



August 29, 2018

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Highway 406 S - Geneva Street N/S Ramp
Bridge, Structure Site 18-168
Highway 406 Structural Rehabilitation from
Fourth Avenue to Westchester Avenue,
St. Catharines, Ontario
G.W.P. 2453-13-00**

Submitted to:
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REPORT



GEOCRES: 30M3-299

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**FOUNDATION DESIGN REPORT - HWY 406 S - GENEVA STREET
N/S RAMP BRIDGE, G.W.P. 2453-13-00
HIGHWAY 406, ST. CATHARINES ONTARIO.**

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APPENDIX B

Drawing D-5147-2 General Plan

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PART A

**FOUNDATION INVESTIGATION REPORT
HIGHWAY 406 S - GENEVA STREET N/S RAMP BRIDGE
STRUCTURE SITE NO. 18-168
HIGHWAY 406 STRUCTURAL REHABILITATION
FOURTH AVENUE TO WESTCHESTER AVENUE
ST. CATHARINES, ONTARIO
G.W.P. 2453-13-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the future rehabilitation of nine structures along Highway 406 from Fourth Avenue to Westchester Avenue in the City of St. Catharines, Ontario.

This report presents the geotechnical site conditions at the site of the existing Highway 406 S – Geneva Street N/S Ramp structure. It was developed with information from a foundation investigation completed in 1962 at the bridge site, reported as follows:

- **MTO GEOCREs No. 30M03-43:** *Highway #58 and Geneva Street, Access Ramp at Old Welland Canal, City of St. Catharines, Dist. #4, dated July 12, 1962.*
- **MTO GEOCREs No. 30M03-43:** *Additional Borings, Hwy. 406 and Geneva Street, Access Ramp at Old Welland Canal, City of St. Catharines, Dist. #4, W.P.126-58-1, W.J. 62-F-62, dated October 19, 1962.*

The Terms of Reference and Scope of Work for the foundation engineering services are outlined in MTO's Request for Proposal, dated September 2015 that form part of the Consultant's Agreement (Number 2014-E-0075) for this project. The Scope of Work for the Geneva Street N/S Ramp bridge foundation assessment is comprised of a Desktop Study as reported herein. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for this project, dated June 2016.

2.0 SITE DESCRIPTION

The Highway 406 S - Geneva Street (N-S) Ramp structure is located north of the Highway 406 underpass structure at Westchester Avenue and connects Highway 406 northbound vehicle traffic to Geneva Street, near downtown St. Catharines (see Key Plan on Drawing 1). The structure spans the broad gully that was part of the old second Welland Canal (now called the Canal Valley; adjacent to Twelve Mile Creek), of which the crest-to-crest width is about 210 m. The General Plan of the site available in GEOCREs 30M03-43 (Drawing D 5147-2, dated May 1963) show topographic contour lines indicating that the crest of the valley bank was about 8.3 m above the ground surface of the base of the valley. The General Plan drawing shows the location of the original Old Welland Canal and indicates that a culvert was constructed to the east of the Old Welland Canal. It is understood that old Welland Canal discharged to Twelve Mile Creek approximately 680 m to the west of the Ramp Bridge through a three-cell buried structural culvert that was constructed between Piers 1 and 2 of the Ramp Bridge. The General Plan further indicates that Old Welland Canal was filled-in to Elevation 87.5 m, and that in the vicinity of Pier 4 and between Pier 1 and the east abutment the "gravely clay fill, miscellaneous fill, ash and rubble fill" was to be subexcavated to "firm strata" and the subexcavation replaced with "select earth fill".

The existing Ramp structure is a five-span bridge that was constructed in about 1964, and has a total length of approximately 114 m. The current grade of the Ramp is at about Elevation 97 m near the east abutment and the rises to about Elevation 102 m at the west abutment.

Drawing No. D 5147-3 titled "Foundation Layout" indicates that the abutments and piers are supported on pile caps founded on 14BP73 steel H-piles (equivalent to HP360x108), driven into a till stratum underlying the silty clay deposit, to practical refusal as determined by the Hiley Formula (D.H.O. Std. BD 16-3,4).



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Golder visited the site on December 2016, to perform a cursory observations of the structure from a foundations perspective. There were no obvious visual signs of foundation-related issues in terms of foundation and embankment instability or settlement at that time.

3.0 PREVIOUS INVESTIGATION

The foundation investigations for the GEOCRETS reports referenced in Section 1.0 were conducted between May and October, 1962. During this periods, a drill rig advanced a total of twenty one boreholes using wash-boring techniques, with the exception of Borehole 12 which is indicated as an “augerhole” on the borehole record. The boreholes were advanced to depths between about 2.3 m and 20.3 m below the ground surface. Boreholes 1 and 21 are both located at distant offsets to the south of the structure and are not relevant to the present desk top study nor shown on the Borehole Locations and Soil Strata Drawing.

The GEOCRETS foundation investigation report indicates that soil samples were obtained at 0.75 m to 3 m depth intervals using 50 mm outside diameter split-spoon samplers driven by manual hammers, in accordance with the Standard Penetration Test (SPT) procedure. In the soft to stiff cohesive deposit, thin-walled Shelby tube samples were also taken and in situ field vane testing was conducted to measure the undrained shear strength of the deposit. Dynamic Cone Penetration Testing (DCPT) was conducted from the ground surface in the immediate vicinity of Boreholes 1 to 4 and 6 to 11.

Observations of the water levels in the boreholes were recorded on some boreholes logs; however, piezometers were not installed in any of the boreholes.

Selected samples obtained from the boreholes were subjected to classification testing and the results are resented the Record of Borehole sheets in Appendix A.

The boreholes locations as provided on the Record of Borehole sheets in Station and Off-set were plotted on the General Arrangement Drawing No. R2-1, dated Nov. 2016, provided by MTO on January 31, 2017, and the borehole coordinates were interpreted from the coordinate system superimposed on the plan. The borehole locations in MTM NAD 83 (Zone 10) coordinates, ground surface elevations referenced to Geodetic datum and the drilled depths are as follows:

Borehole No.	MTM NAD83 Northing (m)	MTM NAD 83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m) DCPT Depth (m)
2	4,779,902.1	326,137.8	88.1	10.1 (9.4)
3	4,779,872.8	326,101.6	88.1	14.2 (9.1)
4	4,779,919.8	326,065.0	90.5	15.4 (9.0)
5	4,779,944.1	326,041.6	96.3	20.3
6	4,779,894.3	326,083.7	87.3	20.3 (10.5)
7	4,779,859.3	326,118.1	88.8	15.7 (11.1)
8	4,779,866.7	326,140.7	89.0	15.2 (8.8)
9	4,779,844.5	326,163.9	89.7	15.5 (8.6)
10	4,779,855.2	326,189.6	90.2	15.9 (4.9)
11	4,779,908.1	326,068.2	90.8	18.4 (7.9)
12	4,779,860.7	326,155.0	89.5	15.7



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Borehole No.	MTM NAD83 Northing (m)	MTM NAD 83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m) DCPT Depth (m)
13	4,779,850.7	326,140.0	89.3	4.6
14	4,779,879.9	326,139.3	88.7	3.5
15	4,779,882.6	326,153.6	89.0	3.5
16	4,779,875.5	326,175.7	89.6	4.6
17	4,779,933.1	326,063.1	91.1	3.5
18	4,779,899.4	326,050.5	89.0	5.5
19	4,779,905.9	326,032.6	90.2	14.8
20	4,779,922.9	326,033.8	91.4	2.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 406 is located within the Iroquois Plains physiographic region, as delineated in the *Physiography of Southern Ontario* (Chapman and Putnam, 1984)¹. The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession. This site is bound to the north by shoreline beach deposits from Glacial lake Iroquois such as the Homer Bar on which downtown St Catharines is located, and the Niagara Escarpment located some 3 km to the south.

Surficial soil in this area of the Iroquois Plain is typically comprised of silty and clayey till of the Halton Till sheet according to the *Quaternary Geology of the Niagara-Welland Area* (Ontario Geological Survey Map 2496; Feenstra, 1984)². The Halton Till sheet is underlain by an older red sandy and silty till, possibly the Wentworth Till sheet (OGS Preliminary Map 764, Feenstra, 1972)³. Shallow depressions on the surface of the clay plain upslope of the Homer Bar are infilled with bog sediments while fill materials comprised of earth and rock fill associated with the canal construction occur in the vicinity of the former Welland Canal (OGS Preliminary Map 764, Feenstra 1972)³.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes in the vicinity of and pertinent to the Highway 406 S - Geneva Street (N/S) Ramp bridge structure (i.e. Boreholes 2 to 20, inclusive) and the results of in situ and geotechnical laboratory testing, where available from both of the 1962 reports, are given on the Record of Borehole sheets contained in Appendix A, following the text of this report. In the discussion below, the depth below ground surface is referenced to the ground surface on the borehole logs at the time of the investigation in 1962 and is not referenced to current ground surface.

