PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT FOR
DESJARDINS CANAL, EBL AND WBL
HIGHWAY 403
GWP 2357-09-00, SITE NO. 36-36/1 AND 36-36/2
CENTRAL REGION, ONTARIO

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for
Desjardins Canal Bridge, EBL (Str. Site # 36-36/1) and WBL (Str. Site # 36-36/2)
GWP 2357-09-00
Highway 403
Central Region, Ontario

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1. INTRODUCTION

The Foundation Engineering Services for this assignment involve the preliminary foundation investigation and design for rehabilitation/replacement of the Hwy 403 EBL and WBL Bridges over the Desjardins Canal near Hamilton, Ontario.

The study was carried out for Morrison Hershfield Limited on behalf of Ministry of Transportation of Ontario (MTO).

The purpose of this Preliminary Foundation Investigation and Design Report (PFIDR) is to provide a preliminary design level summary of the subsurface and groundwater conditions based on available foundation reports in order to address the foundation aspects of the proposed structural rehabilitation, widening or replacement strategy. The purpose of this document is also to update the foundation design recommendations provided in the available reports to limit state design terminology in conformance with the requirements of the Canadian Highway Bridge Design Code (CHBDC). Further, this report comments on the current relevance and adequacy of the foundation design recommendations provided in the available reports and updates the recommendations for bearing resistance to the current standard for design.

This PFIDR is based on desk top level review of available foundation reports in the MTO GEOCRES Library that are related to these sites. No additional subsurface investigations were carried out. The subsurface description is inferred from the information in the Foundation Investigation Report referenced in Section 2 of this PFIDR. The foundation assessment is inferred from the recommendations in the Foundation Design Report referenced within this memorandum.

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There are two components addressed in the report. The first addresses the evaluation of the existing foundations. The second addresses the widening option.

Axial bearing resistance was assessed as an indicator of the adequacy of the foundations for the proposed rehabilitation. The original recommendations are summarized and translated to limit state design terminology per the “Highway Bridge Design Code” (CHBDC). In addition, Foundation Design recommendations in accordance with the requirements of the current CHBDC, based on assessment of subsurface conditions described in the previous Foundation Investigation Report, are provided. The results of the evaluation of the existing foundations the related recommendations also apply to the widening option.

The widening component of the report includes evaluation of options for locations of foundations and for types of foundations.

2. SOURCES OF INFORMATION

The following report and drawings were available for review and information for the Desjardins Canal Bridges.

1. Foundation Investigation Report, Proposed Desjardins Canal, EBL, HWY 2, Report No. 8-500/55/T-151-1, Hamilton, Ontario, Racey MacCallum Associates. November 1955.

2. Foundation Investigation Report, Proposed Desjardins Canal Bridge, WBL, HWY 403, District No. 4 (Hamilton), W.P. 193-60-1, Hamilton, Ontario, Geocon Ltd., May 1960. In Geocon’s investigation, 10 boreholes were drilled - 6 boreholes were carried out over water from a raft supplied by the Department of Highways. Locations and Soil Strata (Drawing No. 57067-1) dated June 1960.

3. Foundation Plan (possible Drawing No. TWP #1336-36-2-A) dated August 1961.

4. Pile Loading Test Report, Proposed Desjardin Canal Bridge, EBL, Hwy 403, District No. 4 (Hamilton), W.J. 62-F-49, Hamilton, Ontario, Geocon Ltd., January 1963.

5. Bridge Deck Rehabilitation (WP No 229-77-04 Sheet 60) dated April 1984.
6. Bridge Deck Rehabilitation (Drawing No. 1) dated December 1984.
7. Bearing Replacement (Drawing No. 1) dated September 2009.

3. SITE BACKGROUND AND GEOLOGY

The Desjardins Canal Bridges on Highway 403 are located near Hamilton in Ontario. A key plan based on Goggle map is shown in Figure 1. Figure 2 provides a detailed site plan illustrating the topography of the site and the existing bridges and approach embankments.

The site forms part of the Lake Iroquois offshore deposits consisting mainly of fine-grained sands, becoming silty with depth and resting on early Lake Iroquois clays. The deposits are generally stratified and may include beaches and swampy areas.

4. SITE RECONNAISSANCE

As part of the current foundation engineering assessment study, a site reconnaissance of the Desjardins Canal Bridge was carried out in spring 2013. A photographic record of the site visit is attached in Appendix C. If there is insufficient information in this PFIDR to reach decision on the foundation strategy for this project, a preliminary foundation investigation and design investigation can be carried out and include a more comprehensive foundation inspection.

The following observations are based on the photographic record for the purpose of assessing the performance of the foundations only. They are not intended to represent a structural assessment of the bridges. The observations and assessments are limited to performance of the foundations.

4.1 Eastbound Lane (EBL)

The site photos illustrate current conditions at the north and south abutments and west piers of the EBL structure based on visual observations, including the general appearance of the structure as it relates to the performance of the foundations, conditions at the abutments, piers and deck, conditions of the existing trail and the vegetation in the immediate vicinity (photographs 1 to 19). There were no cracks or discontinuities observed that indicated foundation distress. If it is planned to retain the foundations for a rehabilitated structure, the extent of scour would have to be assessed.

4.2 Westbound Lane (WBL)

The site photos present current conditions at the north and south abutment and piers of the WBL structure based on visual observations, including the general appearance of the structure as it relates to the performance of the foundations, conditions relating to slope stability, bridge deck, joints and the vegetation in the immediate vicinity (photographs 20 to 23). There were no cracks or discontinuities observed that indicated foundation distress. If it is planned to retain the foundations for a rehabilitated structure, the extent of scour would have to be assessed.

5. FOUNDATION INVESTIGATIONS AND SUBSURFACE CONDITIONS

5.1 Eastbound Lane

A foundation report was prepared by Racey MacCallum Ltd. in 1955 for the Desjardins Canal Bridge, EBL. Based on the information in the Foundation Investigation Report (Reference 1), a general description of the subsurface conditions at EBL site follows:

- The subsoil consisted of organic loose silt extending from the surface to elevation 53.3 m. The bedrock surface was at elevation 51.8 m in the deepest part of the ravine and consisted of red clay shale with hard clean shale interbeds.
- The organic soil was very compressible and susceptible to settlements and stability issues under applied loads.

Refer to Appendix A, Drawing A-1 for a borehole location plan and indication of depths of probes at the EBL structure (from Reference 1).

5.2 Westbound Lane

A foundation report was completed by Geocon Ltd. in May 1960 for the proposed Desjardins Canal Bridge, WBL. Based on the information in the Foundation Investigation Report (reference 2), a general description of the subsurface conditions at WBL site follows:

- Ten (10) boreholes, accompanied by dynamic penetration tests, were drilled. Six (6) boreholes were carried out over water from a raft. The drilling was carried out using a standard machine drilling. Boreholes were advanced up to 31 m in depth.
- The subsoil at the site generally consisted of very loose to compact fine to medium uniformly graded sand, becoming dense to very dense with depth. Above the sand deposit and beneath the canal bottom and extending into the banks at either side of the canal was a layer of very loose to loose organic sandy silt about 0.6 to 3 m in thickness. The fine to medium sand stratum contains thin layers or lenses of silt and clay and at depth graded to compact to very dense sandy silt which in turn graded to very stiff clay interbedded with silty and sandy layers.
- Groundwater level observation pipes were installed in 4 boreholes on May 1960. Groundwater levels measured were found to be equal and stationary at elevation 75.1m. This elevation corresponded with that of the water level in the canal at the time of the observation.

Refer to the Appendix A, Drawings A-2a and A-2b for borehole location plans and stratigraphical profiles for the WBL structure (from Reference 2). The Record of Borehole sheets and groundwater information of the 10 boreholes from Reference 2 are presented in Appendix B.

6. DISCUSSION AND PRELIMINARY FOUNDATION RECOMMENDATIONS

6.1 Existing EBL and WBL Structures

These bridges carry Highway 403 over the Desjardins Canal which connects Hamilton Harbour and Cootes Paradise in Hamilton. The canal is about 30.5 m wide and about 3 m deep.

There are two structures at the site. The East Bound Lane (EBL) carries Toronto-bound traffic; the West Bound Lane (WBL) carries Hamilton-bound traffic. Refer to Appendix D for general arrangements of the EBL and WBL structures and for a section illustrating the proximity of the structures and their foundations.

The following details were provided by Morrison Hershfield:

Original drawings of the substructures are not available. The only information available is from Contract 85-68 for the rehabilitation of the EBL and WBL structures. Sheet 41 shows the east elevation of the EBL structure, including the footings and abutments, but with no dimensions shown. By scaling off the drawing, approximate dimensions and elevations of the footings are as presented below.

Clear Gaps between EBL structure and WBL structure decks

Refer to Section Illustrating Proximity of Structures and Foundations in Appendix D.

If the section is approximately correct, then the clear gap between the footings is approximately 3.2 m and the clear gaps between edges of the decks of the two bridges are as follows:

- 5.06 m near south abutment
- 4.92 m near south pier
- 4.76 m near north pier
- 4.99 m near north abutment

EBL structure

The span arrangement for the EBL structure is approximately 13.7 m, 24.4 m, 13.7 m.

- Pier footings are 4.8 m wide along the bridge and 2.0 m deep. Drawings showing width in in the transverse direction are not available.
- It has been assumed the footing extends 2.0 m (same as depth) beyond the pier.
- The footings are founded at elevation 70 m.

WBL structure

The span arrangement for the WBL structure is approximately 17.4 m, 24.4 m, 17.4 m.

- Pier footings are 4.57 m x 3.66 m, with top of footing at approx. 72.9 m as scaled from Contract 85-68 sheet 60, and this agrees with the original contract drawing (within 0.1 m);
- Battered piles are at 4 vertical to 1 horizontal. The tip elevations of piles are not known.
- There are also piles at the abutments.

In the absence of detailed information for the EBL structure the following information has been collected from the Foundation Report for the WBL structure.

- The EBL bridge is a three-span reinforced concrete structure. From available information, it is inferred that the abutments are founded on stepped spread footings in the sand stratum. The east and west ends of the north abutment are assumed to be founded at elevation 79.2 and 77.4 m respectively. The corresponding elevations at the south abutment are 78.3 and 76.8 m respectively. The pier footings are assumed to be founded in the sand stratum at about elevation 70m. The allowable bearing pressure for spread footings for the WBL structure was about 300 kPa. In the absence of information about bearing pressure at the EBL structure, it assumed that the bearing resistance would have been similar or lower.

- The WBL bridge is a three-span simply supported steel structure. The maximum pier and abutment structural loadings were reported to be in the order of 4450 kN and 3115 kN respectively. The foundation report (reference 2 in Section 2) indicated that the WBL bridge structure could be founded on spread footing foundations. However, to accommodate probable scour requirements, it was further recommended that the bridge could be founded on either 200 mm tip diameter timber piles or 305 mm diameter pipe piles. Pile resistances were recommended in the Foundation

Investigation and Design Report (Reference 2). The original foundation design bearing recommendations are summarized in Tables 1a and 1b along with PML recommended upper level bearing resistance values updated to reflect current industry practice.

6.2 Rehabilitation of Existing EBL and WBL Structures

It has been reported that the EBL and WBL decks were previously rehabilitated in 1984 and the WBL bearings were replaced in 2009.

Refer to Tables 1a and 1b for preliminary planning recommendations regarding axial bearing resistance for the EBL and WBL structures respectively. Axial bearing resistance was used as the indicator of the adequacy of the foundations for the proposed rehabilitation and the recommended resistance values are intended to be compared with actual loads for the rehabilitated structures. Tables 1a and 1b provide the original foundation design axial bearing recommendations in both original working stress format and translated to limit state design format along with PML recommended upper level bearing resistance values updated to reflect current industry practice.

The values presented in the table are based on information in the available previous foundation reports and contract drawings referenced in Section 2.

The previous report and detailed contract drawings for the EBL structure were either unavailable or not fully legible and recommendations for design bearing resistance were not available. The values shown in Table 1a for bearing resistance for the EBL structure are inferred from comments in the foundation report for the WBL structure and their accuracy is not considered to be reliable. The values shown are for illustration purposes only and are not to be used for design.

The values shown in Table 1b for resistance of the pile foundations for the WBL structure are inferred from the referenced report and a subsequently discovered general arrangement drawing for that structure. However, there is fully reliable information was available as to the length of the piles. The lengths for timber piles that were assumed for assessment are indicated in Table 1b. The values shown are for illustration purposes only and are not to be used for design.

To determine the acceptability of the existing foundations for the proposed rehabilitations, the loads imposed by the rehabilitation designs should be checked against the PML preliminary planning recommendations for upper level bearing resistance values for conformance to the requirements of the CHBDC. Consideration should also be given to the advanced age of the existing foundations, the presence of untreated timber piles (which are susceptible to deterioration especially at the water-air interface) at the WBL piers and the existence of weak and compressible deposits that could potentially impose additional loads on existing or new foundations through settlement or slope instability mechanisms.

6.3 Widening of EBL or WBL Structure

It is noted that widening of the EBL structure was not identified as an option by others, probably due to the advanced age of the that structure.

Refer to Tables 2 and 3 for evaluations of foundation location options and foundation types, respectively. The information provided in the tables and in this report is for Preliminary Design and planning purposes and is not intended for detail design. Detail Design level Foundation Engineering services and reporting would be required within the scope of detail design to provide recommendations for design.

