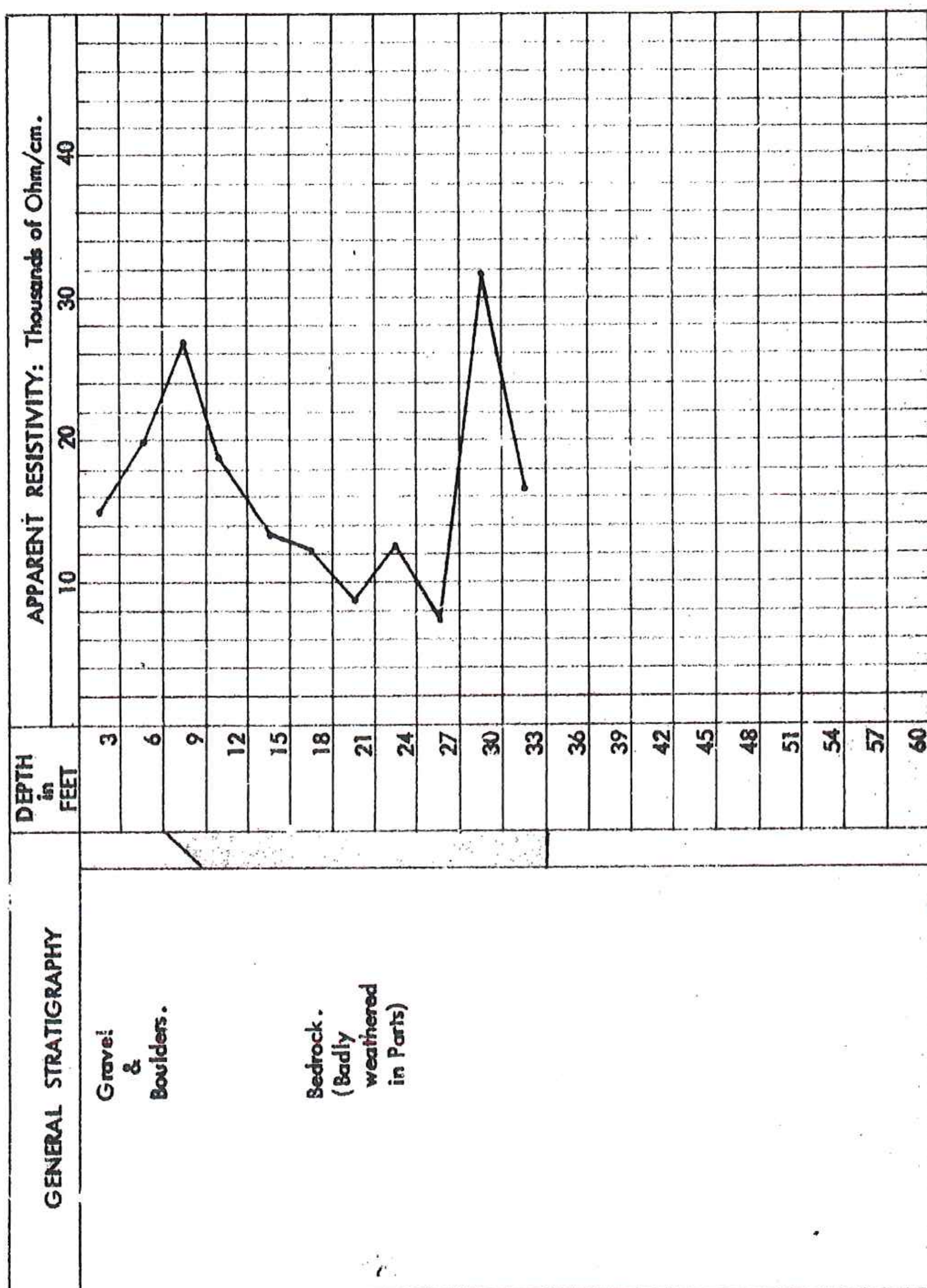
DRG. NO. 14 ORDER NO. T.340/58

UNIVERSAL
GEOTECHNIQUE
LIMITED



PROJECT Waterloo Township Bridge No. 12	UNIVERSAL
TITLE Electrical Resistivity Sounding: RS. 14	GEOTECHNIQUE
DRG. NO. 15 ORDER NO. T.340/58	LIMITED