¹ Chapman, L.J. and Putnam, D. F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

² Feenstra, B.H. 1984. Quaternary Geology of the Niagara-Welland Area. Ontario Geological Survey, Map 2496, Quaternary Geology Series. Scale 1:50,000

³ Feenstra, B.H. 1972. Quaternary Geology of the Niagara Area, Southern Ontario. Ontario Division of Mines, Preliminary Map P.764, Geological Survey. Scale 1:50,000



4.2.1 Topsoil and Fill

Fill was encountered at ground surface in all boreholes with the exception of Boreholes 5 and 20, which were located near the crest of the valley slope near the west abutment. Topsoil was encountered in Borehole 20 at ground surface and it extends to a depth of 0.3 m below ground surface.

The thicknesses of the fill ranges between about 0.6 and 3.4 m, and the deposit extends between about Elevations 90.5 m and 85.0 m.

The fill material is generally cohesive and is described as consisting of sand, clay and gravel, containing ash and/or organic matter. Layers of rubble and refuse were encountered in places, generally as the bottom layer of the fill deposit.

Standard Penetration Test (SPT) "N"-values in the fill deposit are variable, ranging between about 1 blow and 22 blows per 0.3 m of penetration, suggesting a soft to stiff consistency. One SPT "N"-value of 56 blows per 0.3 m of penetration measured in the fill material may be indicative of the split-spoon bouncing on rubble.

Atterberg limits tests were carried out on three (3) samples of the fill material, and measured plastic limits between about 17 per cent and 32 per cent, liquid limits between about 33 per cent and 60 per cent, and corresponding plasticity indices between about 16 per cent and 28 per cent. These test results indicates that this material a portion of the deposit is clayey silt of low plasticity and a portion of the deposit is clay of high plasticity.

The reported measured bulk weight of the fill material is about 16 kN/m³. The water content of the fill material ranges from about 22 per cent to 60 per cent. The organic content measured on a sample of the fill is 106 per cent.

4.2.2 Silty Clay to Clay Containing Organics to Organic Clay

Underlying the fill, a deposit of silty clay to clay containing organic matter described in places as organic clay was encountered in Boreholes 2, 3, 7, 8, 11 to 16 and 19) at depths between 2.3 m and 3.4 m below ground surface (between Elevations 87.9 m and 85.0 m) and extends to depths between 3.2 m and 5.5 m below ground surface (between Elevations 85.8 m and 83.2 m). The thickness of the silty clay to clay containing organics to organic clay ranges from about 0.3 m to 2.1 m.

The SPT "N"-values measured within the organic deposit range from 0 to 14 blows per 0.3 m of penetration. In situ field vane tests carried out within the deposit measured undrained shear strength ranging from 13 kPa to 77 kPa, with sensitivities ranging from 1.7 to 11.2. These field vane test results, together with the SPT "N"-values, indicate that the silty clay to clay containing organics to organic clay deposit has a soft to stiff consistency.

Atterberg limits tests were carried out on eight (8) samples of the silty clay to clay containing organics portion of the deposit, and measured plastic limits between about 22 per cent and 30 per cent, liquid limits between about 42 per cent and 58 per cent, and plasticity indices between about 17 per cent and 30 per cent. These test results indicate that this material is a silty clay of medium plasticity to clay of high plasticity, in places containing organic matters that classifies the material as organic silt of low to high plasticity.

The reported bulk weight of the silty clay to clay containing organics deposit ranges from about 16 kN/m³ to 19 kN/m³. The water content of the silty clay to clay containing organics deposit ranges from about 25 per cent to 62 per cent.



4.2.3 Clayey Silt to Clay

A deposit of clayey silt to clay was encountered at ground surface in Borehole 5 and underlying the fill or silty clay with organics in all of the remaining boreholes advanced for this site at depths ranging between about 0.6 m and 5.5 m below ground surface (between about Elevations 96.3 m and 84.8 m), and extends to between about Elevations 76.7 m and 69.2 m, where fully penetrated. The silty clay contains occasional seams of silt to silty sand, particularly in boreholes advanced near the valley floor. The deposit as shown in the Records of Borehole sheets is considered to be comprising of two layers / zones: an upper crust of higher consistency; and a lower layer of somewhat lower consistency, as described below.

Upper Crust – Cohesive Deposit

The upper crust of the cohesive deposit extends to between about Elevations 88.7 m to 82.3 m suggesting the thickness of the crust varies between 1.6 m at Borehole 4 and 7.6 m at Borehole 5. The measured SPT “N”-values of the upper crust range between 10 blows and 42 blows per 0.3 m of penetration. In situ field vane tests carried out within the upper crust of the cohesive deposit measured undrained shear strength ranging from 53 kPa to 143 kPa, with sensitivities ranging from 1.7 to 11.2. The field vane test results together with the SPT “N”-values indicate that the upper weathered crust of the cohesive deposit has a stiff to hard consistency.

Atterberg limits tests were carried out on ten (10) samples of the upper crust, and measured plastic limits between about 19 per cent and 23 per cent, liquid limits between about 42 per cent and 58 per cent and plasticity indices between about 23 per cent and 32 per cent. These test results indicate that this material is predominantly a silty clay of medium plasticity to clay of high plasticity. One Atterberg limits test result of a sample from Borehole 4 measured a plastic limit of about 12 per cent, a liquid limit of about 20 per cent and plasticity index of about 8 per cent, indicating that the portion of the deposit at Borehole 4 is classified as a clayey silt of low plasticity.

The reported bulk weight of the upper crust portion of the cohesive deposit ranges from about 17.7 kN/m³ to 20.4 kN/m³. The water content of the clayey silt to silty clay deposit ranges from about 22 per cent to 32 per cent. The organic content measured on two samples of the upper crust is 0.4 per cent and 3.8 per cent.

Lower Layer - Cohesive Deposit

Underlying the upper crust, the cohesive deposit exhibits less plasticity and is described as a clayey silt to clay. The measured SPT “N”-values range between 0 blows in Borehole 14 to 16 blows per 0.3 m of penetration. In situ field vane tests carried out within this lower portion of the cohesive deposit measured undrained shear strengths ranging from about 19 kPa to 143 kPa and sensitivities ranging from 1.3 to 10. The field vane test results together with the SPT “N”-values indicates that the lower layer of the cohesive deposit has a soft to very stiff consistency.

Atterberg limits tests were carried out on fifty two (52) samples of the lower layer of the cohesive deposit, and measured plastic limits between about 15 per cent and 22 per cent, liquid limits between about 20 per cent and 58 per cent, and plasticity indices between about 6 per cent and 36 per cent. These test results indicate that this material is predominantly a silty clay of medium plasticity to clay of high plasticity; however, the test results from two samples (from Borehole 3 and 11) indicate that the material would be classified as a clayey silt of low plasticity.

The reported bulk weight of the clayey silt to clay deposit ranges from about 16.8 kN/m³ to 22.3 kN/m³. The water content of the clayey silt to clay deposit ranges from about 12 per cent to 38 per cent. The organic content measured on two samples of the silty clay portion of the cohesive deposit is 0.7 per cent and 2.3 per cent.



4.2.4 Sandy Silt Till

A till deposit described as reddish brown sandy silt or silt was encountered underlying the clayey silt to clay deposit in Boreholes 3 to 12 and 19 at depths of between about 12.8 m and 19.8 m below ground surface (between about Elevations of 76.7 m and 69.2 m). These boreholes terminated within the till deposit at depths between about 14.2 m and 20.3 m below ground surface (between about Elevation 76 m and 67.1 m). The till deposit was penetrated for depths between about 0.2 m and 2.6 m.

The measured SPT “N”-values measured in the till deposit range from about 22 blows to 159 blows per 0.3 m of penetration, suggesting that the till deposit has a compact to very dense relative density.

Atterberg limits test were carried out on one sample of the till deposit from Borehole 5, and measured a plastic limit of about 16 per cent, a liquid limit of about 23 per cent and plasticity index of about 7 per cent. This test result indicates that the fines component of this deposit is classified as a clayey silt of low plasticity to silt of slight plasticity.

It is noted that in the Foundation Investigation Report for Highway 406 underpass at Westchester Avenue, located approximately 200 m to the south, the soil deposit encountered at about the elevation at which the sandy silt till deposit was encountered at the Geneva St. Ramp Bridge site was re-classified as a sandy clayey silt till based on grain size distribution analysis and Atterberg limits testing that was carried out on five (5) samples. It is likely that the sandy silt till deposit encountered in the boreholes at the Geneva Street Ramp Bridge site represents the same deposit and could be classified as a sandy clayey silt; however, only one Atterberg limit test was carried out at the Geneva Street Ramp Bridge site and therefore the soil description for this site is based on the information provided on the Record of Borehole sheets from the 1962 investigation.