Further evaluation would be required at the detail design phase of the project to develop recommendations for widening approach embankments. These recommendations might include requirements for preloading widened portions, utilizing wick drains provided that this would not adversely affect existing foundations and using lightweight fill in the zone immediately behind abutments.

It may be beneficial to consider offsetting the location of abutments and piers in relation to those for the existing WBL structure. Temporary and/or permanent retaining structures may be required to facilitate construction and for the permanent configuration.

The consensus from industry sources is that work from ground or lake level would be more efficient for construction of the foundation for the widening than would work from the existing bridge deck. This constraint would also be consistent with the recommendation that the no additional load from the widening should be imposed on the foundations for the existing WBL structure.

Full scale pile load tests would be required if newer technologies such as auger cast piles or micro piles are adopted for the widening.

A monitoring program would be required with 24/7 alert and contingency protocol to manage the risks of loss of ground and vibration adversely affecting the performance of the existing WBL structure.

6.4 Replacement of EBL or WBL Structure with new bridge

The option of replacing the existing structure(s) became a consideration during the course of the preliminary design. The configuration of a replacement bridge at either alignment would consist of a single span structure, probably founded on driven H-piles.

6.5 Evaluation of Foundation Options for Widening and Replacement of EBL or WBL Structures

Refer to Tables 2 and 3 for evaluations of foundation location options and foundation types, respectively. The information provided in the tables in this report is for Preliminary Design and planning purposes and is not intended for detail design. Detail Design level Foundation Engineering services and reporting would be required within the scope of detail design to provide recommendations for design. The information provided in the tables applies where relevant to both the widening and the replacement options.

Table 2 - Evaluation of Foundation Location Options

Widening between the existing EBL and WBL structures is not recommended from a foundation engineering perspective due to concerns with insufficient space for construction, distress to both the EBL and WBL structures caused by loss of ground and /or vibrations from construction activities. The following details relate to these concerns:

- The spread footings of the EBL structure extend beyond the limits of the EBL bridge deck. This encroachment further reduces the already minimal space between the bridges.
- The pier foundations of the WBL structure include battered piles, which encroach on the already limited space between the structures.
- The pile foundations at the piers of the WBL structure are untreated timber piles susceptible to breakage by physical impact such as might occur during installation of new driven piles for a widening.

Widening on the west side of the existing WBL is feasible. There would be sufficient space for construction activities, although off shore operations may be required. Also the issue of distress to the existing structures due to construction activities would probably be restricted to the WBL structure. For this option, the widening should be self-supporting and no additional loads from the widening should be imposed upon the foundations of the existing WBL structure.

The issue of avoiding interference between the existing foundations would still exist but would be restricted to the foundations for the existing WBL structure. The interference issue would exist at the pier locations and may exist at the abutment locations. Further investigation during detail design would be required to assess the foundation interference issue at the abutments.

The foundations engineering aspects of an option consisting of a <u>new bridge located to the west of the existing WBL structure</u> was also considered. The issues are largely consistent with those for the widening option to the west of the existing WBL structure. However, consideration could be given to the benefits of a separate structure for staging for inevitable future replacement of the existing WBL structure. A new bridge located to the east of the EBL structure was also considered to be a feasible option. Similarly this new bridge would be valuable for staging for the future replacement of either the WBL or EBL structures.

Table 3 - Evaluation of Foundation Type Options

The driven pile option is considered to be viable but carries concerns with the effects of vibration on the foundations of the existing structures. Further assessment of this option would be required during the detail design phase if further consideration is to be given to selecting driven piles. For preliminary design purposes, it should be assumed that driven piles must be outside a 5 m radius of existing WBL pier pile caps and of existing EBL spread footings at both abutments and piers.

Caissons are not a preferred foundation type due to concerns with undermining the foundations of the existing structures.

Auger cast piles are not a preferred foundation type at this stage of the investigation due to concerns with cost and capacity and its suitability for a bridge widening project of this nature. Further assessment of this option would be required during the detail design phase of the work.

Micro piles are considered to be a viable option for new foundations at both piers and abutments of both the EBL and WBL structures. This option carries concerns with the effects of loss of ground on the foundations of the existing structures. Further assessment of this option would be required during the detail design phase if further consideration is to be given to micro piles. For preliminary design purposes, it should be assumed that micro piles must be outside a 3 m radius of existing WBL pier pile caps and of existing EBL spread footings at both abutments and piers. Their construction would be expensive due to the complexity of the installation operation and the measures required to achieve recommended resistances and to minimize loss of ground. The micro pile installation would also require measures to manage concrete transport if off-shore work is required and to manage the environmental issues around spoil from drilling the micro piles.

Supporting information for determining the minimum offset for the both driven piles and micro-piles follows:

- The Quantity sheet and contract general layout are the references to justify assumptions about the horizontal encroachment of the battered WBL pier piles (timber piles) outside the confines of the footing.

- The General Arrangement for Chedoke Bridge dated Aug. 15, 1961 indicates the following length for pier piles:

South Pier		North Pier	
West side	40’ (12.2 m)	West side	30’ (9.1 m)
East side	20’ (6.1 m)	East side	20’ (6.1 m)

- The contract drawings for the existing structure, the battered piles are at 1H:4V.

- The Q sheet shows 80 timber piles for a total length of 2200 feet, which corresponds to an individual pile length of 27.5 feet (8.4 m).

- The pile lengths inferred in the two documents are slightly different, but the details are consistent enough for preliminary design purposes for planning the minimum offset for new deep foundations.

- From this, the horizontal component of the battered piles is 6.875 feet and subtracting the distance that the top of piles are from the edge of pile cap (1.5 feet), the horizontal encroachment of these battered piles is 5.375 feet or 1.64 m beyond the edge of pile cap.

- The minimum offset for driven piles is 5 m from edge of existing footing in order to mitigate the risk of physical damage to the existing piles and settlement of the existing piles induced by vibration.

- The minimum offset for micropiles is 3 m from the edge of existing footing in order to mitigate the risk of physical damage to the existing piles and settlement of the existing piles induced by loss of ground.

7. REQUIREMENTS FOR FOUNDATIONS ENGINEERING FOR DETAIL DESIGN

The preliminary design options presented in this report, including the risk, cost and capacity aspects are intended for planning purposes. These options should be considered in consultation with MTO including the MTO Foundations office to select the preferred option for detail design, which will probably be a deep foundation relying on skin friction.

Additional site investigation and analysis would be required for detail design including further assessment of the following aspects:

- stability and settlement of the proposed widened approach embankments including requirements for subexcavation, shoring, benching, preloading/surcharging and/or lightweight fill materials
- consideration of down drag loads imposed on existing abutment foundations by embankment widening
- specific ground resistance parameters for design of the selected foundation
- consideration of construction methodology to minimize the potential for causing settlement of the existing WBL structure foundations that could be caused by loss of ground or vibration
- development of a strategy for instrumentation and monitoring of potential distress to the structures by the proposed construction activities
- full-scale load test if micro-pile foundations selected
- investigation for shoring and retaining walls required for alignment changes to Hwy 403

8. CLOSURE

This PFIDR was prepared by Mr. David Dundas, P.Eng., Senior Engineer with the assistance of Ms. Marzieh Kamranzadeh, MSc, Project Supervisor and was reviewed by Mr. Carlos Nascimento, P.Eng., Principal Consultant, Project Manager and MTO Designated Principal Contact.

We trust this memo is sufficient for your immediate needs. Please do not hesitate to contact us if you have any inquiries and/or comments.

Yours very truly,

Peto MacCallum Ltd.

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TABLE 1a

FOUNDATION DESIGN ASSESSMENT – SPREAD FOOTINGS (Preliminary Recommendations)

EBL Structure [Toronto Bound]

Foundation	Foundation Type	Founding Elevation		Previous Working Stress Values		Previous Equivalent Limit State Design Values				Limit State Design Values updated to current industry practice			
						Bearing Resistance (kPa)**		Load Resistance (kN)		Bearing Resistance (kPa)**		Load Resistance (kN)	
		(ft)	(m)	Safe Bearing Resistance (tons/sf)*	Safe Load Resistance (tons)*	SLS	Factored ULS	SLS	Factored ULS	SLS	Factored ULS	SLS	Factored ULS
North Abutment	Spread footing	Stepped 260 east side and 254 west side	Stepped 79.3 east side and 77.4 west side	3	N/A	<300	<500	N/A	N/A	<300	<500	N/A	N/A
North Pier	Spread footing	230	70	3	N/A	<300	<500	N/A	N/A	<300	<500	N/A	N/A
South Pier	Spread footing	230	70	3	N/A	<300	<500	N/A	N/A	<300	<500	N/A	N/A
South Abutment	Spread footing	Stepped 257 east end and 252 at west end	Stepped 78.3 east end and 76.8 at west end	3	N/A	<300	<500	N/A	N/A	<300	<500	N/A	N/A

Notes: * Inferred values from those reported for WBL structure since actual values and subsurface information for EBL are not available. For preliminary design purposes.
** The Factored ULS preliminary design values are based on the working stress. The SLS preliminary design values are inferred.

TABLE 1b

FOUNDATION DESIGN ASSESSMENT – PILE FOUNDATIONS (Preliminary Recommendations)

WBL Structure [Hamilton Bound]

Foundation	Foundation Type	Length/Founding Elevation± (m)		Previous Working Stress Values: Safe Bearing Resistance (tons/sf)	Previous Working Stress Values: Safe Load Resistance (tons)	Previous Equivalent Limit State Design Values: Bearing Resistance (kPa) SLS	Previous Equivalent Limit State Design Values: Bearing Resistance (kPa) Factored ULS	Previous Equivalent Limit State Design Values: Load Resistance (kN)* SLS	Previous Equivalent Limit State Design Values: Load Resistance (kN)* Factored ULS	Limit State Design Values updated to current industry practice: Bearing Resistance (kPa) SLS	Limit State Design Values updated to current industry practice: Bearing Resistance (kPa) Factored ULS	Limit State Design Values updated to current industry practice: Load Resistance (kN)* SLS	Limit State Design Values updated to current industry practice: Load Resistance (kN)* Factored ULS
North Abutment	Steel Pipe Piles 300.4 mm OD 6 mm wall thickness	15/66.0		N/A	40 used but may support up to 70	N/A	N/A	400	600	N/A	N/A	400	600
North Pier	Untreated timber 200 mm diameter timber piles*	W side 9/63.5	E Side 6/66.6	N/A	25	N/A	N/A	250	375	N/A	N/A	250	375
South Pier	Untreated timber 200 mm diameter timber piles*	W side 12/60.5	E side 6/66.6	N/A	25	N/A	N/A	250	375	N/A	N/A	250	375
South Abutment	Steel Pipe Piles 300.4 mm OD 6 mm wall thickness	15/66.0		N/A	40 used but may support up to 70	N/A	N/A	400	600	N/A	N/A	400	600

Note: * The Factored ULS preliminary design values are based on the working stress. The SLS preliminary design values are inferred.

TABLE 2

Widening of WBL Structure and Replacement of EBL or WBL Structures

Evaluation of Foundation Location Options

Option #	Description	Advantages	Disadvantages	Relative Risk Level*	Relative Cost*
1	- Widening on both sides of existing WBL structure. - Abutment and pier supports located adjacent to foundations of existing EBL and WBL structures but foundations offset in longitudinal direction to span across existing abutment and pier support locations to avoid interference with existing EBL and WBL foundations.	N/A	- Severe space restriction between existing EBL and WBL structures. - Proximity to foundations of existing EBL and WBL structures could cause distress to the either or both structures through undermining by loss of ground, settlement due to vibration during construction of widening foundations, physical damage to the foundations of either or both bridges especially in view of the battered timber pile foundations at the piers of the existing WBL structure and the extension of the spread footings beyond the edges of the deck at the existing EBL structure - Dual foundations at each support required to span in longitudinal direction across area of interference with existing foundations would be required. - Caution required in selection of foundation construction method to avoid causing distress to foundations of existing WBL structure. - Monitoring of vibrations and settlement of existing bridges required.	very high	high

TABLE 2

Widening of WBL Structure and Replacement of EBL or WBL Structures

Evaluation of Foundation Location Options

Option #	Description	Advantages	Disadvantages	Relative Risk Level*	Relative Cost*
2	- Widening on both sides of existing WBL structure. - Abutment and pier support and their foundations located outside zone of interference of foundations of existing EBL and WBL structures.	NA	- Severe space restriction between existing EBL and WBL structures. - Proximity to foundations of existing EBL and WBL structures could cause distress to the either or both structures through undermining by loss of ground, settlement due to vibration during construction of widening foundations, physical damage to the foundations of either or both bridges especially in view of the battered timber pile foundations at the piers of the existing WBL structure and the extension of the spread footings beyond the edges of the deck at the existing EBL structure required. - Potential different span deflection characteristics between widening and existing WBL bridge due to differing span geometries. - Caution required in selection of foundation construction method to avoid causing distress to foundations of existing EBL and WBL structures. - Monitoring of vibrations and settlement of existing EBL and WBL bridges required.	high	high