FORM G-3 500
UNITED STATIONERY



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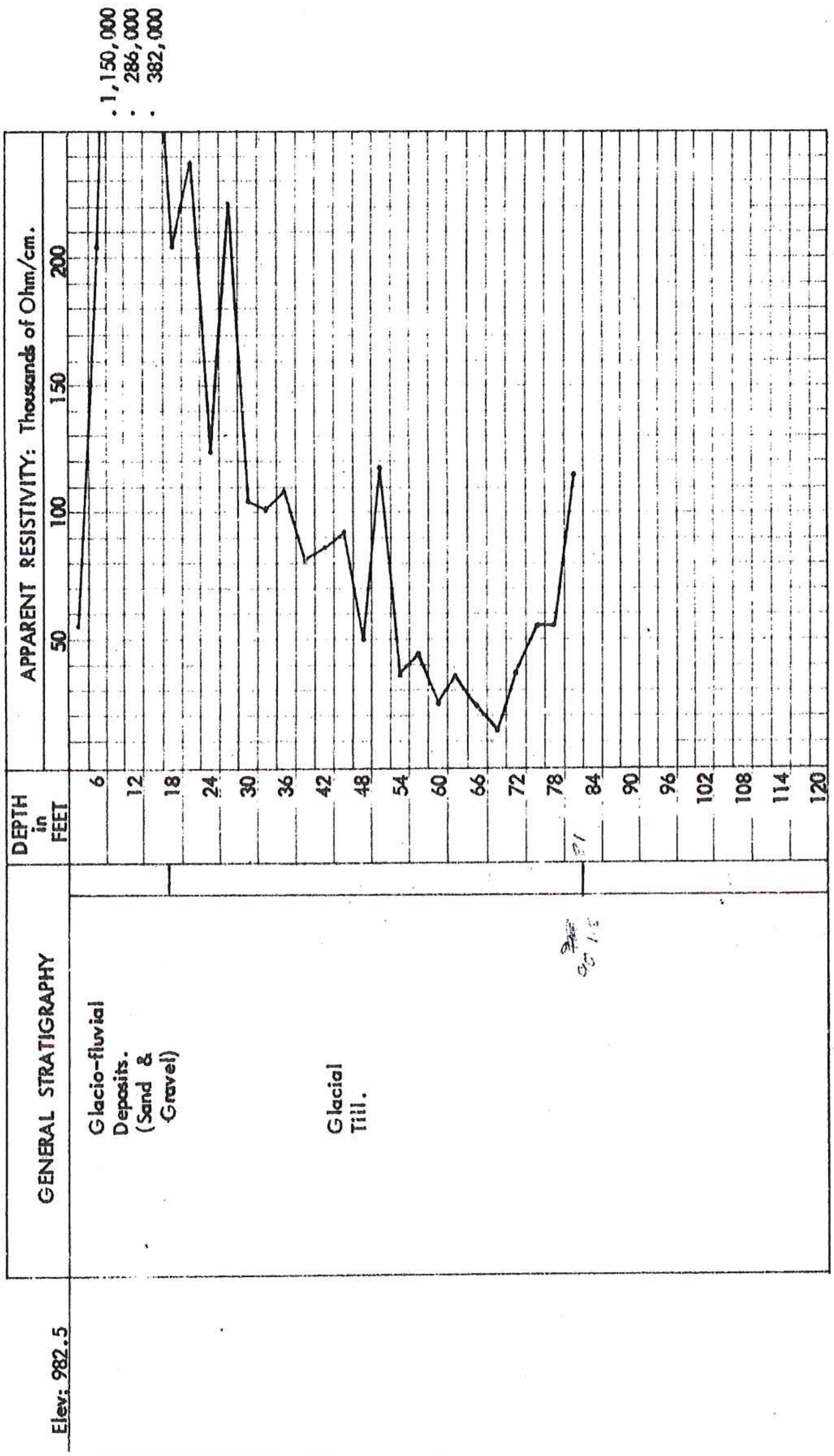
PROJECT Waterloo Township Bridge No. 12

TITLE Electrical Resistivity Sounding: RS. 15

DRG. NO. 16 ORDER NO. T.340/59

UNIVERSAL
GEOTECHNIQUE
LIMITED

FORM G-3 500 UNITED STATIONERY CO.