The reported bulk weight of one sample of the till is about 21 kN/m³ and the water content of that sample of the deposit is 15 per cent. The water content measured on two other samples of the sandy silt till deposit is 9 per cent and 10 per cent.

4.2.5 Groundwater

The 1962 foundation investigation indicates that groundwater was encountered in Boreholes 2, 3, 4 and 7 to 16 at depths between about 0.9 m and 3.5 m below the ground surface (between about Elevations 88.4 m and 86.0 m); and Boreholes 17, 18 and 20 are indicated as dry upon completion of drilling. There was no indication that a piezometer was installed in these boreholes, although the borehole location plan from the 1962 foundation investigation indicates that four boreholes were designated as a “piezometer hole”. The groundwater elevations noted in the 1962 report are inferred to represent the water level immediately after drilling with wash boring techniques and therefore do not represent stabilized levels.

The groundwater level is expected to fluctuate seasonally and to be higher during wet periods of the year.



5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Sandra McGaghran, M.Eng., P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng, a MTO Foundations Designated Contact and a Senior Consultant of Golder, conducted an independent technical and quality control review of this report.

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N/S RAMP BRIDGE, G.W.P. 2453-13-00
HIGHWAY 406, ST. CATHARINES ONTARIO.**

PART B

**FOUNDATION DESIGN REPORT
HIGHWAY 406 S - GENEVA STREET N/S RAMP BRIDGE
STRUCTURE SITE NO. 18-168
HIGHWAY 406 STRUCTURAL REHABILITATIONS
FOURTH AVENUE TO WESTCHESTER AVENUE
ST. CATHARINES, ONTARIO
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report presents an assessment of and discussion on foundation resistances and provides foundation engineering recommendations for the proposed rehabilitation of the existing Geneva Street Ramp bridge structure. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 1962 investigations at the structure site. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the foundation rehabilitation alternatives and carry out the design of the structure foundations, as may be required. This Foundation Investigation and Design Report, discussions and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Summary of Existing Foundations

Based on the General Plan (Drawing D 5147-2, revision dated July 9, 1969 - "as-constructed") obtained from GEOCRESS, the existing bridge is a five-span structure with a total length of approximately 114 m. The abutments and piers are founded on a pile cap supported on 14BP73 steel H - Piles (equivalent of HP 360 x 108 steel H-piles). The piles are shown to be battered at angles between 8 vertical to 1 horizontal (8V:1H), 6V:1H and 3V:1H. The Foundation Layout drawing (D 5147-3, revision dated July 9, 1969 - "as constructed"), which is available in the GEOCRESS documents, indicates that the piles are to be driven to practical refusal as determined by the Hiley Formula (D.H.O. Std. BD 16-3.4). The pile length below the "cut-off elevation" is noted on the Foundation Layout drawing. The following summarizes the cut-off elevation for the piles, the pile length, the tip elevation and the estimated tip depth below the glacial till surface:

Foundation Unit	Cut-off Elevation of Pile at Pile Cap (m)	Pile Length below Cut-off Elevation (m)	Estimated Pile Tip Elevation (m)	Approximate Till Surface Elevation (m)	Estimated Pile Tip Depth Below Till Surface (m)
West Abutment	97.7	23.8	73.9	76.6	2.7
Pier 4	88.7	16.4	72.3	75.0	2.7
Pier 3	85.0	17.1	67.9	69.2	1.3
Pier 2	85.0	13.1	71.9	75.2	3.3
Pier 1	87.3	14.0	73.3	74.9	1.6
East Abutment	92.7	20.4	72.3	74.6	2.3

The Foundation Layout drawing recommends a "design load" of about 489 kN (55 tons) for each pile at Piers 2 and 3 and a "design load" of about 445 kN (50 tons) for the abutments and Piers 1 and 4. The October 19, 1962



Foundation Investigation Report (GEOCRE 30M03-43) suggests a “safe load” of 530 kN (60 tons) per pile for 12 ¾ inch (0.324 m) diameter steel tube piles driven to practical refusal and recommends that they be driven some 1.8 m into the till stratum, although, as noted above, the as-constructed drawings indicate that 14BP73 H-piles were used as the deep foundation elements at this site.

6.3 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC (2014)) and its Commentary, the proposed underpass structure and foundation system may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a “typical consequence level” associated with exceeding limits states design. In addition, given the desktop nature of the assessment based on the limited level of foundation investigation completed to date (i.e. number of boreholes, borehole depth, laboratory testing that consisted of Atterberg limits and grain size distribution), as presented in Sections 3.0 and 4.0 and that most of the boreholes are not positioned on/at the foundations footprints in comparison to the degree of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design is considered to be a “low degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC (2014) have been used for design in Section 6.4. Also provided in Section 6.4 are the available resistances based on a “typical” degree of site understanding although additional geotechnical investigation would be required to be able to apply the factors for a “typical” degree of site understanding.

6.4 Assessment of Existing Foundations

The geotechnical resistance values are sensitive to the position of the deposits, general level of the groundwater table and the strength of the stratum that the steel H-piles are driven into. Further, as no piezometers were installed during the 1962 foundation investigations to provide an indication of the long-term water level, we have assumed a water level at about Elevation 86 m, which corresponds to an inferred water level approximately 1 m below ground surface.

In accordance with the interpretation of Section 6.5 of the CHBDC (2014), a “low” degree of site understanding site is defined by “limited representative information (e.g. previous experience, extrapolation from nearby and/or similar sites) combined with conventional prediction models to achieve a lower level of confidence with performance predictions”. Whereas, a “typical” understanding of the site is defined as a “typical project-specific investigation procedures and/or knowledge are combined with conventional prediction models to achieve a typical level of confidence with performance predictions”. There is generally one borehole that penetrates the till deposit (where the piles are driven into) on / near the footprint of the foundation units for Piers 3 and 4 and the east abutment; however, some of the boreholes are between about 6 m and 16 m from the perimeter of the foundation units. During the 1962 investigations there were in situ vane shear tests carried out in the cohesive deposits, and the laboratory analysis consisted of water contents, grain size distribution analysis and Atterberg limits, which meets the MTO requirements. Considering that there is not a borehole at each foundation unit and also considering that the boreholes did not penetrate 3 m into soil having an SPT “N”-value of greater 100 blows (i.e. Refusal condition) a “low” degree of site understanding is considered applicable based on the available information from GEOCRE.

As discussed in Section 6.2 the existing piles were reportedly driven to practical refusal into the glacial till stratum as determined by the Hiley Formula (D.H.O. Std. BD 16-3,4). It is noted that the pile tips at the abutment and Piers



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2 and 4 extend to between about 1 m and 3 m beyond the base of the boreholes advanced during the 1962 and therefore the soil conditions at the pile tips are unknown, although in assessing the geotechnical resistances it has been assumed that since the piles were reportedly driven to practical refusal that they were driven into “100-blow” soil. This condition will need to be confirmed by additional boreholes, as proposed and discussed in Section 6.6.

Based on our interpretation of the available information in the GEOCRETS reports from 1962, the reported pile tip elevations presented on Drawing D 5147-3, revision dated July 9, 1969 - “as constructed”, and applying the applicable resistance factors from Tables 6.1 and 6.2 of the CHBDC (2014) for a “typical” consequence level and “low” degree of site understanding, the factored ultimate geotechnical resistance and the factored serviceability geotechnical resistance for 25 mm of settlement for the abutments and piers founded on a pile cap supported on steel H-piles driven to practical refusal are provided below.

Foundation Unit	“Low” Degree of Site Understanding	
	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN) (for 25 mm of Settlement)
East and West Abutments	1,600	2,200 (see Note 1)
Piers 1 and 2	850	1,500 (see Note 1)
Piers 3 and 4	1,100	1,500 (see Note 1)

Note 1. The factored serviceability geotechnical resistance at SLS (for 25 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS condition will govern.

6.5 Summary of Proposed Superstructure Replacement

We understand that the proposed replacement of the super structure does not envisage the need for new foundations or additional piles to accommodate the anticipated additional design loads, provided that the foundation subsoils have sufficient geotechnical resistances to accommodate the expected load increases from the replacement of the superstructure.

Based on the loading evaluations carried out by the MTO Bridge Office, the estimated foundation resistances for the existing and proposed future bridge loads (and per cent differences) are provided below:

Loading / Resistance	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement) (kN)
Existing	640	490
Proposed	850 (33 % greater than existing)	600 (22 % greater than existing)



6.6 Geotechnical Assessment of Proposed Rehabilitation Options

The available factored ultimate geotechnical resistance and the factored serviceability geotechnical resistance, for a "low" degree of site understanding, (CHBDC 2014) are equivalent to or are greater than the required resistances (loadings) at each foundation element for the proposed superstructure replacement. While it is recognized that a "typical" degree of site understand would yield numerically higher values of available factored ultimate and serviceability resistances, providing that the soil conditions below the pile tips is consistent with that encountered in the 1962 investigation, the MTO Foundations standard Terms of Reference for detail design of bridge foundations typically require that two boreholes be advanced into a refusal stratum at each foundation unit. Taking into consideration the existing information from the 1962 investigation, it is recommended that one borehole be advanced at each foundation unit at the east and west abutments and Piers 1 and 2 to confirm the surface of the glacial till deposit and to advance the boreholes 3 m into the glacial till stratum having SPT "N"-values greater than "100-blows", such that the available geotechnical resistances can be confirmed from foundation element specific data for a "typical" degree of site understanding.