TABLE 2

Widening of WBL Structure and Replacement of EBL or WBL Structures

Evaluation of Foundation Location Options

Option #	Description	Advantages	Disadvantages	Relative Risk Level*	Relative Cost*
3	- Widening on west side of WBL structure only. - Abutment and pier supports located adjacent to foundations of existing WBL structures but foundations offset in longitudinal direction in to span across abutment and pier support locations to avoid interference with existing WBL bridge foundations.	- Sufficient space for construction of foundations.	- Proximity to foundations of existing WBL structure could cause distress to the structure through undermining by loss of ground, settlement due to vibration during construction of widening foundations, physical damage to the foundations especially in view of the battered timber pile foundations at the piers of the existing WBL structure. - Dual foundations at each support required to span in longitudinal direction across area of interference with existing WBL structure foundations would be required. - Caution required in selection of foundation construction method to avoid causing distress to foundations of existing WBL structure. - Monitoring of vibrations and settlement of existing WBL bridge required.	Moderate to high	medium

TABLE 2

Widening of WBL Structure and Replacement of EBL or WBL Structures

Evaluation of Foundation Location Options

Option #	Description	Advantages	Disadvantages	Relative Risk Level*	Relative Cost*
4	- Widening on west side of WBL structure only. - Abutment and pier supports and their foundations located outside zone of interference of foundations of existing EBL and WBL structures.	- Sufficient space for construction. - Eliminates concerns with foundation construction causing distress to existing EBL structure foundations. - Reduces concerns with construction causing distress to existing WBL structure foundations.	- Caution required in selection of foundation construction method to avoid causing distress to foundations of existing WBL structure. - Monitoring of vibrations and settlement of existing bridges required.	moderate	medium
5	- New independent bridge either on east side of existing EBL structure or on west side of the existing WBL structure.	- Sufficient space for construction. - Eliminates concerns with foundation construction causing distress to existing EBL structure foundations. - Reduces concerns with construction causing distress to existing structure foundations. - Freedom to design span geometry for most practical foundation layout without concern with integration with existing structure deflections. - Facilitates future replacement of existing bridges.	- Caution required in selection of foundation construction method to avoid causing distress to foundations of existing EBL or WBL structures respectively. - Monitoring of physical damage, loss of ground and vibrations and of settlement of existing bridges required.	low	medium

***** Refer to Section 6 Preliminary Foundation Investigation and Design Report for details.

TABLE 3

Evaluation of Foundation Type Options

(applicable to either widening or replacement options)

Option #	Description	Advantages	Disadvantages	Relative Risk Level*	Relative Cost**
1	Driven steel piles – either H-piles or tube piles. Probably 300mm equivalent diameter.	- Routine equipment and installation techniques.	- Extensive space requirements for cranes and pile driver, concrete installation for pile caps with probable off-shore work from barge. - Vibrations from pile driving could induce settlement of existing foundations of existing structures.	- Installation problem risk low. - Loss of ground risk very low. - Vibration risk high, although there is a possibility it could be mitigated through optional pile driving techniques such as hydraulic-based pile driving.	- Moderate cost, which would increase if there is off-shore work. - Estimated costs are $300/m of installed length, each pile to be approximately 20m in length and providing an estimated resistance of 1000 kN at Factored ULS and 800 kN at SLS.
2	Caissons – in the order of 1m diameter	- Routine equipment and installation techniques.	- Extensive space requirements for cranes, caisson machine, concrete installation for caissons and pile caps with probable off-shore work from barge. - Caisson installation introduces high risk of distress to foundations of existing structures from loss of ground. - Issues with managing spoil requiring environmentally acceptable procedures.	- Installation problem risk moderate. - Loss of ground risk high. - Vibration risk low.	- High cost, which would increase if there is off-shore work. - Estimated costs are $3000/m of installed length, each caisson to be approximately 20m in length and providing an estimated resistance of 4000 kN at Factored ULS and 3000 kN at SLS.

TABLE 3

Evaluation of Foundation Type Options

(applicable to either widening or replacement options)

Option #	Description	Advantages	Disadvantages	Relative Risk Level*	Relative Cost**
3	Auger cast piles - Procedures involve installing an approximately 300mm diameter hollow stem auger and filling the hole through the auger while simultaneously withdrawing the auger.	- Reduces vibration concern.	- Specialized equipment and installation techniques required. - Extensive space requirements for cranes, caisson machine, concrete installation for caissons and pile caps with probable off-shore work from barge. - Auger cast pile installation introduces moderate risk of distress to foundations of existing structures from loss of ground. - Issues with managing spoil requiring environmentally acceptable procedures.	- Installation problem risk moderate. - Loss of ground risk moderate. - Vibration risk very low.	- High cost due to probable off-shore work although augering can be done relatively quickly. - Consider an estimated resistance of 500 kN at Factored ULS and 350 kN at SLS per 20m long auger cast pile costing $500/m. - Requirement for pile pre-contract pile load testing to verify design, but which could potentially yield higher design capacities.

TABLE 3

Evaluation of Foundation Type Options

(applicable to either widening or replacement options)

Option #	Description	Advantages	Disadvantages	Relative Risk Level*	Relative Cost**
4	Micro piles - Procedures involve installing an approximately 300mm diameter tube, filling the tube with concrete, installing centralized rebar in and any additional reinforcing in unset concrete.	- Reduces space constraint issues. - Reduces loss of ground risk. - Eliminates vibration concerns. - More efficient foundation unit proof testing procedures.	- Specialized equipment and installation techniques used at only a few MTO projects previously. - Challenges with transporting concrete for installations with off-shore work. - Issues with managing spoil requiring environmentally acceptable procedures.	- Installation problem risk low. - Loss of ground risk very low. - Vibration risk very low.	- High cost due to probable off-shore work, relatively larger number of micro piles required and inherent risk of breakdown, low productivity and construction issues in micro pile installation. - Consider an estimated resistance of 750 kN at Factored ULS and 500 kN at SLS per 20m long micropiles costing $500/m. - Requirement for pre-contract pile load testing to verify design, but which could potentially yield higher design capacities. - The size of rebar in micropiles would have to be increased to permit pile load testing. - There is an opportunity for design micropiles with post grouting options to increase initial capacity.

***** Refer to Section 6 Preliminary Foundation Investigation and Design Report for details.

** Values are estimated presented for comparison and planning purposes only and are not be used in design.

REFERENCE: THIS FIGURE WAS PREPARED FROM THE GOOGLE MAP - MAPDATA @ 2014 GOOGLE

GEOCRES No. 30M5-301

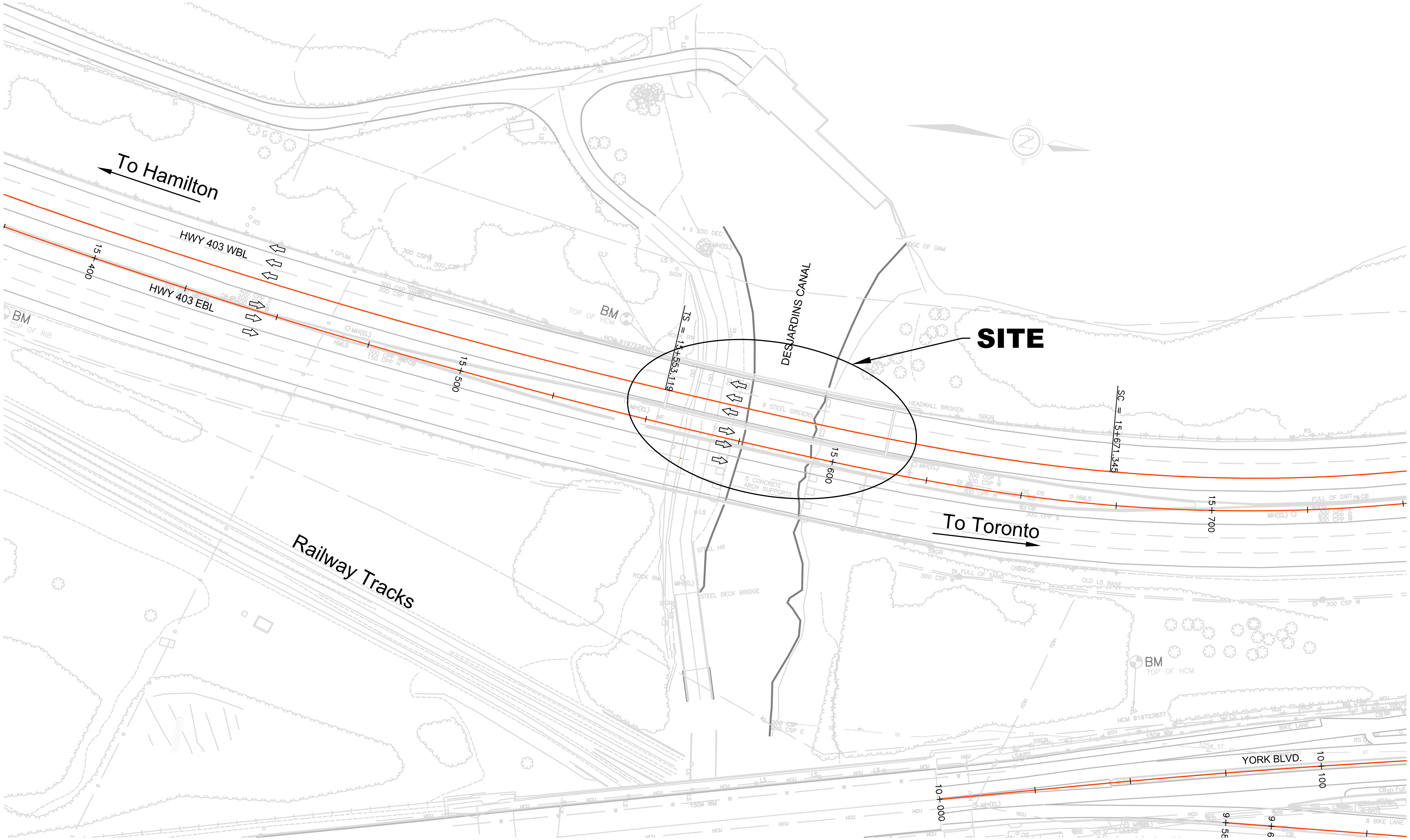
PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN

DESJARDINS CANAL BRIDGE (EBL AND WBL) HAMILTON, ONTARIO

KEY MAP

Peto MacCallum Ltd.
CONSULTING ENGINEERS

DRAWN:	N.A.	DATE	SCALE	JOB NO.	FIGURE NO.
CHECKED:	M.K.	MAY 2014	1 : 10,000	13TF017A	1
APPROVED:	C.N.				



GEOCRES No. 30M5-301

EXISTING CONDITIONS - SITE PLAN
HIGHWAY 403 / DESJARDINS CANAL BRIDGES
HAMILTON, ONTARIO

PLAN
SCALE

METRIC

Ontario

MORRISON HERSHFIELD

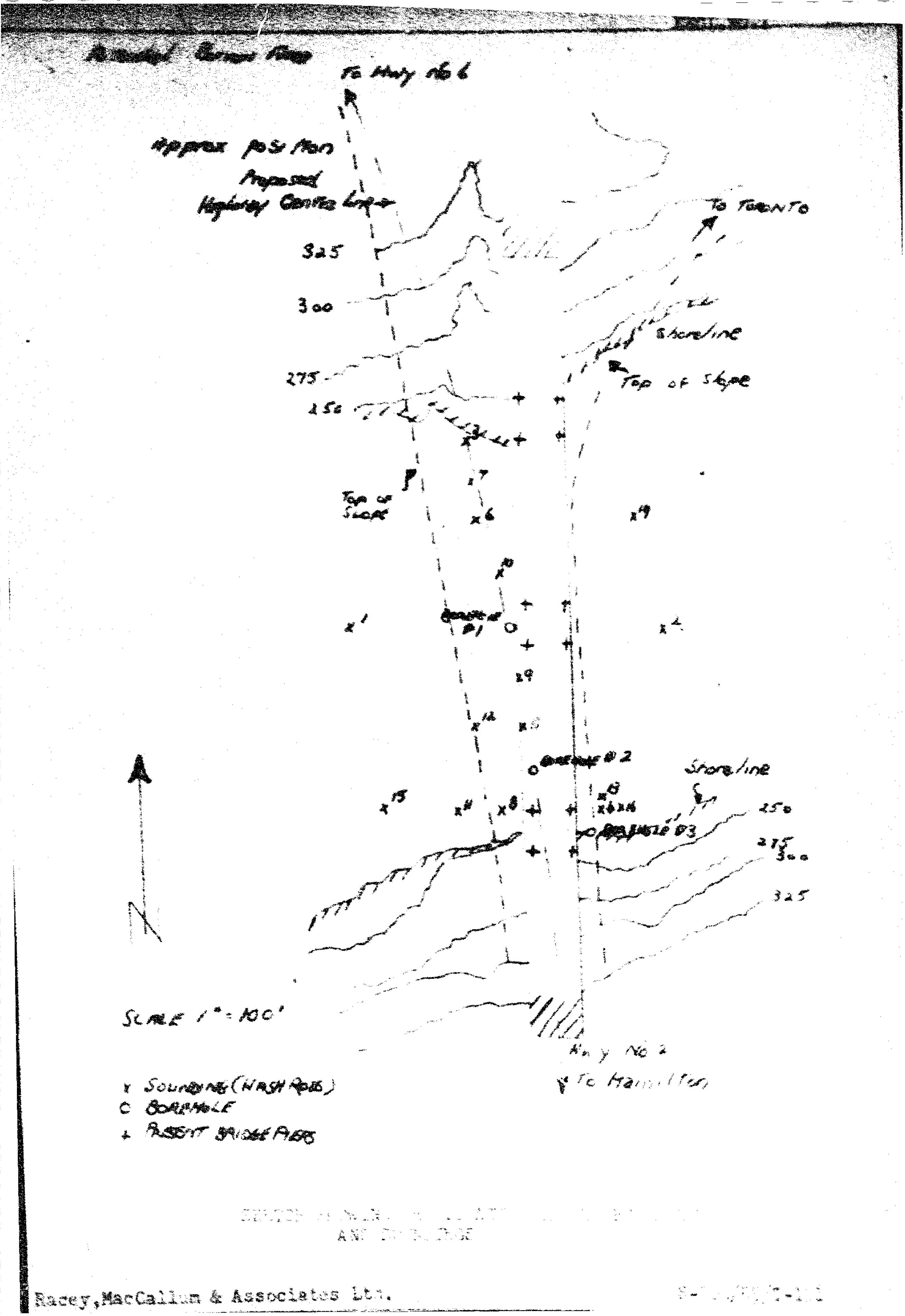
Peto MacCallum Ltd.
CONSULTING ENGINEERS