PROJECT Waterloo Township Bridge No. 12

TITLE Electrical Resistivity Sounding: RS. 21

DRG. NO. 17 ORDER NO. T.340/58

UNIVERSAL
GEOTECHNIQUE
LIMITED

THURBER

Appendix C

Foundation Comparison

THURBER

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Bedrock	Driven Piles	Socketed H-Piles
Advantages: i. Relative ease of construction. ii. Shallower excavation depth than footings on bedrock. iii. High resistance values are available at shallow depth at east abutment. iv. Lower cost than deep foundations. ***Disadvantages:*** i. Potential large variation in support capability in the valley base deposits. ii. Excavation and cofferdam installation required for footing construction in river valley. iii. Organic and soft soils may extend deeper than anticipated. iv. Potential for undermining by river scour v. Unexpected high river levels may delay construction. **FEASIBLE AT EAST ABUTMENT**	***Advantages:*** i. Relatively high resistance values for footing design. ii. Bedrock surface can be examined to confirm degree of weathering and soundness. iii. Existing bridge is supported on footings on rock, confirming feasibility of system. ***Disadvantages:*** i. Relatively deep excavation below river water levels, requiring cofferdam installation and significant dewatering. ii. Poor bedrock quality limits design resistance available. iii. Potential for disturbing existing footings. iv. Potential for river scour. **FEASIBLE AT PIERS & WEST ABUTMENT**	***Advantages:*** i. Piles will develop high geotechnical resistance when driven into bedrock or very dense soil. ii. Pile installation may continue in freezing weather. iii. May require less excavation than footing construction. ***Disadvantages:*** i. Higher unit cost than footings. ii. Shallow depth to bedrock in the valley base. iii. Difficulty penetrating very dense soil, boulders and rock fragments. iv. Pre-augering may be required. v. Piles may encounter refusal on strong rock layer over weaker weathered layer. vi. Potential for pile damage while driving. vii. Excavation required for pile cap construction. **NOT RECOMMENDED**	***Advantages:*** i. High resistance is available for piles socketed into bedrock. ii. Length of pile and socket can be controlled. iii. Avoids pile damage during installation. iv. Construction could continue in freezing weather. v. May require less excavation than footing construction. ***Disadvantages:*** i. Higher cost than footings. ii. Difficulty penetrating very dense soil, boulders, rock fragments, and highly weathered bedrock. iii. Temporary steel liners may be required to support side walls above bedrock surface. iv. Grouting of socket around pile is required. v. Difficulty in cleaning and inspecting bases. **FEASIBLE**

COMPARISON OF FOUNDATION ALTERNATIVES (cont'd)

Drilled-in Pipe Piles	Caissons	Micropiles
Advantages: i. High resistance is available for piles drilled into bedrock. ii. Avoids potential for meeting refusal on boulders and rock fragments above design tip level. iii. No need for temporary liner. iv. Excavation for pile cap construction below river level could be avoided. v. Construction could continue in freezing weather. ***Disadvantages:*** i. High cost. ii. Relatively deep socket required to contact sound bedrock. iii. Advancing through boulders, rock fragments and highly weathered bedrock may slow production. iv. Limited number of contractors with suitable equipment. **FEASIBLE**	***Advantages:*** i. High resistance is available for caissons socketed into rock. ii. May require less excavation than footing construction. iii. Construction could continue in freezing weather. ***Disadvantages:*** i. Higher cost than driven piles. ii. Difficulty penetrating very dense soil, boulders, rock fragments, and highly weathered bedrock. iii. Temporary steel liners may be required to support side walls above bedrock surface. iv. Tremie concrete methods may be required. v. Difficulty in cleaning and inspecting bases. **FEASIBLE**	***Advantages:*** i. Lower unit cost than steel piles and caissons. ii. Axial pile resistance may be increased through load tests. iii. May more readily penetrate the boulders and rock fragments than larger diameter foundation types. iv. Pile installation may continue in freezing weather. v. May require less excavation than footing construction. ***Disadvantages:*** i. Very low lateral resistance available. ii. Prohibitively large number of piles may be required to resist foundation loads. iii. Little proven experience supporting large highway bridges on micropiles. iv. Potential for losing grout into the river through bedrock voids. **NOT RECOMMENDED**