7.0 CLOSURE

This report was prepared by Ms. Sandra McGaghran, M.Eng., P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng, a MTO Foundations Designated Contact and a Senior Consultant of Golder, conducted an independent technical and quality control review of this report.

GOLDER ASSOCIATES LTD.



Sandra McGaghran, M.Eng., P.Eng.
Senior Geotechnical Engineer, Associate

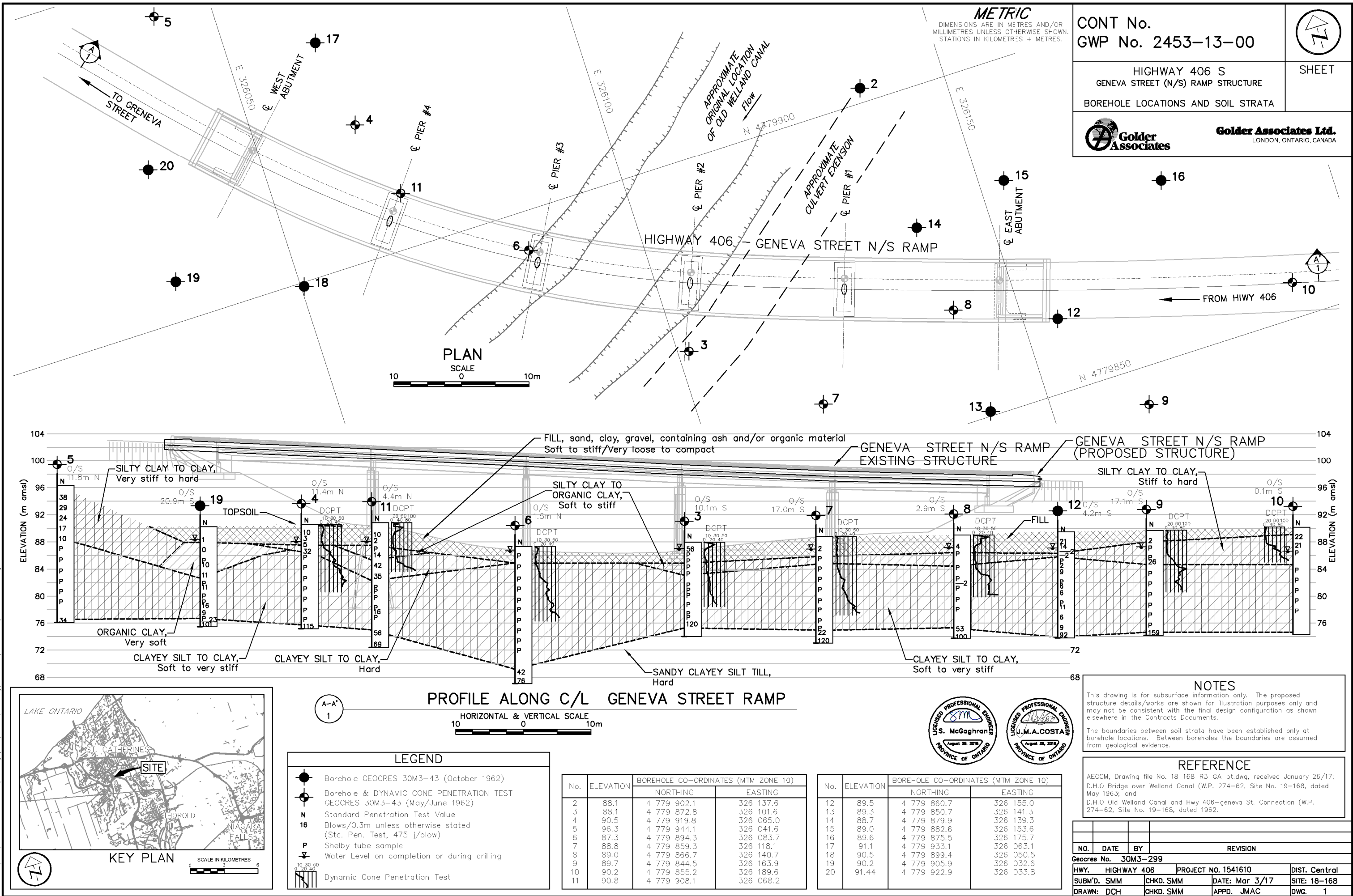


Jorge M.A. Costa, P.Eng.
MTO Foundations Designated Contact, Senior Consultant

SMM/JMAC/rb

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APPENDIX A

GEOCRE 30M3-43 Record of Borehole Sheets and Drawing D-5147-2 Borehole Details

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS:-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	F.H.		SAMPLE ADVANCED HYDRAULICALLY
	P.M.		SAMPLE ADVANCED MANUALLY

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
Q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR = $\frac{c_v t}{d^2}$ (d , DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

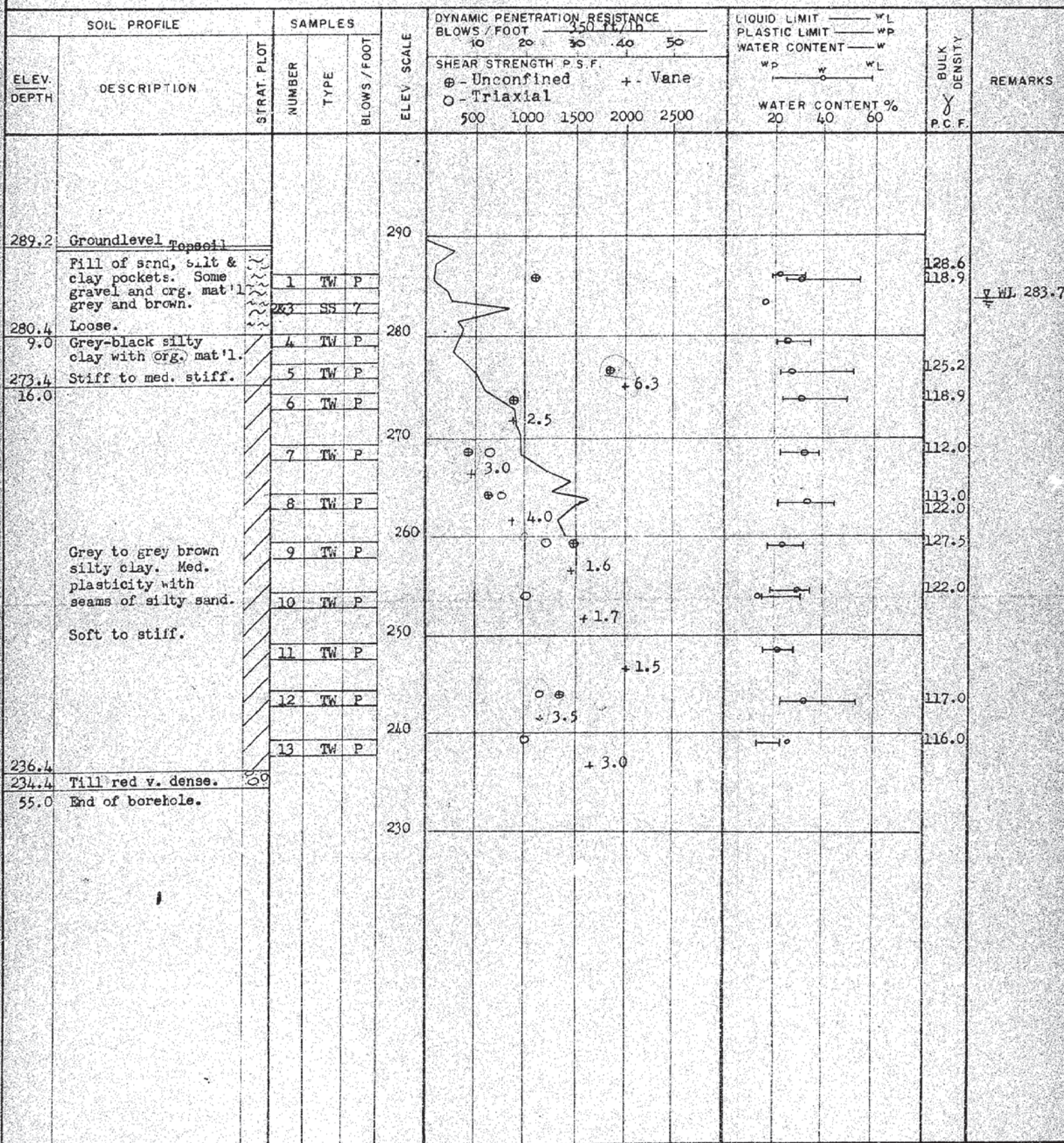
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

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RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 62-F-62 LOCATION Sta. 262+35 (143' Lt.) ORIGINATED BY B.K.
W P 126-58(-1) BORING DATE May 3, 1962. COMPILED BY B.K.
DATUM Geodetic BOREHOLE TYPE Washboring. CHECKED BY M.D.