HIGHWAY 403
G.W.P. 2357-09-00

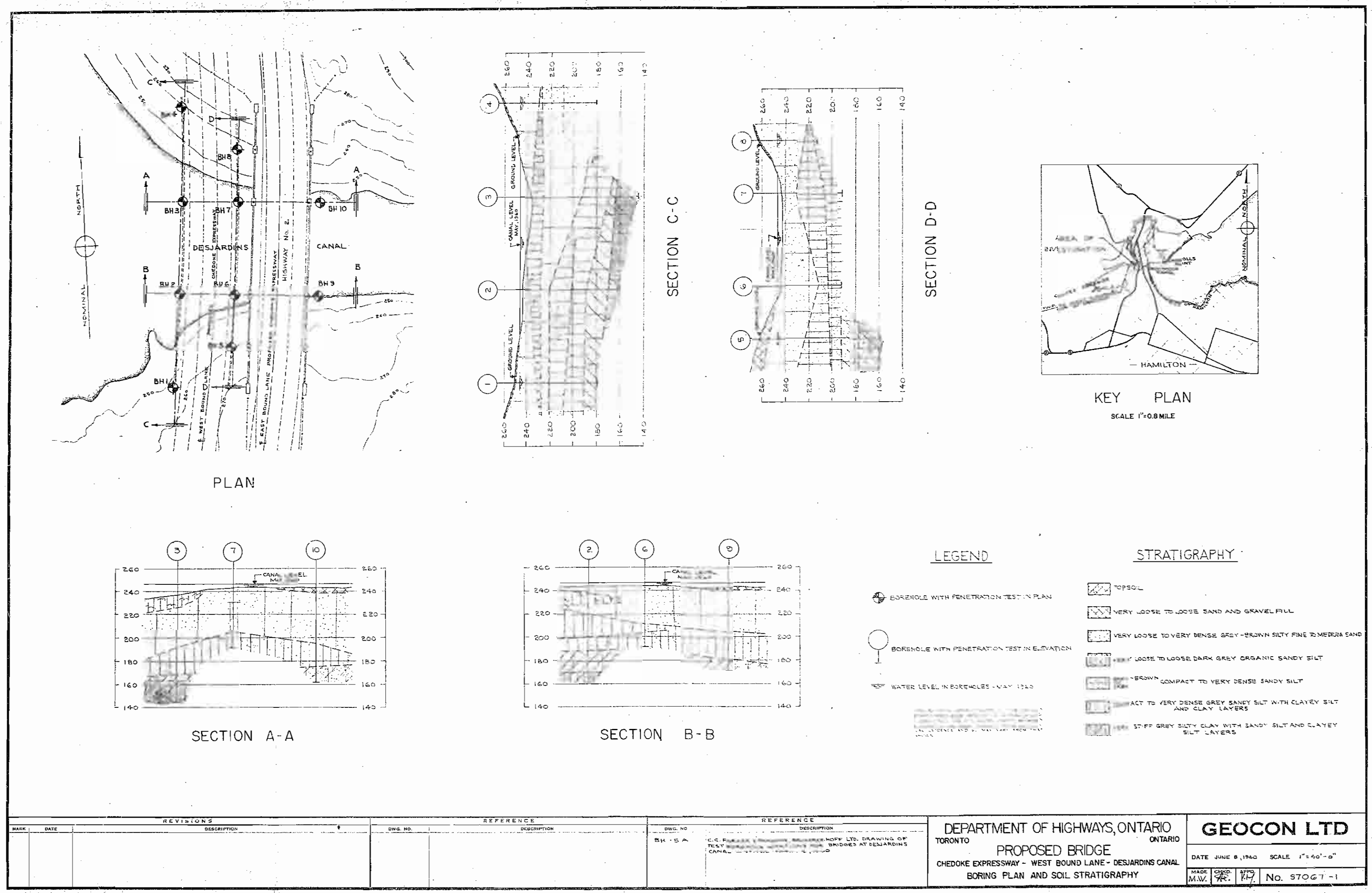
FIGURE
2

APPENDIX A

Borehole Locations and Soil Stratigraphy



Drawing A-1 Borehole Location Plan and Depth of Probes EBL Structure



Drawing A-2a Borehole Location Plan and Stratigraphical Profiles WBL Structure

Drawing A-2b Foundation Plan and Test Bore Results WBL Structure

APPENDIX B

Relevant Record of Borehole Sheets

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57067 BORING # 1 DATUM GEODETIC CASING BX
BORING DATE MAY 5, 1969 REPORT DATE MAY 14, 1969 COMPILED BY M.W. CHECKED BY
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE

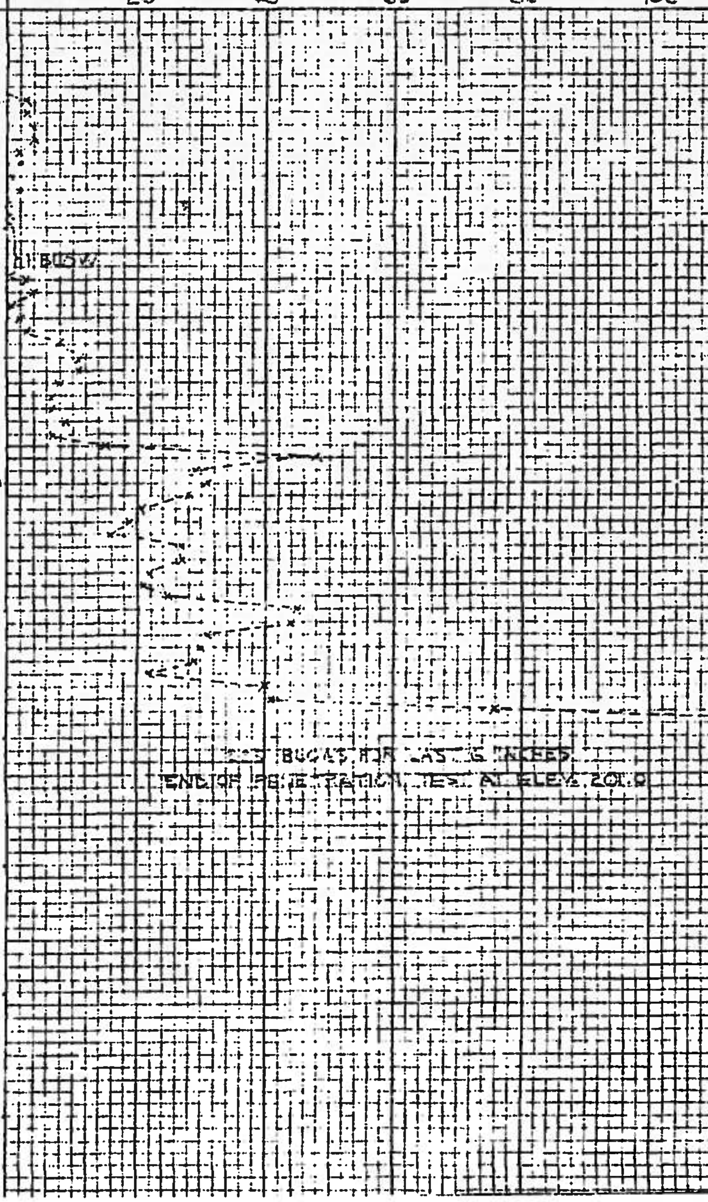
ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION
250.5		GROUND LEVEL
0.0		TOPSOIL
1.5		VERY LOOSE GREY-BROWN AND BROWN SILTY FINE SAND
231.0 19.5	W.L. @ EL. 246.5 MAY 1969	LOOSE DARK GREY ORGANIC SANDY SILT
227.5 23.0		VERY LOOSE BROWN SILTY FINE SAND
220.5 30.0		COMPACT TO DENSE GREY-BROWN SANDY SILT
206.5 44.0		DENSE TO VERY DENSE GREY SANDY SILT
190.5 60.0		COMPACT GREY SANDY SILT WITH CLAYEY SILT AND CLAY LAYERS
178.5 72.0		END OF HOLE

STRAT. PLOT

ELEVATION SCALE: 250, 240, 230, 220, 210, 200, 190, 180, 170

WATER CONTENT Ws O NAT. □ Lw △ Pw

DYNAMIC PENETRATION TEST BLOWS PER FOOT 20 40 60 80 100



OTHER TESTS / **SAMPLES**

OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
		2" DO	1	1
M		DO	2	MANUAL PUSH
M		DO	3	1
		DO	4	8
		DO	5	2
		DF	6	—
M		DF	7	24
		DF	8	14
		DF	9	34
		DF	10	34
		DF	11	62
		DF	12	>100
		DF	13	18
		DF	14	23
		DF	15	19

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7Q67 BORING # 2 DATUM GEODETIC CASING BX
BORING DATE APRIL 2, 22, 1960 REPORT DATE MAY 13, 1960 COMPILED BY M.W. CHECKED BY
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES
- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

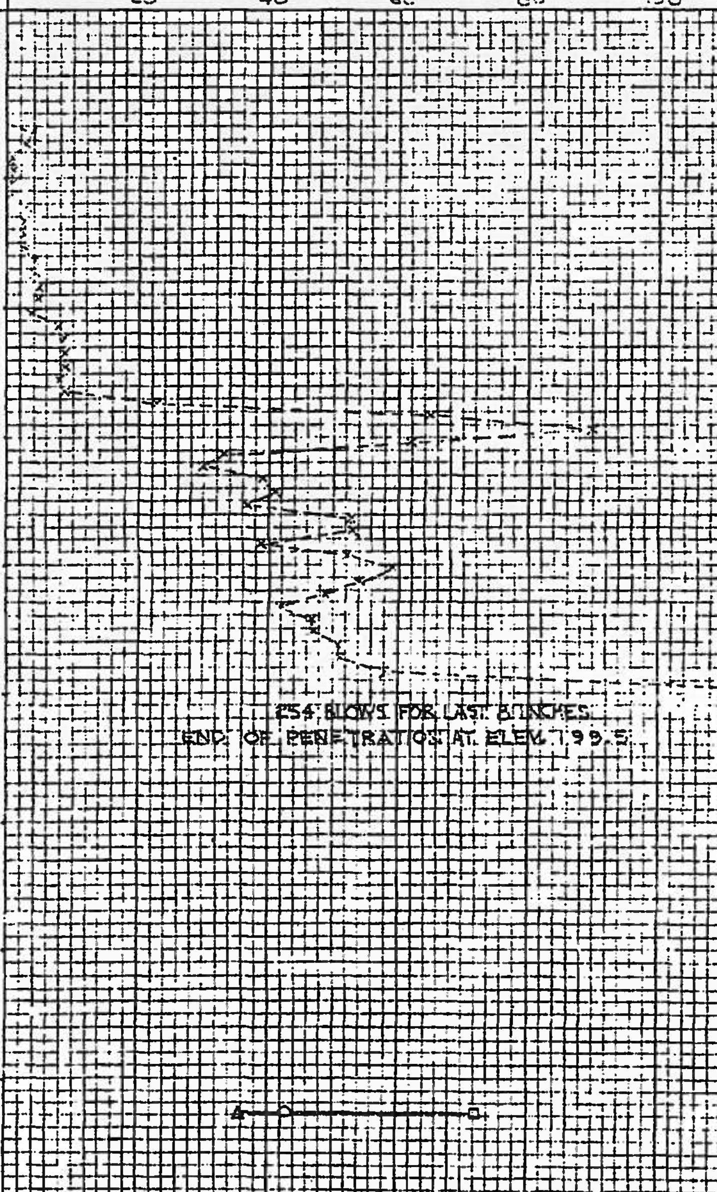
ABBREVIATIONS
- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
246.5 / 0.0		CANAL LEVEL
244.2 / 2.3		CANAL BOTTOM
238.2 / 8.3		VERY LOOSE GREY-BROWN SILTY FINE SAND
228.2 / 18.3		VERY LOOSE DARK GREY ORGANIC SANDY SILT
220.2 / 26.3		LOOSE TO COMPACT GREY-BROWN FINE SAND
200.2 / 46.3		COMPACT GREY-BROWN SANDY SILT
189.2 / 57.3		VERY DENSE GREY SANDY SILT
170.2 / 76.3		COMPACT TO DENSE GREY SANDY SILT WITH CLAYEY SILT AND CLAY LAYERS
161.7 / 84.8		VERY STIFF GREY SILTY CLAY WITH SANDY SILT AND CLAYEY SILT LAYERS
		END OF HOLE