THURBER

Appendix D

Borehole Locations and Soil Strata Drawing

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

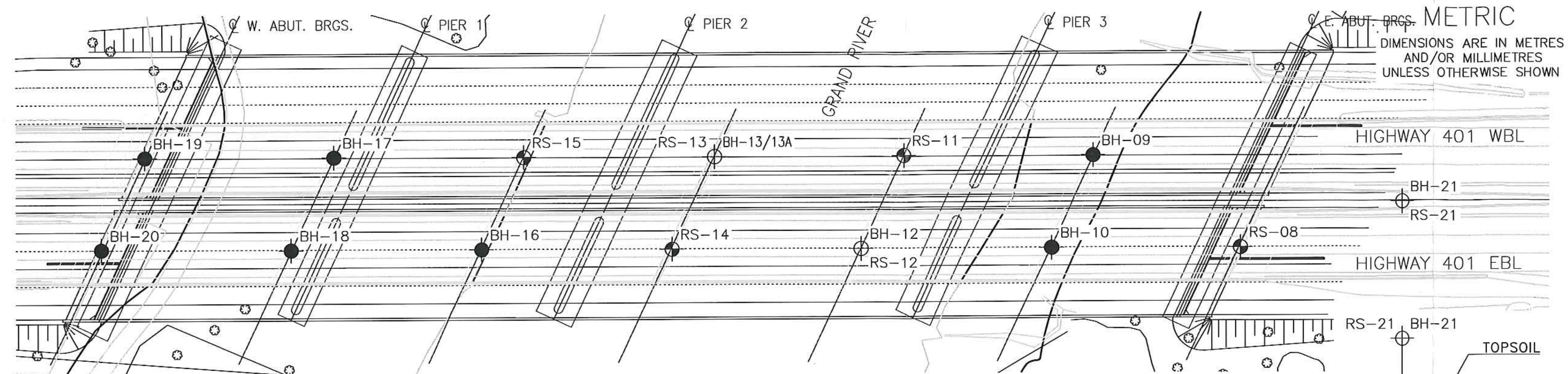
CONT No
WP No

HIGHWAY 401
WESTBOUND LANES
OVER GRAND RIVER
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