FOUNDATION SECTION

CHECKED BY M.D.



SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		BLOWS / FOOT		
						10 20 30 40 50	W _p ———— W _L 20 40 60	
297.0	Groundlevel Topsoil		1	SS	10			
	Fill of sand, silt & clay pockets, some org. mat'l. Gravel & cinders; brown to red brown.		2	SS	3			
287.0	Loose to very loose.		3	SS	5			wl 289.0
10.0	Grey to grey-brown silty clay with varying amounts of silt and sand, hard to stiff.		4	TW	32			
			5	TW	P			115
			6	TW	P			
	Layer of red-brown gravel, sandy firm clay between elev 285.0 and 281.0'		7	TW	P			123
			8	TW	P			
			9	TW	P			
			10	TW	P			
			11	TW	P			
248.0								
246.5	Red till-Very dense.		12	SS	115			
50.5	End of borehole.							

FOUNDATION SECTION

ORIGINATED BY G.C.

COMPILED BY G.C.

CHECKED BY M.D.

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w		BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	10	20	30	40	50	w_p			w_L
						SHEAR STRENGTH P.S.F.					WATER CONTENT %				
						⊕ - Unconfined + - Vane ○ - Triaxial									
						500 1000 1500 2000 2500					20 40 60				
016.0 0.0	Groundlevel														
	Brown clay to silty clay. Hard to stiff.		1	SS	38	310									
			2	SS	29										
			3	SS	24	300									
			4	SS	17										
291.0 25.0			5	SS	10	290									
	Gray clay to silty clay. Med. stiff to stiff.		6	TW	P										
			7	TW	P	280									
			8	TW	P										
			9	TW	P	270									109
			10	TW	P										125
			11	TW	P	260									134
251.0 249.5 66.5	Till (Dense) End of borehole.		12	SS	34	250								134	

FOUNDATION SECTION

CHECKED BY M.D.

[illegible]

FOUNDATION SECTION

CHECKED BY M. D.

236

FOUNDATION SECTION

[illegible]

FOUNDATION SECTION

ORIGINATED BY G.C.

COMPILED BY G.C.

CHECKED BY M.D.

Lost.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO

12

FOUNDATION SECTION

JOB 62-F-62

LOCATION Sta. 259 + 70 (10' Lt.)

ORIGINATED BY * JP

W.P. BORING DATE 27th Sept. and 1st Oct. 1962

COMPILED BY MC

DATUM BOREHOLE TYPE Augerhole

CHECKED BY JP

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. ⊕ - Unconfined Field vane test +					WATER CONTENT %			
293.5	Ground level						1000	2000	3000	4000	5000				
0.0	Grey-brown gravelly clay fill														
288.0			1	SS	21	290									
4.5	Rubble fill (ash, clay, gravel sand, organics)		2	SS	14	285									
283.2			3	SS	2										
10.3	Soft black organic clay		4	SS	2										
281.5							+ 3.0								
12.0			5	TW	p	280									
			6	SS	2		+ 2.0								
	grey		7	SS	9	275			+ 2.3						
23.0			8	TW	p	270									
			9	SS	6		+ 2.0								
168.5							+ 1.3								
27.0			10	SS	6	265			+ 2.2						
	grey with traces of red		11	TW	p	260									
	Clay		12	SS	11				+ 1.7						
	silty					255									
40.0			13	SS	6				+ 2.2						
43.0						250									
	very silty, many silt seams		14	SS	9				+ 1.7						
	grey with red seams (varved)					245									
	Reddish brown (silt till)		15a												
242.5			15b	SS	92										

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 13

FOUNDATION SECTION

JOB 62-F-62 LOCATION Sta. 260 + 00 (50' Lt.) ORIGINATED BY * JP
W.P. BORING DATE Oct. 1, 1962 COMPILED BY MC
DATUM BOREHOLE TYPE Augerhole CHECKED BY JP

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT _____ WL PLASTIC LIMIT _____ WP WATER CONTENT _____ W _____ WP _____ WL WATER CONTENT %			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.						
293.0	Ground level												
0.0													
	Gravelly clay fill					290							
289.0													
4.0													
	Rubble fill (ash, clay, gravel, sand, organics)		1	SS	6	285							
283.5													
9.5	Organics												
281.7			2	SS	2								
11.3	Soft black organic clay					280							
280.5													
12.5	Soft grey clay												
278.0													
15.0	End of borehole												

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MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 14

FOUNDATION SECTION

JOB	62-P-62	LOCATION	Sta. 260 + 40 (30' Rt.)	ORIGINATED BY	* JP
W.P.		BORING DATE	2nd Oct. 1962	COMPILED BY	MC
DATUM		BOREHOLE TYPE	Augerhole	CHECKED BY	JP



SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.			WATER CONTENT %				
							WP	W	WL	WP	W			WL
291.0	Ground level													
0.0														
	Gravelly clay fill, trace of organics					290								
285.5														
5.5	Rubble fill (ash, clay, gravel, sand, organics)		1	SS	5	285							285.2 5.8 5/10/62	
283.0														
8.0	Soft black organic clay													
280.5														
279.5	Soft grey clay		2	SS	0	280								
11.5	End of borehole													

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MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 15

FOUNDATION SECTION

JOB 62-F-62 LOCATION Sta. 260 + 00 (50' Rt.) ORIGINATED BY * JP
W.P. BORING DATE 1st October 1962 COMPILED BY MC
DATUM BOREHOLE TYPE Augerhole CHECKED BY JP

SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL		PLASTIC LIMIT — WP		WATER CONTENT — W		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.		WP — W — WL		WATER CONTENT %			
292.0	Ground level													
0.0	Miscellaneous fill (clay, gravel, sand, ash, organics)		1	SS	4	290								
	Loose						285							
283.5	Soft black organic clay													
281.2	Soft grey clay		2	SS	0	280								
280.8	End of borehole													
11.5														

307
283
21

285.7
6.3
1/10/62

307
283
24

285.7
6.3
1/10/62

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 16

FOUNDATION SECTION

JOB 62-F-62 LOCATION Sta. 259 + 25 (50' Rt.) ORIGINATED BY * JP
 W.P. BORING DATE 1st Oct. 1962 COMPILED BY MC
 DATUM BOREHOLE TYPE Augerhole CHECKED BY JP

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE			LIQUID LIMIT — *L PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT			SHEAR STRENGTH P.S.F.				
294.0	Ground level													
0.0														
	Gravelly clay fill, some organics					290								
288.5			1	SS	13									
5.5	Rubble fill, (clay, gravel, sand, ash, organics)					285								
	Loose		2	SS	4									
282.7														
11.3	Soft black organic clay													
281.0														
13.0	Soft grey clay					280								
279.0														
15.0	End of borehole													

286.7
7.3
1/10/62

FOUNDATION SECTION

CHECKED BY JP

* Compiled by Dominion Soil Investigation Limited, Ref: 2-9-L6

DEPARTMENT OF HIGHWAYS - ONTARIO
 MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 18

FOUNDATION SECTION

JOB 62-F-62 LOCATION Sta. 263 + 25 (35' Lt.) ORIGINATED BY * JP
 W.P. BORING DATE 3rd Oct. 1962 COMPILED BY MC
 DATUM BORHOLE TYPE See remarks CHECKED BY JP

SOIL PROFILE			SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	PLASTIC LIMIT — WP	WATER CONTENT — W		
297.0	Ground level										
0.0	Fill										
2.0	gravelly										
			1	SS	11	295					0 - 5'0" - wash bore
			2	SS	20	290					5'0" - 18'0" - auger
	Stiff Silty Clay with traces of organics - Brown to Grey-Brown.		3	SS	33	285					297.0 3.0
			4	SS	19	280					5/10/62 (apparently rain water)
279.0											Open and dry
18.0	End of borehole										3/10/62

FOUNDATION SECTION

CHECKED BY JP

* Compiled by Dominion Soil Investigation Limited. Ref: 2-9-L6

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 20

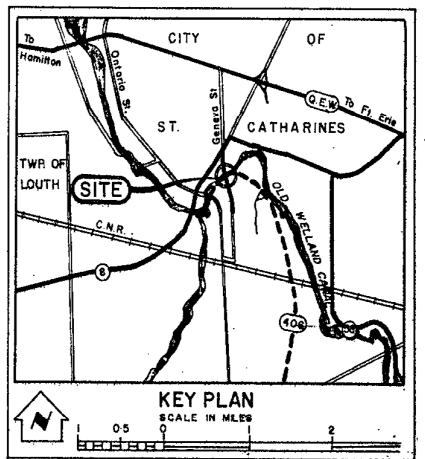
FOUNDATION SECTION

JOB 62-F-62 LOCATION Sta. 264 + 15 (25' Lt.) ORIGINATED BY * JP
W.P. BORING DATE 4 October 1962 COMPILED BY MC
DATUM BOREHOLE TYPE Augerhole CHECKED BY JP