STRAT. PLOT — ELEVATION SCALE: 250, 240, 230, 220, 210, 200, 190, 180, 170, 160

WATER CONTENT W% — O NAT. □ LW Δ PW — 10, 20, 30, 40, 50
DYNAMIC PENETRATION TEST BLOWS PER FOOT — 20, 40, 60, 80, 100



SAMPLES

OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
		DF	1	1
		DF	2	1
		DF	3	4
M		DF	4	29
M		DF	5	27
M		DF	6	28
		DF	7	24
		DF	8	29
		DF	9	66
		DF	10	83
		SO	11	44
		SO	12	34
		SO	13	44
		SO	14	47
		SO	15	26
		SO	16	39

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57067 BORING # 3 DATUM GEODETIC CASING BX
BORING DATE APRIL 27 & 29, 1960 REPORT DATE MAY 4, 1960 COMPILED BY M.W. CHECKED BY
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc. - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

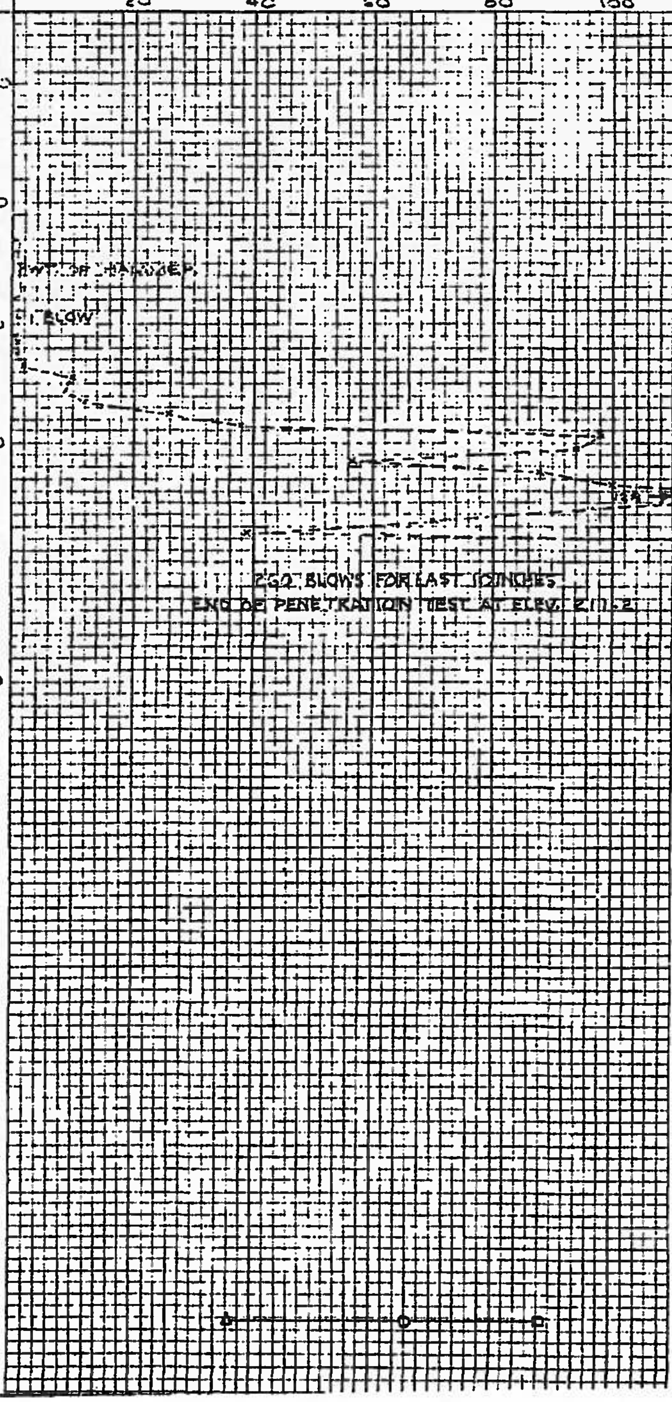
SOIL PROFILE

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
246.5 / 0.0		CANAL LEVEL
237.0 / 9.5		CANAL BOTTOM
		VERY LOOSE DARK GREY ORGANIC SANDY SILT
226.5 / 20.0		DENSE GREY-BROWN SILTY FINE TO MEDIUM SAND
192.0 / 55.5		DENSE GREY SANDY SILT
169.0 / 77.5		DENSE GREY SANDY SILT WITH CLAYEY SILT AND CLAY LAYERS
163.0 / 83.5		VERY STIFF GREY SILTY CLAY WITH SANDY SILT AND CLAYEY SILT LAYERS
145.0 / 101.5		END OF HOLE

ELEVATION SCALE: 250, 240, 230, 220, 210, 200, 190, 180, 170, 160, 150

WATER CONTENT Ws ○ NAT. ⊐ Lw △ Pw — 10, 20, 30, 40, 50

DYNAMIC PENETRATION TEST BLOWS PER FOOT — 20, 40, 60, 80, 100



SAMPLES

OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
		WS	1	—
M		2"DO	2	13
M		DO	3	54
M		DO	4	42
M		DO	5	>100
		DO	6	42
		DO	7	>100
		DO	8	36
		DO	9	36
		DO	10	36
		DO	11	34
		DF	12	—
		DF	13	31
		SO	14	38
		SO	15	34
		DF	16	—
		SO	17	31
		SO	18	25
		SO	19	18
		SO	20	20

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7067 BORING # 4 DATUM GEODETIC CASING BX

BORING DATE APRIL 12 to 16, 1960 REPORT DATE MAY 13, 1960 COMPILED BY M.N.V. CHECKED BY [illegible]

SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

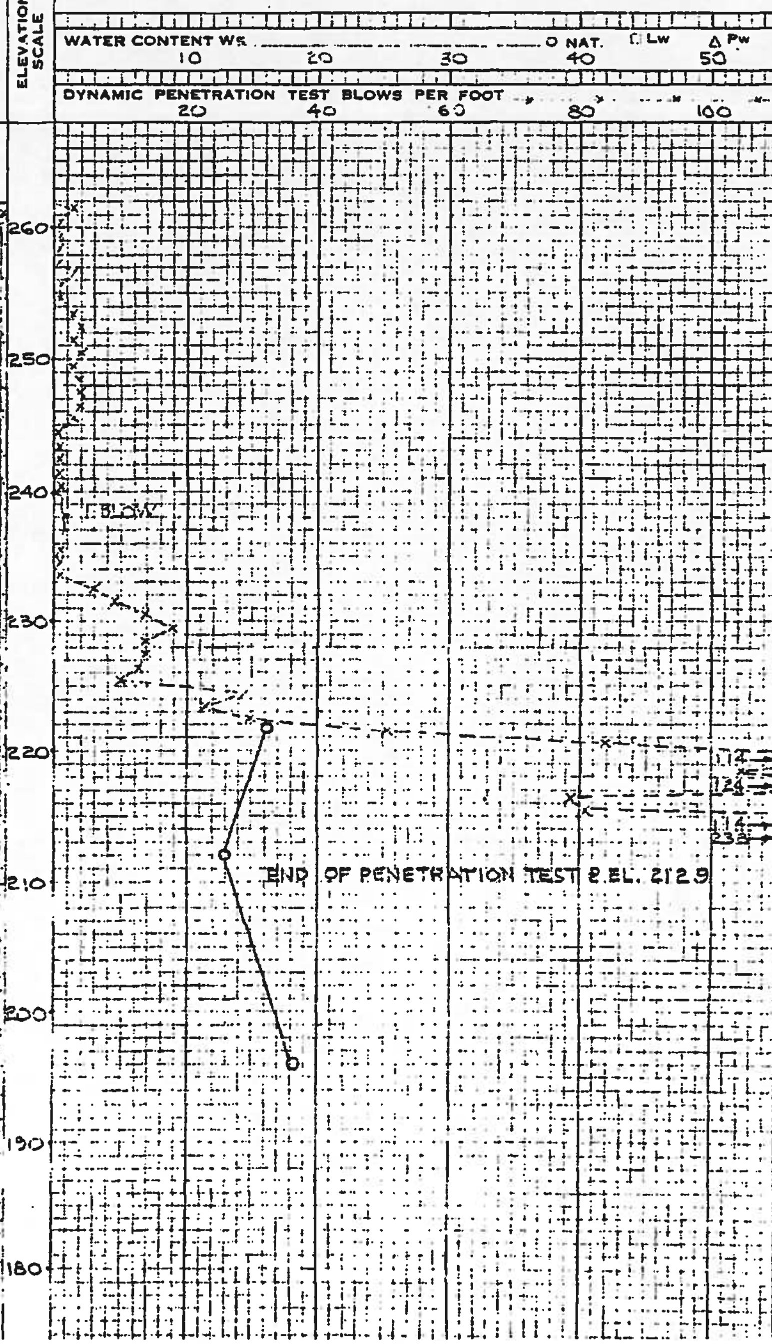
- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc. - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
261.8 / 0.0		GROUND LEVEL
1.5		TOP SOIL
	W.L. @ EL. 246.5 MAY, 1960	VERY LOOSE TO LOOSE GREY-BROWN & BROWN FINE TO MEDIUM SAND
230.8 / 31.0		COMPACT DARK GREY ORGANIC SANDY SILT WITH COARSE SAND LAYERS
33.0		LOOSE TO VERY DENSE GREY-BROWN SILTY FINE SAND
209.8 / 52.0		DENSE TO VERY DENSE GREY SILTY FINE SAND
180.4 / 81.5		END OF HOLE



OTHER TESTS

- M (approx. EL. 242)
- M (approx. EL. 228)
- M, M, γ=133 (approx. EL. 223)
- γ=125 (approx. EL. 212)
- M, γ=131 (approx. EL. 196)

SAMPLES

TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
2" DO	1	3
DO	2	6
DF	3	—
DO	4	5
DF	5	1
DF	6	1
WS	7	—
2" DF	8	28
DO	9	5
SO	10	43
SO	11	66
SO	12	>100
SO	13	>100
SO	14	—
DF	15	—
SO	16	>100
DO	17	42
SO	18	31
SO	19	50
DF	20	

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57067 BORING # 5 DATUM GEODETIC CASING 5×
BORING DATE APRIL 7 to 12, 1960 REPORT DATE MAY 14, 1960 COMPILED BY M.W. CHECKED BY
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

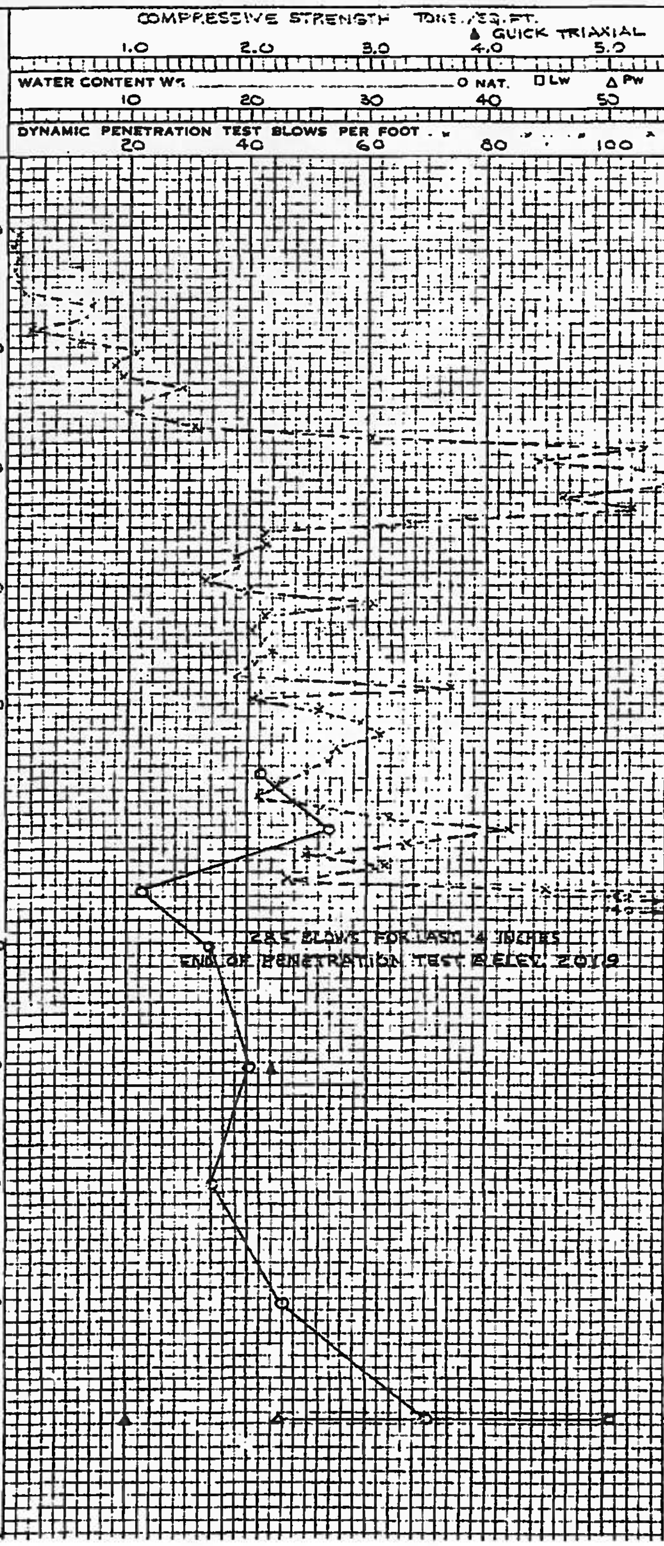
SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES
- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS
- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- QC - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEV'N / DEPTH	WATER CONDITIONS	DESCRIPTION
260.2		GROUND LEVEL
0.0		LOOSE BROWN SAND AND GRAVEL FILL
251.7 / 8.5	W.L. @ EL. 246.5 MAY 1960	COMPACT TO VERY DENSE GREY-BROWN AND BROWN FINE TO MEDIUM SAND
222.2 / 38.0		DENSE GREY-BROWN SANDY SILT
201.2 / 59.0		VERY DENSE GREY SANDY SILT
191.2 / 69.0		COMPACT TO DENSE GREY SANDY SILT WITH CLAYEY SILT AND CLAY LAYERS
176.2 / 84.0		VERY STIFF GREY SILTY CLAY WITH SANDY SILT AND CLAYEY SILT LAYERS
158.2 / 102.0		END OF HOLE