wsp

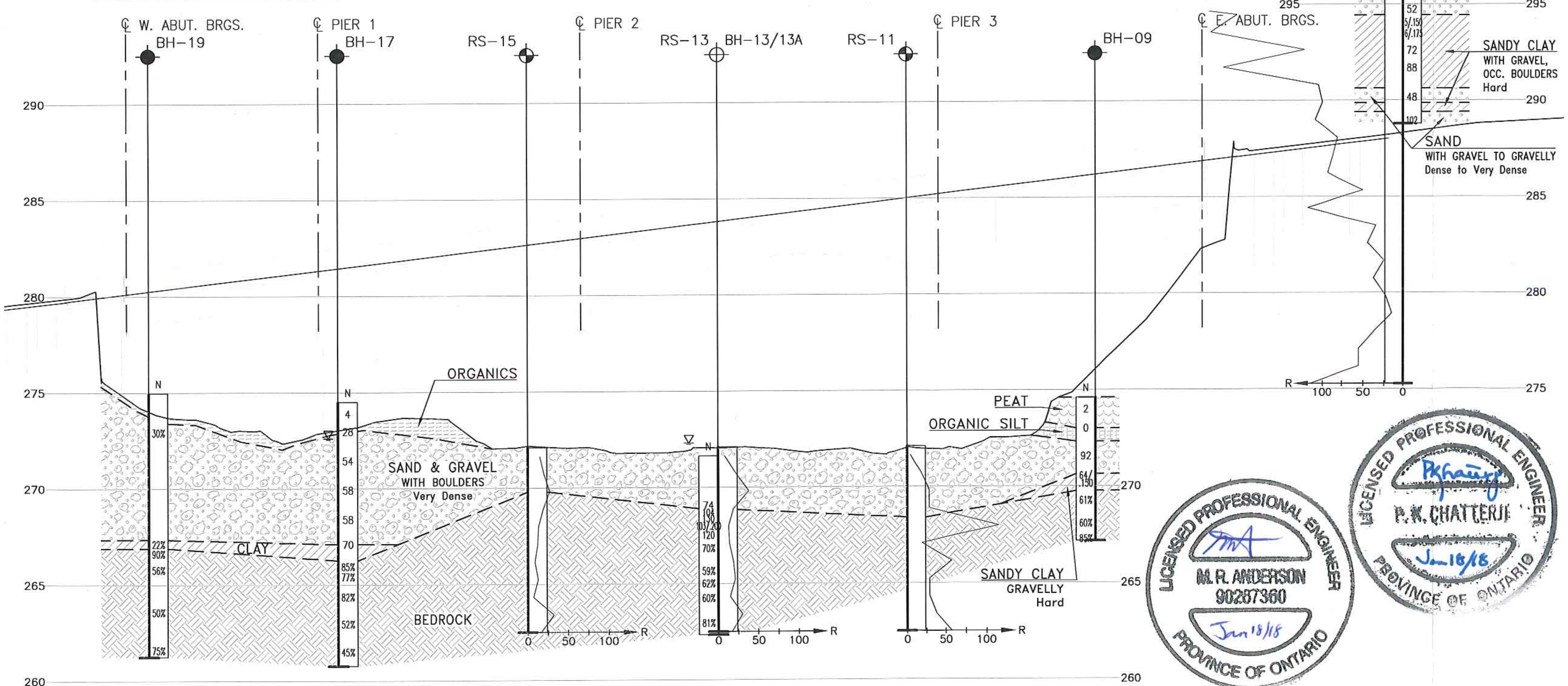
THURBER ENGINEERING LTD.



NO	ELEVATION	NORTHING	EASTING
RS-13	272.1	4 806 839.8	233 013.1
RS-14	272.1	4 806 819.1	233 017.5
RS-15	272.1	4 806 816.2	232 981.3
RS-21	299.5	4 806 917.5	233 134.2

PLAN

SCALE 1:1000



NOTE: All borehole information from GEOCRES No. 40P08-023

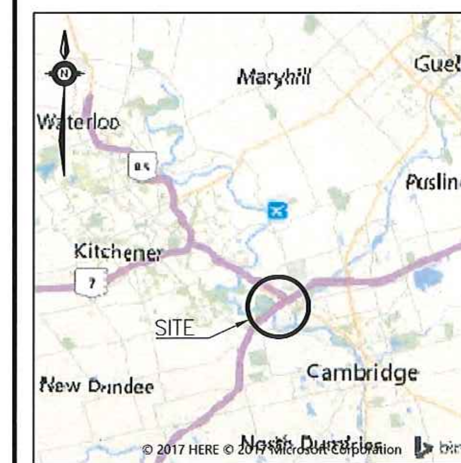
PROFILE ALONG ℄ HIGHWAY 401 WBL

H 1:1000

V 1:250

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
90287360
Jan 18/18
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jan 18/18
PROVINCE OF ONTARIO