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE			LIQUID LIMIT ——— w_L			BULK DENSITY γ P.C.F.	REMARKS	
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	BLOWS / FOOT			PLASTIC LIMIT ——— w_p				
							SHEAR STRENGTH P.S.F.			WATER CONTENT ——— w				
300.0	Ground level													
299.0	Organic clayey topsoil												Open and dry	
1.0													5/10/62	
	Stiff brown clay													
						295								
292.8			1	SS	20									
7.5	End of borehole													

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS & RESEARCH DIVISION		RECORD OF BOREHOLE NO. 21		FOUNDATION SECTION	
JOB	62-F-62	LOCATION	Sta. 265 + 28 (38' Lt.)	ORIGINATED BY	* JP
W.P.		BORING DATE	4th October 1962	COMPILED BY	MC
DATUM		BOREHOLE TYPE	Augerhole	CHECKED BY	JP

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	BLOWS / FOOT					WATER CONTENT %				
							20	40	60	80	100	WP	W			WL
						SHEAR STRENGTH P.S.F.										
345.0	Ground level															
0.0																
	Miscellaneous refuse, ash, organics, sand, gravel, clay		1	SS	8										Open and dry 4/10/62	
			2	SS	11											
			3	SS	14											
			4	SS	15											
319.2	Stiff grey silty clay		5	SS	21											
25.8			6	SS	21											
315.5	End of borehole															
29.5																



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Piezometer Hole
- Water Levels established at time of field investigation June 1962

NO.	ELEVATION	STATION	OFFSET
2	289.0	260+75	90' RT.
3	289.0	261+46	28' LT.
4	297.0	263+32	38' RT.
5	316.0	264+49	31' RT.
6	286.5	262+35	9' RT.
7	291.3	260+79	48' LT.
8	292.0	260+21	6' LT.
9	294.3	259+25	49' LT.
10	295.8	258+59	5' RT.
11	298.0	262+97	18' RT.
12	293.5	259+70	10' LT.
13	293.0	260+00	50' LT.
14	291.0	260+40	30' RT.
15	292.0	260+00	50' RT.
16	294.0	259+25	50' RT.
17	299.0	263+75	60' RT.
18	297.0	263+25	35' LT.
19	296.0	263+75	60' LT.
20	300.0	264+15	25' LT.

NO'S 1 & 21 NOT SHOWN ON PLAN

BORE HOLE DETAILS

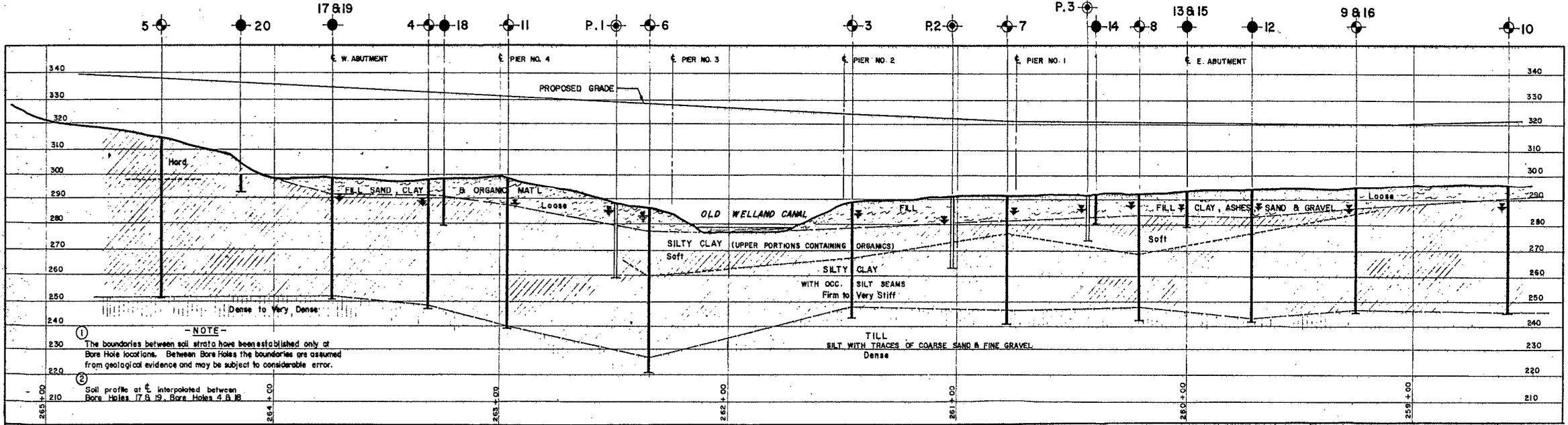
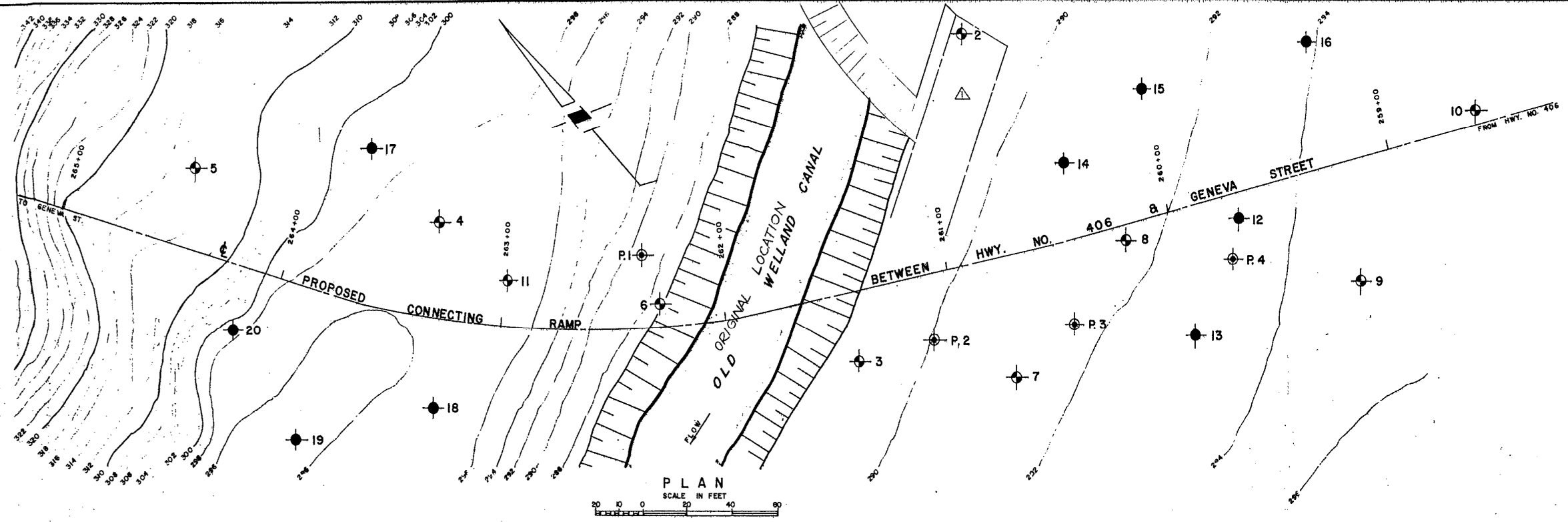
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

OLD WELLAND CANAL
AND
HWY. NO. 406 - GENEVA ST. CONNECTION

ST. CATHARINES

ORIGINATED G. CHERRINGTON	DISTRICT NO. 4	DATE OCT. 12, 1962
DRAWN F. CLARK	W.P. NO. 274-62	JOB NO. 62-F-62
CHECKED G. G.	CONT. NO. 62-F-62	DRAWING NO.
APPROVED J. J. J.	ST. ST. 62-85	D-5147-2

24/10/62 J.S.G. POSITION OF CANAL EXTN SHOWN
THIS DRAWING SUPERCEDES DWG 62-F-62A



NOTE
1 The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.
2 Soil profile at 6 interpolated between Bore Holes 17 & 19, Bore Holes 4 & 18

PROFILE

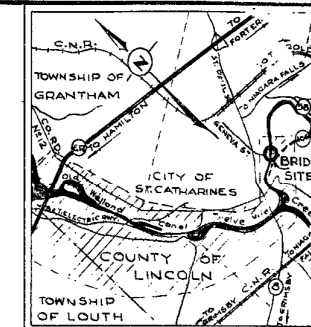
SCALE IN FEET
20 10 0 20 40 60



APPENDIX B

Drawing D-5147-2 General Plan

Drawing D-5147-3 Foundation Layout



KEY PLAN
SCALE: 1/4" = 100'

NOTES

TO ENGINEER

CONCRETE WORK ON THIS STRUCTURE MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE DISTRICT ENGINEER TO CONTRACTOR.