SAMPLES

OTHER TESTS	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	2" DO	1	10
	DO	2	19
M	DO	3	33
	DO	4	51
	DO	5	54
M	DO	6	34
	SO / DF	7 / 8	50 / —
	DO	9	40
γ=133	SO	10	37
M γ=122	SO	11	33
	SO	12	32
M γ=133	SO	13	74
	SO / DF	14 / 15	>100 / —
γ=129	SO	16	29
	SO	17	39
	SO	18	20
	SO	19	20
	SO	20	36
	SO	21	30
γ=117	SO	22	22

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S70.67 BORING # 6 DATUM GEODETIC CASING BX
BORING DATE APRIL 22 & 25, 1960 REPORT DATE MAY 18, 1960 COMPILED BY M.W. CHECKED BY
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

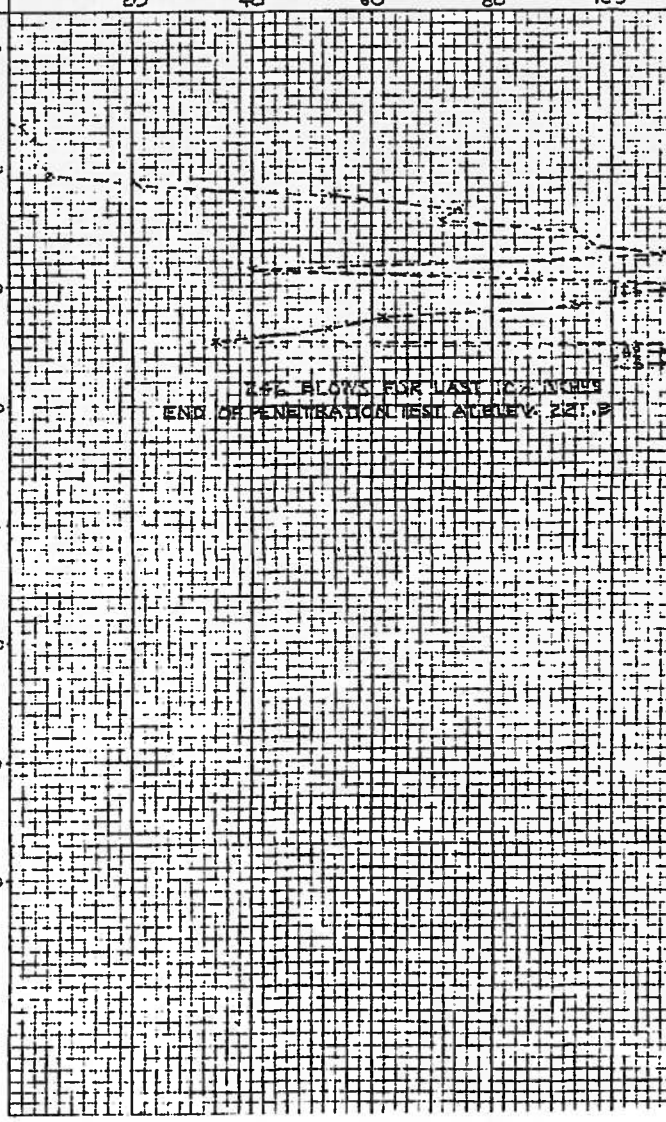
SAMPLE TYPES
- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS
- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT
246.5 / 0.0		CANAL LEVEL	
243.8 / 2.7		CANAL BOTTOM	
		VERY DENSE GREY-BROWN AND BROWN SILTY FINE TO MEDIUM SAND	
219.8 / 55.7		DENSE GREY-BROWN SANDY SILT	
203.8 / 42.7		VERY DENSE GREY SANDY SILT	
191.8 / 54.7		END OF HOLE	

WATER CONTENT W% ○ NAT. □ LW △ PW
DYNAMIC PENETRATION TEST BLOWS PER FOOT



SAMPLES

OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
M		2" DF	1	69
		DO	2	77
M		DO	3	51
		DO	4	57
		DO	5	51
M		DO	6	62
		DO	7	29
		DO	8	46
		DO	9	54
		DO	10	73

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57067 BORING # 7 DATUM GEODETIC CASING BX
BORING DATE APRIL 26 & 27, 1960 REPORT DATE MAY 13, 1960 COMPILED BY M.M. CHECKED BY [illegible]
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES
- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS
- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

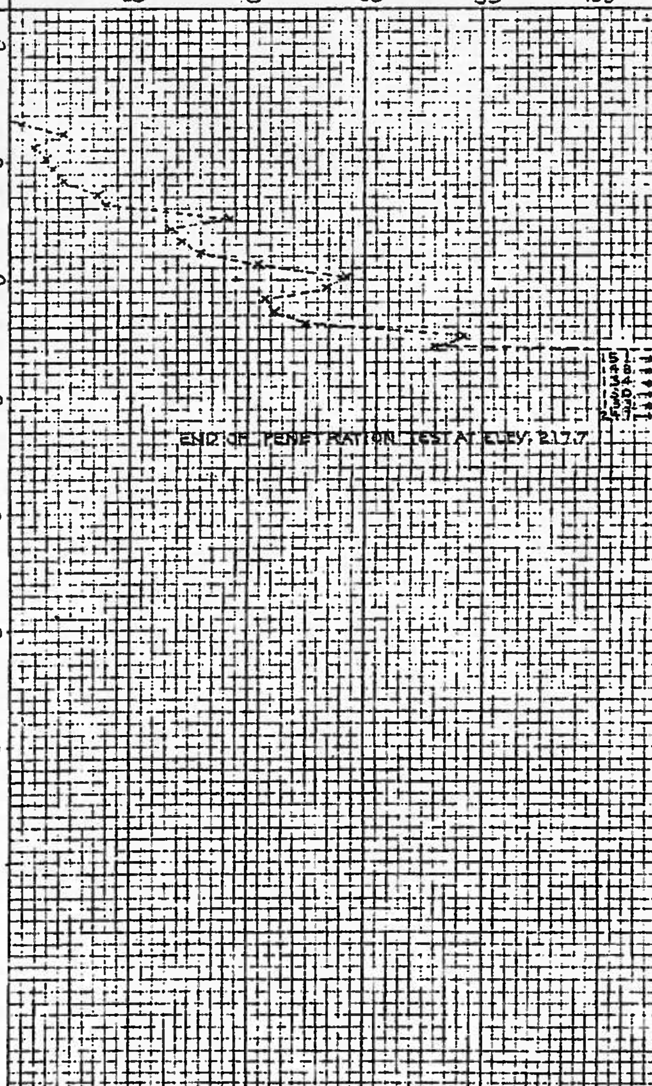
SOIL PROFILE

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION
246.5 / 0.0		CANAL LEVEL
243.7 / 2.8		CANAL BOTTOM
230.7 / 15.8		COMPACT GREY-BROWN FINE TO MEDIUM SAND
205.7 / 40.8		DENSE TO VERY DENSE GREY-BROWN SANDY SILT
191.7 / 54.8		DENSE GREY SANDY SILT
		END OF HOLE

ELEVATION SCALE: 250, 240, 230, 220, 210, 200, 190

WATER CONTENT W% — O NAT. □ Lw △ Pw

DYNAMIC PENETRATION TEST BLOWS PER FOOT: 20, 40, 60, 80, 100



SAMPLES

OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
M		2"DO	1	20
M M		DO	2	27
M		DO	3	47
M		DO	4	>100
M		DO	5	72
		DO	6	81
		DO	7	>100
		DO	8	43
		DO	9	3[illegible]
		DO	10	35

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57067 BORING # 8 DATUM GEODETIC CASING BX
BORING DATE APRIL 19 & 20, 1960 REPORT DATE MAY 13, 1960 COMPILED BY M.W. CHECKED BY
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

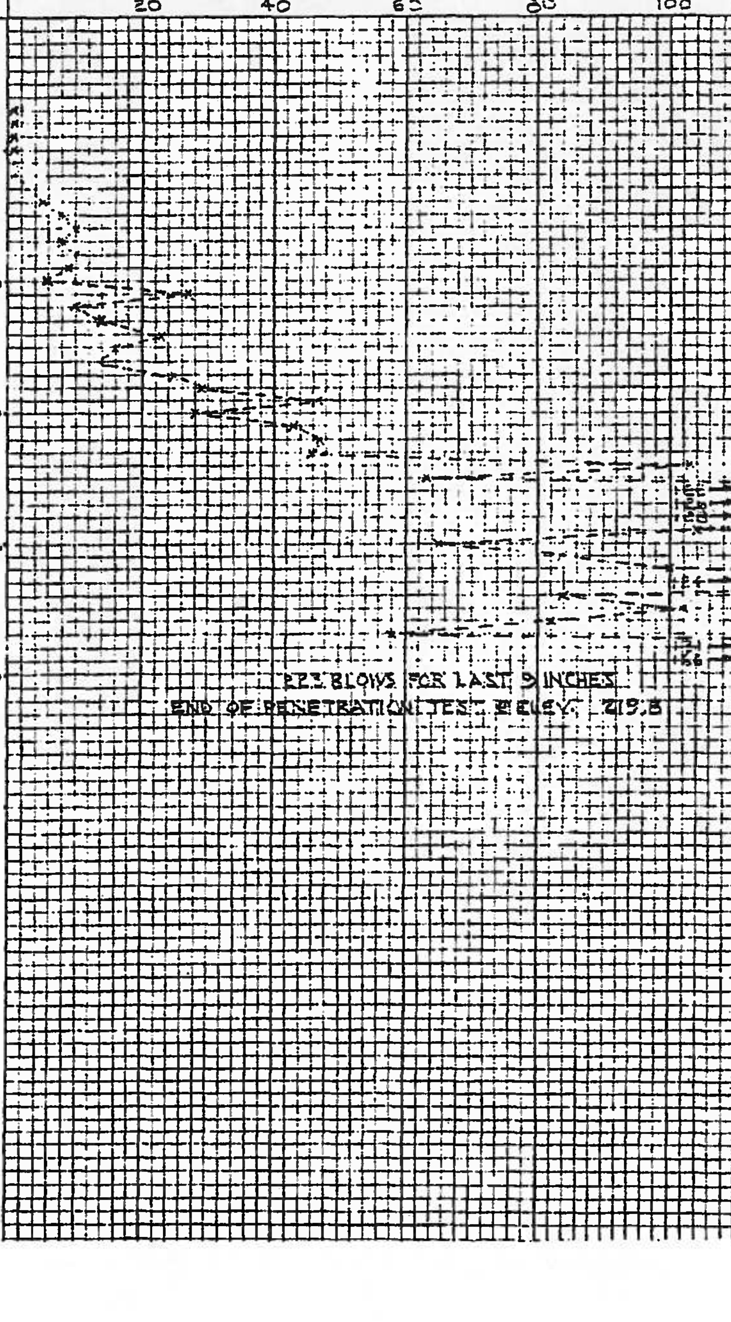
SOIL PROFILE

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
263.8 / 0.0		GROUND LEVEL
	W.L. @ EL. 246.5 MAY, 1960	LOOSE TO VERY DENSE GREY-BROWN AND BROWN SILTY FINE SAND
222.8 / 41.0		DENSE TO VERY DENSE GREY-BROWN SANDY SILT
215.8 / 48.0		VERY DENSE GREY SANDY SILT
212.6 / 51.2		END OF HOLE

ELEVATION SCALE: 260, 250, 240, 230, 220, 210

WATER CONTENT W% ——— O NAT. □ Lw △ Pw

DYNAMIC PENETRATION TEST BLOWS PER FOOT 20 40 60 80 100



OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	2" DO	1	4
	DO	2	14
	DO	3	11
	DO	4	26
	DO	5	42
	DO	6	69
	DO	7	68
	DO	8	45
	SO	9	>100
	DF	10	—
	DO	11	[illegible]
	DF	12	—

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7067 BORING # 9 DATUM GEODETIC CASING BX