KEYPLAN

LEGEND

	Borehole / Resistivity Sounding
	Borehole and Resistivity Sounding
N	Blows /0.3m (Std Pen Test, 475J/blow)
R	Resistivity (Ohm/cm)
PH	Pressure, Hydraulic
	Water Level During Drilling
	Head Artesian Water
	Piezometer
90%	Core Recovery
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
BH-09	274.6	4 806 887.0	233 076.5
BH-10	274.8	4 806 866.3	233 081.0
BH-12	271.5	4 806 842.6	233 049.1
BH-13/13A	271.6	4 806 839.8	233 013.1
BH-16	271.7	4 806 795.5	232 985.6
BH-17	274.5	4 806 792.5	232 949.3
BH-18	274.4	4 806 771.6	232 953.6
BH-19	275.0	4 806 769.0	232 917.6
BH-20	274.6	4 806 748.2	232 921.7
BH-21	299.3	4 806 917.5	233 134.2
RS-08	287.4	4 806 889.9	233 112.7
RS-11	272.1	4 806 863.6	233 044.8
RS-12	272.1	4 806 842.6	233 049.1

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 40P8-246

REVISIONS | DATE | BY | DESCRIPTION

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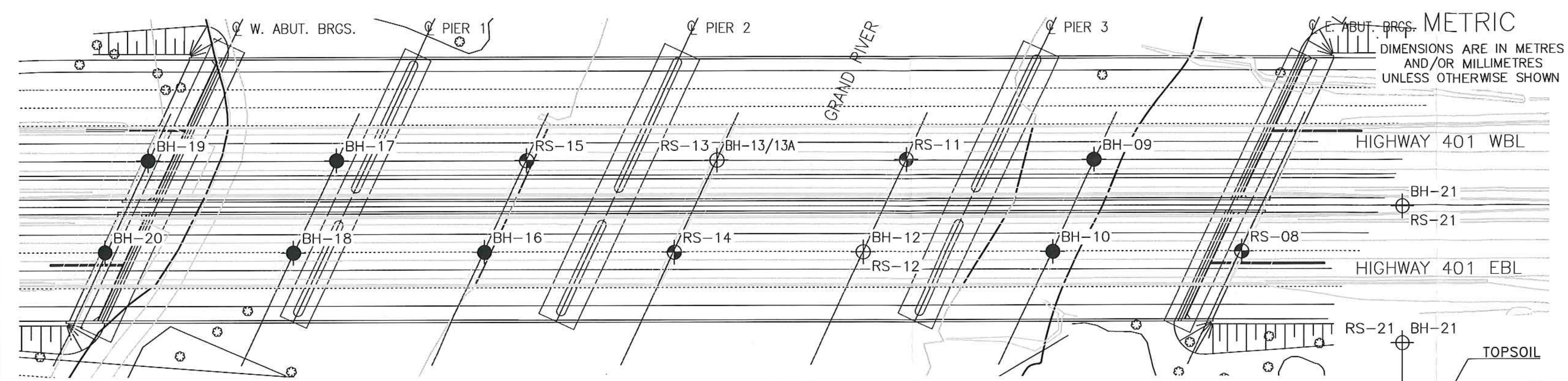
MINISTRY OF TRANSPORTATION, ONTARIO