STRUCTURE TO BE BUILT IN ACCORDANCE WITH FORM No. 9 AND THE SPECIAL PROVISIONS, EXTRA COPIES OF WHICH MAY BE OBTAINED FROM THE DISTRICT ENGINEER.

CONCRETE MIX

	MINIMUM STRENGTH AT 28 DAYS	MAXIMUM SIZE OF AGGREGATE
DECK	4000 PSI	3/4"
FOOTINGS, PIERS & ABUTMENTS	3000 PSI	3/4"

APPROVED ADMIXTURES SUPPLIED BY THE CONTRACTOR WILL BE ADDED TO ALL CONCRETE AS SPECIFIED BY THE ENGINEER.

BOILING DATA
THE COMPLETE SOIL INVESTIGATION REPORT FOR THIS STRUCTURE MAY BE EXAMINED AT THE BRIDGE OFFICE OR FOUNDATION OFFICE, DOWNSVIEW OR AT THE REGIONAL OFFICE AND AT THE HAMILTON DISTRICT OFFICE.

CLEAR COVER ON REINFORCING STEEL:

FOOTINGS-3", ABUTMENTS-3", DECK-1"

CONSTRUCTION NOTES:

ALL EXPOSED EDGES TO BE CHAMFERED 1/4" EXCEPT AS NOTED. ALL CONSTRUCTION JOINTS MUST BE APPROVED BY THE BRIDGE ENGINEER. THE GENERAL CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BRIDGE SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF PLUS OR MINUS 1/8" INCH. IF THEY ARE TOO HIGH THEY SHALL BE GRIND DOWN BY THE GENERAL CONTRACTOR. IF THEY ARE TOO LOW THE GENERAL CONTRACTOR SHALL PROVIDE FULL BEARING SURFACES TO BRING THEM UP TO THE CORRECT ELEVATIONS. THE USE OF GROUT IS PROHIBITED. THE GENERAL CONTRACTOR SHALL BE RESPONSIBLE FOR ENSURING THAT THE FINAL DECK ELEVATIONS COMPLY WITH THE ELEVATIONS SHOWN. NO CONCRETE SHALL BE PLACED ABOVE THE BRIDGE UNTIL THE STRUCTURAL STEEL HAS BEEN PLACED.

SEQUENCE OF CONSTRUCTION

SUBEXCAVATION, 1ST STAGE FILL, CULVERT EXTENSION AND CANAL BACKFILL SHALL BE COMPLETED PRIOR TO COMMENCEMENT OF THE BRIDGE PROJECT.

- 1) DRIVE H-PILES. ALL H-PILES FOR PIERS TO BE PLACED PRIOR TO DRIVING H-PILES FOR ABUTMENTS.
- 2) EXCAVATE AND POUR FOOTINGS.
- 3) PLACE SAND CUSHION BACKFILL AT ABUTMENTS UP TO A BERM LEVEL PRIOR TO ERECTION OF STRUCTURAL STEEL.

LIST OF DRAWINGS

- D 5147-1 GENERAL PLAN
- D 5147-2 BORE HOLE DETAILS
- D 5147-3 FOUNDATION LAYOUT
- D 5147-4 SETTING DATA & ELEVATIONS
- D 5147-5 ABUTMENTS
- D 5147-6 PIERS
- D 5147-7 STRUCTURAL STEEL & BEARING LAYOUT
- D 5147-8 GIRDERS DETAILS
- D 5147-9 BRACING & CONNECTION DETAILS
- D 5147-10 BEARING DETAILS
- D 5147-11 DECK DETAILS
- D 5147-12 APPROACH SLABS & END POST DETAILS
- D 5147-13 HANDRAIL & LIGHTING LAYOUT
- D 5147-14 DETAILS OF PARAPET RAILING
- D 5147-15 BRIDGE LIGHTING DETAIL
- D 5147-16 STEEL SCHEDULE (DECK & FOOTINGS)
- D 5147-17 DO (ABUT & PIERS)

REFERENCE DRAWINGS

- SITE PLAN
- PROFILE - CONTR. No. 62-316 SHEET 62
- PLAN
- SOILS REPORT BA 1463 & 1463A

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION

BRIDGE OVER OLD WELLAND CANAL

CONNECTION BETWEEN HWY. No. 406 AND GENEVA ST.
IN ST. CATHARINES

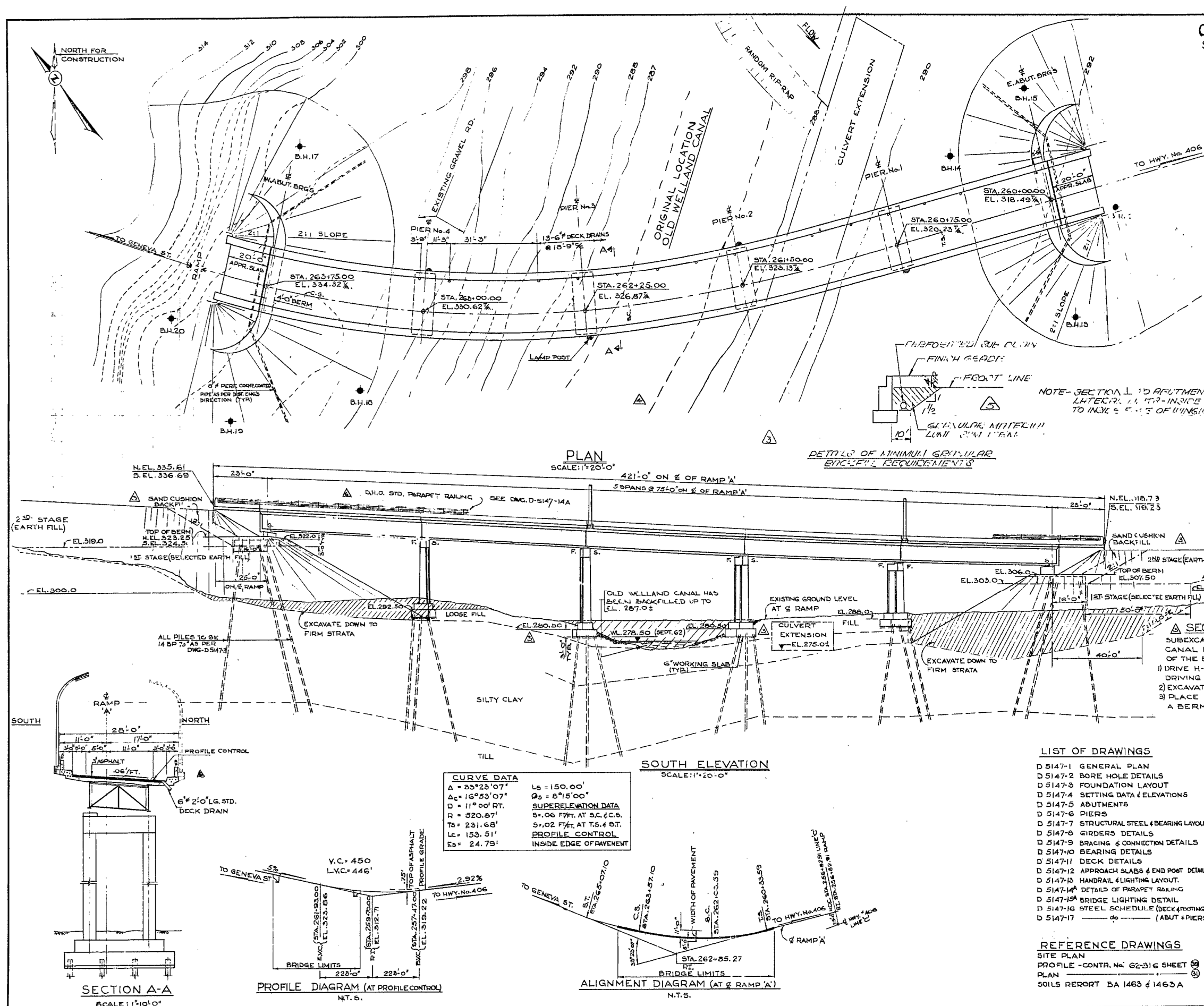
KING'S HIGHWAY No. DIST. No. 4

CO. LINCOLN

CITY OF ST. CATHARINES LOT 1 ON

GENERAL PLAN

APPROVED: <i>[Signature]</i>	SHEET: 19-168	W.P. No. 27A-62
DESIGN: A.U. CHECK: G.S.	CONTRACT: No. 1-05/04-295	
DRAWING: H.M. CHECK: G.S.		
DATE: MAY 63	LOADING: H 20 S 16	DRAWING No. D-51