BORING DATE MAY 3 & 4, 1960 REPORT DATE MAY 14, 1960 COMPILED BY M.W. CHECKED BY [illegible]

SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

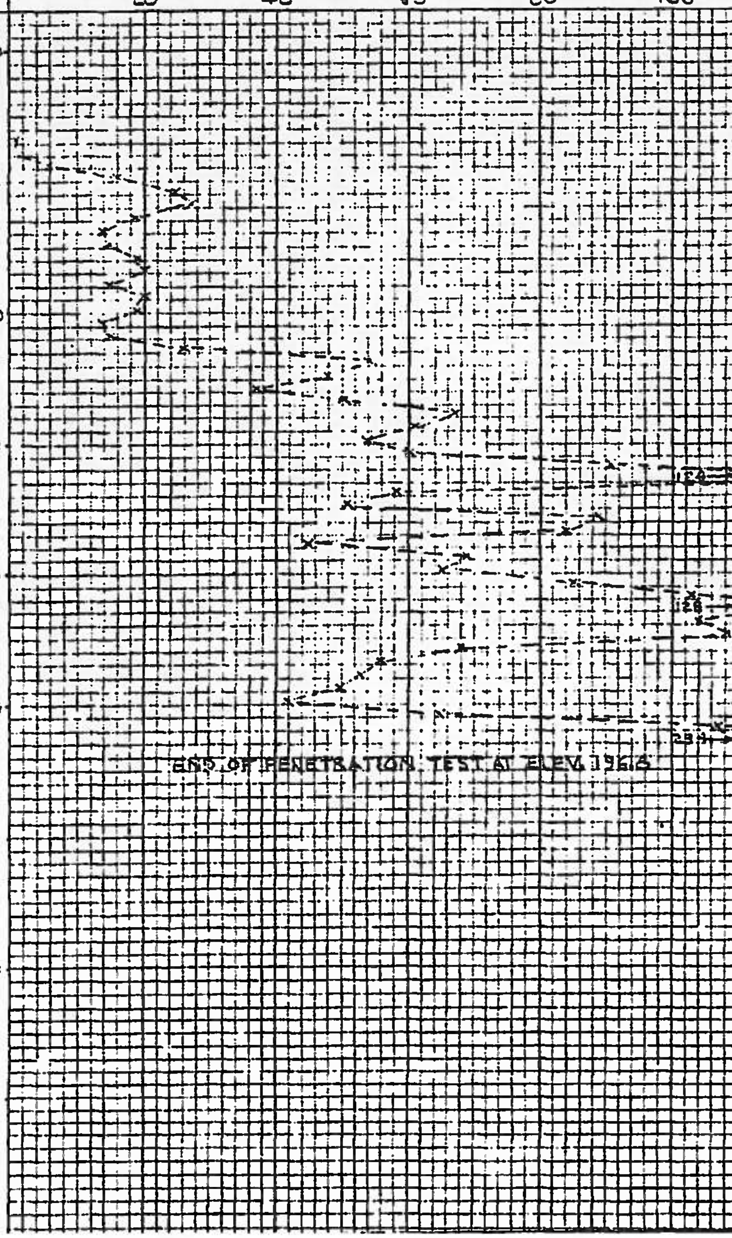
SOIL PROFILE

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
246.5 / 0.0		CANAL LEVEL
243.8 / 2.7		CANAL BOTTOM
		VERY LOOSE BROWN SAND AND GRAVEL FILL
240.3 / 6.2		COMPACT TO DENSE GREY-BROWN AND BROWN SILTY FINE SAND
204.8 / 41.7		COMPACT TO VERY DENSE GREY SANDY SILT
180.8 / 65.7		DENSE TO VERY DENSE GREY SANDY SILT WITH CLAYEY SILT AND CLAY LAYERS
171.8 / 74.7		END OF HOLE

ELEVATION SCALE: 250, 240, 230, 220, 210, 200, 190, 180, 170

WATER CONTENT W% ○ NAT. □ Lw △ Pw

DYNAMIC PENETRATION TEST BLOWS PER FOOT 20 40 60 80 100



OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	2" DO	1	27
	DO	2	21
	DO	3	17
	DO	4	39
	DO	5	42
	DO	6	30
	DO	7	34
	DO	8	23
	DO	9	69
	DO	10	>100
	DO	11	53
	DO	12	36
	DO	13	33
	DO	14	50

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7067 BORING # 10 DATUM GEODETIC CASING EX
BORING DATE REPORT DATE MAY 14 1960 COMPILED BY M.W. CHECKED BY
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

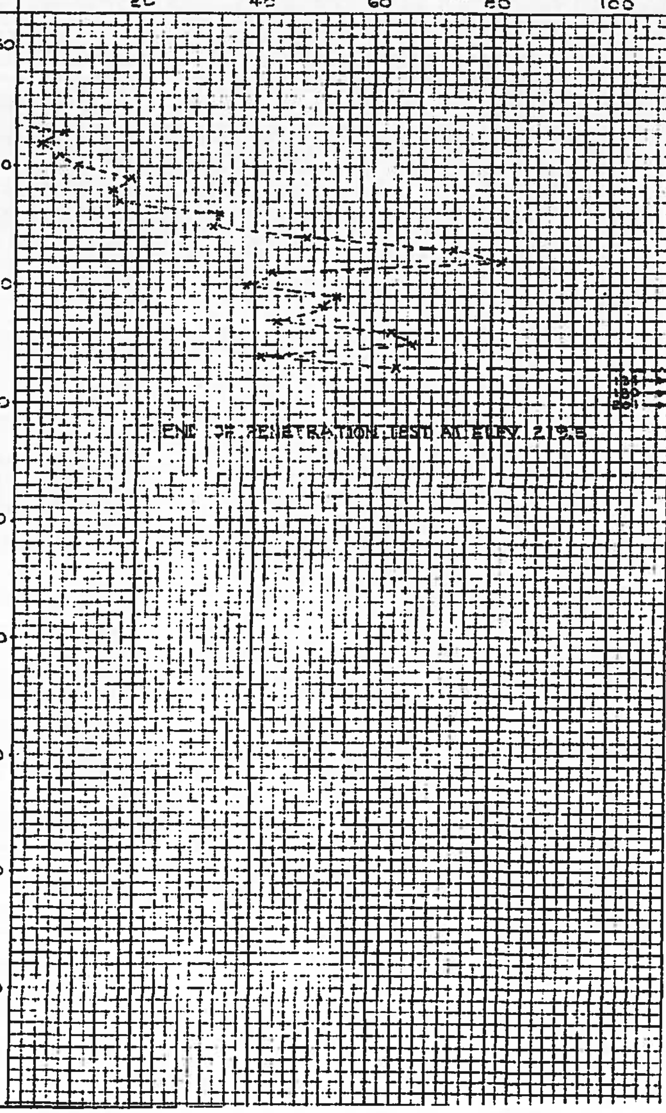
SOIL PROFILE

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
246.5 / 0.0		CANAL LEVEL
243.5 / 3.0		CANAL BOTTOM
239.5 / 7.0		VERY LOOSE TO LOOSE BROWN SAND AND GRAVEL FILL
195.5 / 51.0		COMPACT TO VERY DENSE GREY-BROWN AND BROWN SILTY FINE SAND
175.5 / 71.0		DENSE TO VERY DENSE GREY SANDY SILT
162.0 / 84.5		DENSE TO VERY DENSE GREY SANDY SILT WITH CLAYEY SILT AND CLAY LAYERS
		END OF HOLE

ELEVATION SCALE: 250, 240, 230, 220, 210, 200, 190, 180, 170, 160

WATER CONTENT Ws ○ NAT. □ Lw △ Pw

DYNAMIC PENETRATION TEST BLOWS PER FOOT 20 40 60 80 100



OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	2"DO	1	21
	DO	2	14
	DO	3	50
	DO	4	44
	DO	5	41
	DO	6	34
	DO	7	64
	DO	8	>100
	DO	9	23
	DO	10	52
	DO	11	>100
	DF	12	–
	DF	13	66
	SO	14	36
	SO	15	44
	SO	16	31
	SO	17	61

APPENDIX C

Site Photographs from Field Reconnaissance

Photograph 1: WBL Bridge, West Face.

Photograph 2: WBL Bridge, Deck Rehabilitation.

Photograph 3: WBL Bridge, Deck Rehabilitation.

Photograph 4: WBL Bridge.

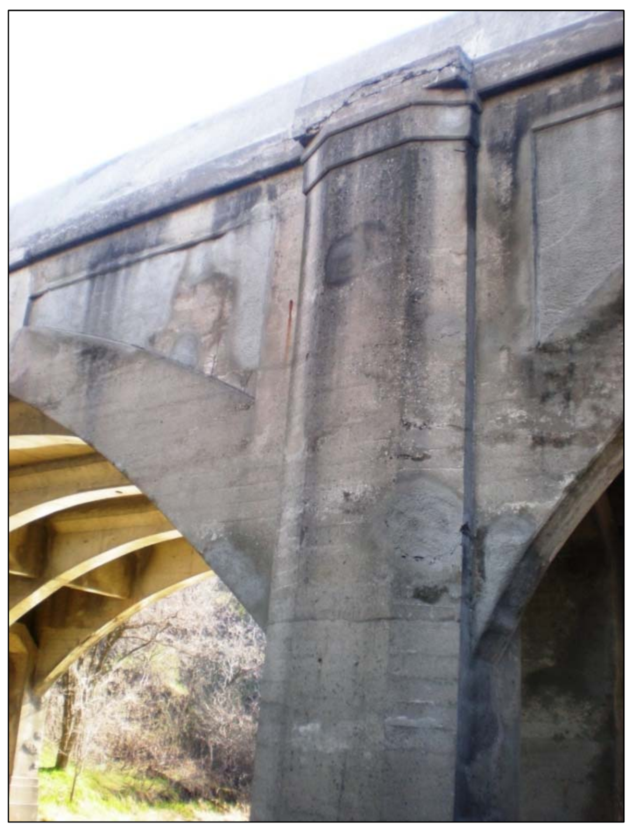
Photograph 5: WBL Bridge, East face.

Photograph 6: EBL Bridge, Looking North.

Photograph 7: EBL Bridge, South West Corner.

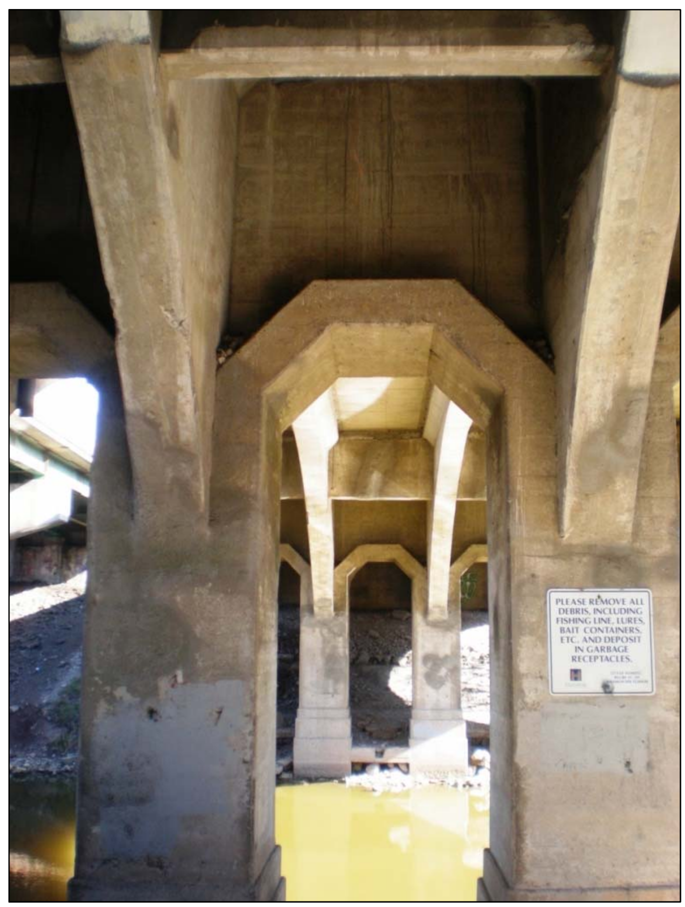
Photograph 8: EBL Bridge, Looking West on Trail.

Photograph 9: EBL Bridge, Looking East on Trail.



Photograph 10: EBL Bridge, Looking East on Trail.

Photograph 11: EBL Bridge, South West Corner.



Photograph 12: EBL Bridge, Looking North.

Photograph 13: EBL Bridge, Bird Nest.

Photograph 14: EBL Bridge, Bird Nest.

Photograph 15: EBL Bridge, Looking North.