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



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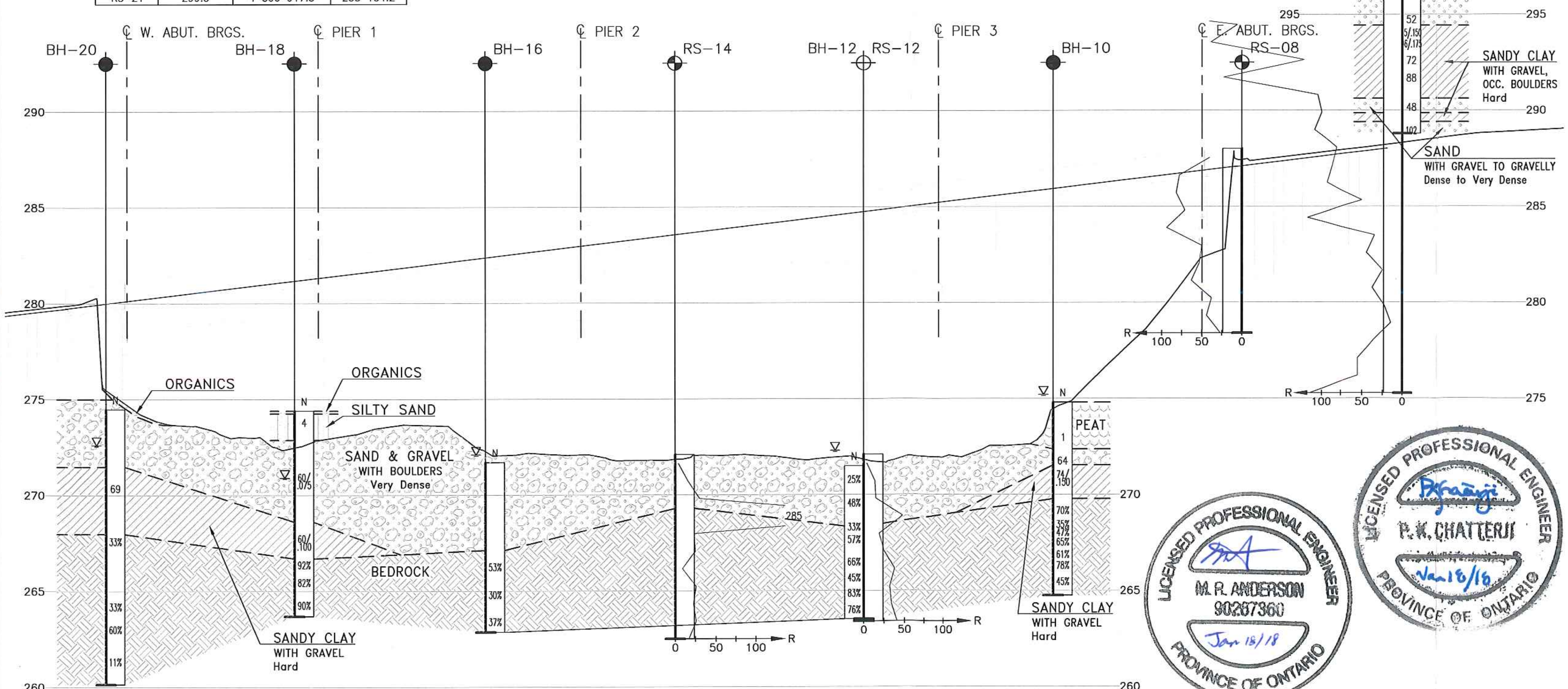
RS-15	272.1	4 806 816.2	232 981.3
RS-21	299.5	4 806 917.5	233 134.2

PLAN

20 0 20 40m
SCALE 1:1000



NOTE: All borehole information from GEOCRES No. 40P08-023

PROFILE ALONG ℄ HIGHWAY 401 EBL

20 0 20 40m H 1:1000
5 0 5 10m V 1:250

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
90287360
Jan 18/18
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
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PROVINCE OF ONTARIO

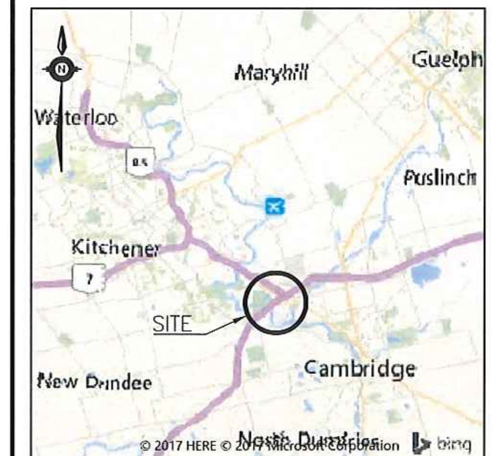
CONT No
WP No

SHEET

HIGHWAY 401
EASTBOUND LANES
OVER GRAND RIVER
BOREHOLE LOCATIONS AND SOIL STRATA

wsp

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

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GEOCRES No. 40P8-246

REVISIONS

DATE	BY	DESCRIPTION

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