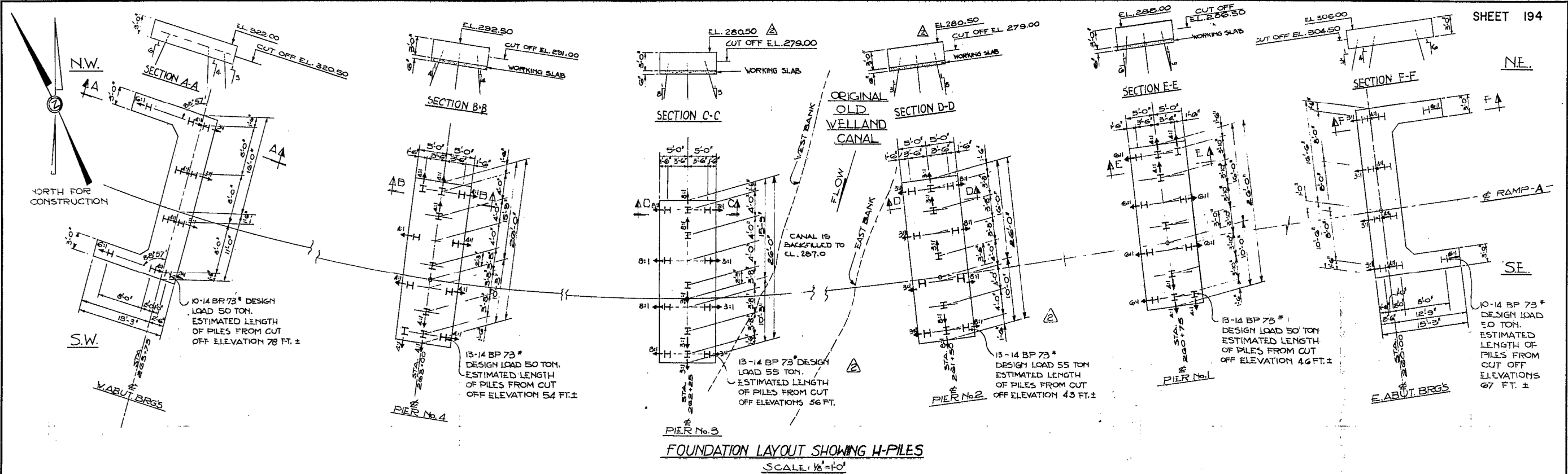
CURVE DATA

$\Delta = 83^{\circ}23'07''$	$L_s = 150.00'$
$\Delta_c = 16^{\circ}53'07''$	$\Delta_c = 8^{\circ}16'00''$
$D = 11^{\circ}00' RT.$	
$R = 520.87'$	
$T_s = 231.68'$	
$L_c = 153.51'$	
$E_s = 24.79'$	

SUPERELEVATION DATA

$S = .06$ FWT. AT S.C. & C.S.
$S = .02$ FWT. AT T.S. & B.T.

PROFILE CONTROL
INSIDE EDGE OF PAVEMENT



NOTES:

SETTING:

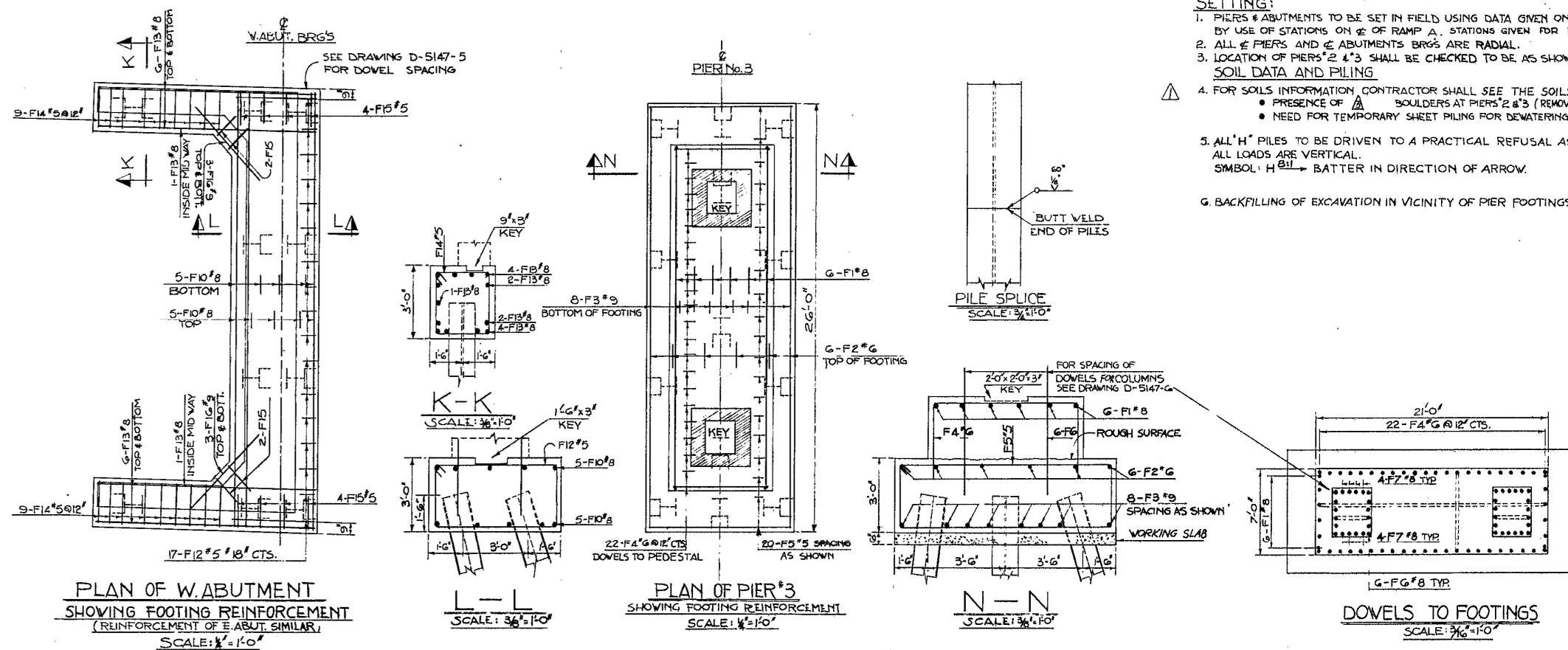
- PIERS & ABUTMENTS TO BE SET IN FIELD USING DATA GIVEN ON DRAWING D-5147-4 TABLE 2. THESE POINTS SHALL NOT BE SET BY USE OF STATIONS ON RAMP A. STATIONS GIVEN FOR FUTURE REFERENCE ONLY.
- ALL PIER AND ABUTMENT BRGS ARE RADIAL.
- LOCATION OF PIERS & ABUTMENTS SHALL BE CHECKED TO BE AS SHOWN ON DRAWINGS IN RELATION TO THE OLD WELAND CANAL.

SOIL DATA AND PILING

- FOR SOILS INFORMATION CONTRACTOR SHALL SEE THE SOILS REPORT. THE FOLLOWING POINTS ARE MENTIONED IN THE REPORT:
 - PRESENCE OF BOULDERS AT PIERS 2 & 3 (REMOVE BEFORE PILING)
 - NEED FOR TEMPORARY SHEET PILING FOR DEWATERING PURPOSES
- ALL H" PILES TO BE DRIVEN TO A PRACTICAL REFUSAL AS DETERMINED BY HILEY FORMULA (SEE D.H.O. STD. BD 16-3, 4). ALL LOADS ARE VERTICAL.

SYMBOL: H 811 → BATTER IN DIRECTION OF ARROW.

G. BACKFILLING OF EXCAVATION IN VICINITY OF PIER FOOTINGS TO BE IN G⁶ COMPACTED LAYERS UP TO TOP OF FOOTING ELEVATIONS.



REVISION	DATE	BY	DESCRIPTION
1	9.7.69	R.T.	REVISED AS-CONSTR.
2	24.10.69	J.G.G.	WORD "SURFACE" REMOVED
3	29.6.73	A.U.	REV ELEVATION NOTES - PIER 3, PIER 4
4	6.6.73	J.G.G.	NOTES REV.

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

BRIDGE OVER OLD WELAND CANAL

CONNECTION BETWEEN HWY. No. 406 AND GENEVA ST.

KING'S HIGHWAY No. 406 IN ST. CATHARINES

CO. LINCOLN

CITY OF ST. CATHARINES

FOUNDATION LAYOUT

APPROVED	DATE	SITE No.	W.P. No.
19.1.68	19.1.68	19-168	274-62
DESIGN	NO.	CHECK	NO.
A.U.	19.1.68	19.1.68	19.1.68
DRAWING	NO.	CHECK	NO.
19.1.68	19.1.68	19.1.68	19.1.68
DATE	NO.	LOADING	NO.
MAY. 63	19.1.68	19.1.68	19.1.68

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