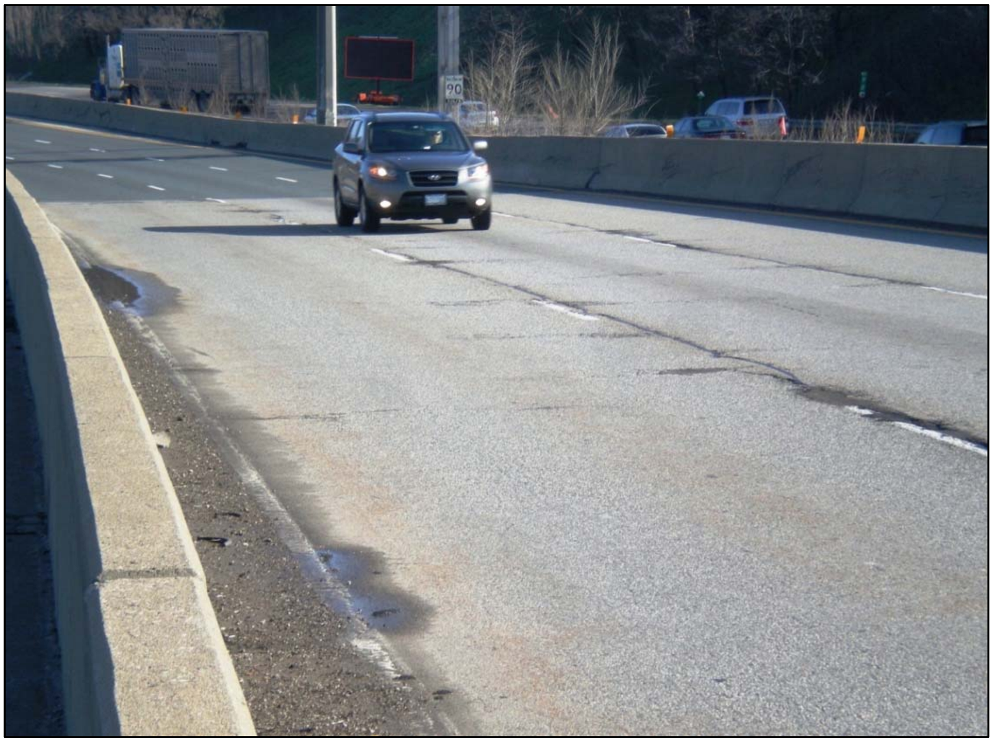
Photograph 16: EBL Bridge, Looking North.

Photograph 17: EBL Bridge, Looking South.

Photograph 18: EBL Bridge, Looking West.

Photograph 19: EBL Bridge, Looking West.

Photograph 20: WBL Bridge looking North.



Photograph 21: WBL Bridge looking North.

Photograph 22: WBL Bridge, Looking North.

Photograph 23: WBL Bridge, South Joint.

APPENDIX D

General Arrangement of EBL Structure

and

General Arrangement of WBL Structure

and

WBL Pier and Abutment Foundation Details

and

Sections Showing Proximity of Structures and Foundations

and

Minimum Clearance for New Deep Foundations Adjacent to WBL Piers

DIST No 4
CONT No 85-68
WP No 229-77-04

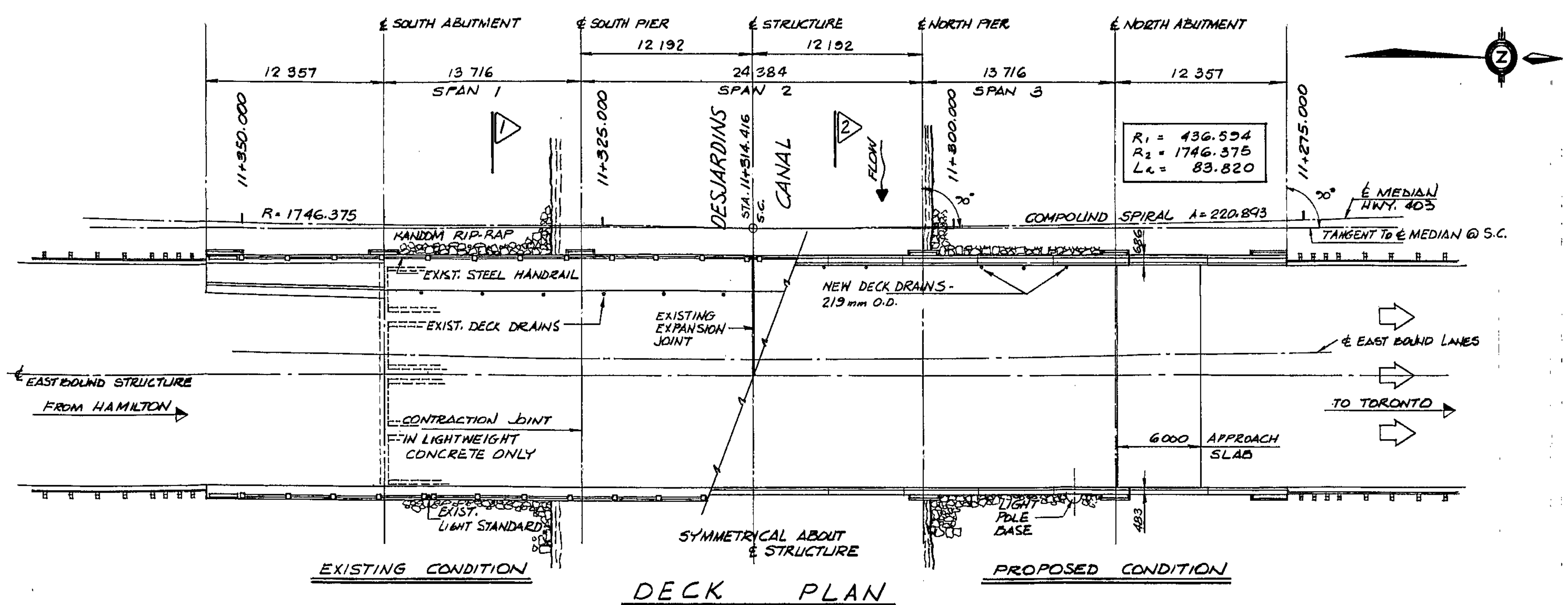
HWY. 403 EAST BOUND LANES OVER DESJARDINS CANAL BRIDGE DECK REHABILITATION
GENERAL ARRANGEMENT

SHEET 41

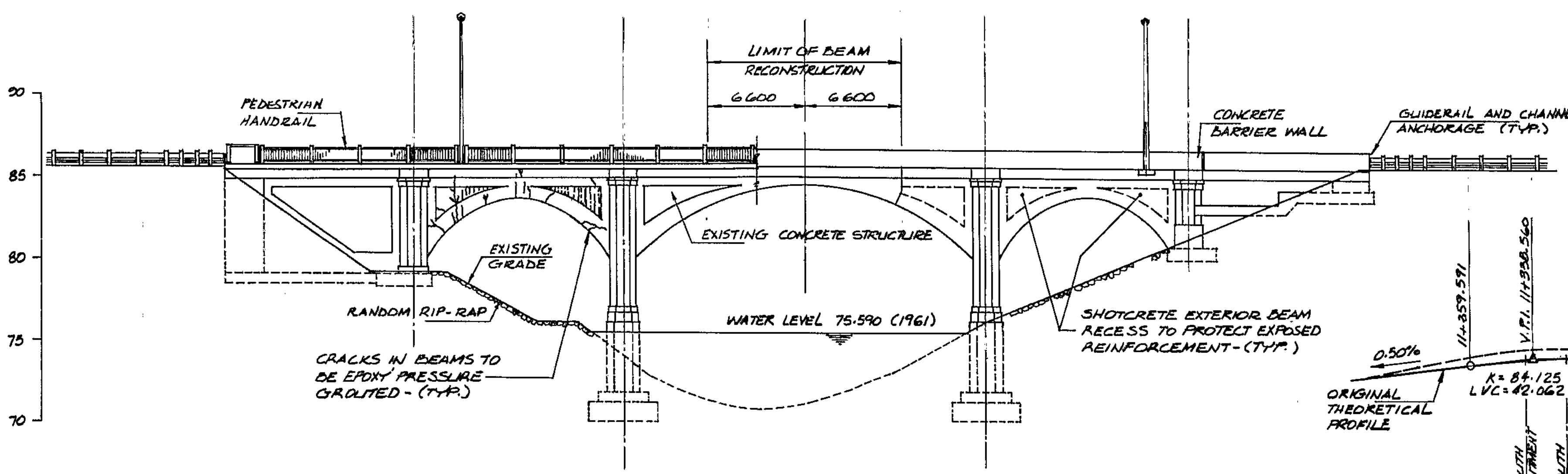
Giffels — Giffels Associates Limited Consulting Engineers

METRIC

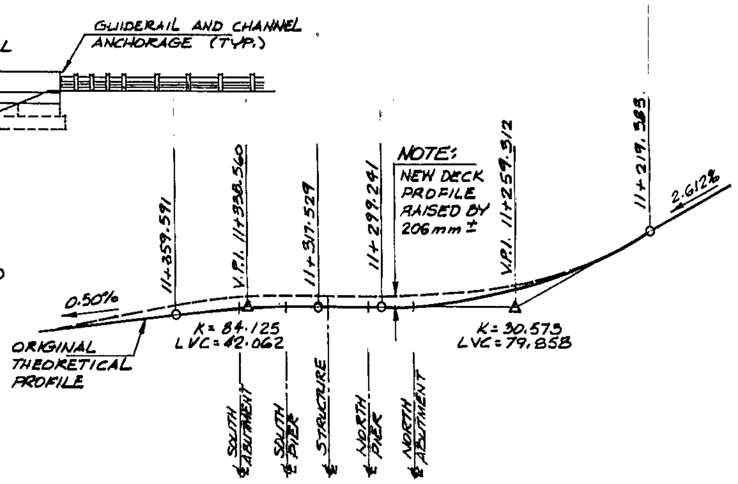
DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE SHOWN.
ELEVATIONS, COORDINATES, CURVE AND ALIGNMENT DATA ARE IN METRES.
STATIONS ARE IN KILOMETRES + METRES.



DECK PLAN
SCALE 1:200



EAST ELEVATION
SCALE 1:200



PROFILE - HWY 403 @ ℄ E.B. LANES

LIST OF DRAWINGS:

GENERAL NOTES

CLASS OF CONCRETE

BEAMS, DECK SLAB, ABUTMENTS, BARRIER WALLS — 30 MPa
APPROACH SLABS — 20 MPa

CLEAR COVER TO REINFORCING STEEL

DECK TOP — 70 ± 20
BOTTOM — 40 ± 20
REMAINDER 70 ± 20 OR AS NOTED

REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400, BARS MARKED WITH SUFFIX C SHALL BE EPOXY COATED BARS
REINFORCING DOWELS SET INTO EXISTING CONCRETE USING NON SHRINK GROUT.

FOR DETAILS OF EXISTING BRIDGE REFER TO THE EXISTING BRIDGE DRAWINGS.
THE CONTRACTOR SHALL FIELD CHECK AND VERIFY ALL CONDITIONS AND MEASUREMENTS AT THE SITE AND REPORT ANY DISCREPANCIES TO THE ENGINEER BEFORE PROCEEDING WITH THE WORK.

WORK DESCRIPTION

REMOVALS

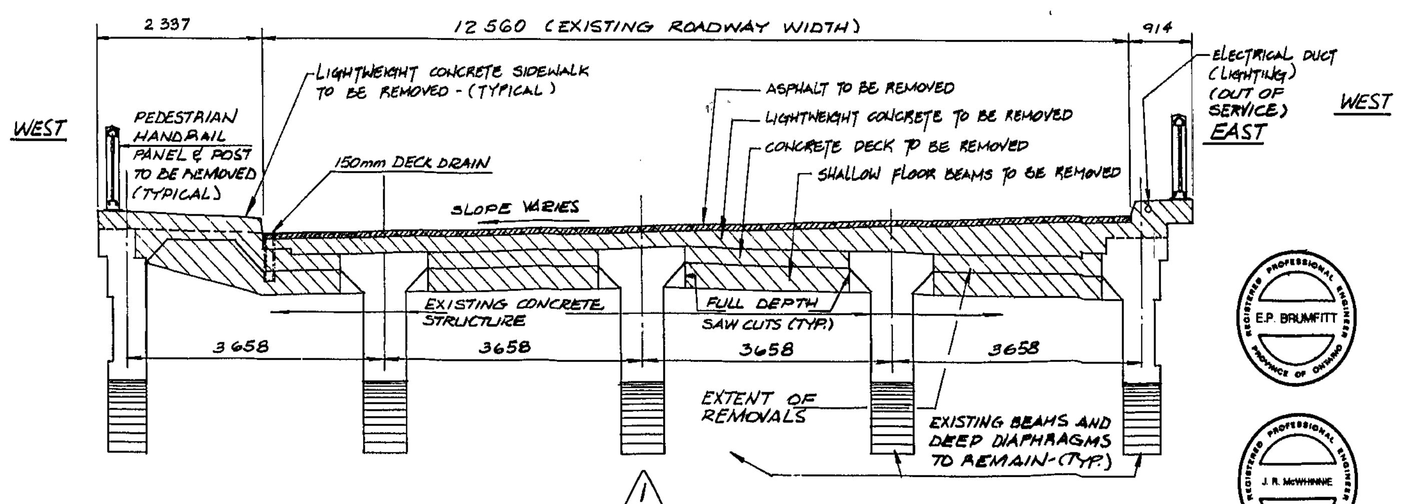
1. LIGHT STANDARDS, HANDRAIL PANELS AND POSTS.
2. ASPHALT AND CONCRETE DECK INCLUDING CURBS, SIDEWALK AND SHALLOW FLOOR BEAMS.
3. CONCRETE END POSTS, CURB AND SIDEWALK ON WINGWALLS
4. BEAMS 6.60 m EACH SIDE OF ℄ OF STRUCTURE.

CONSTRUCTION

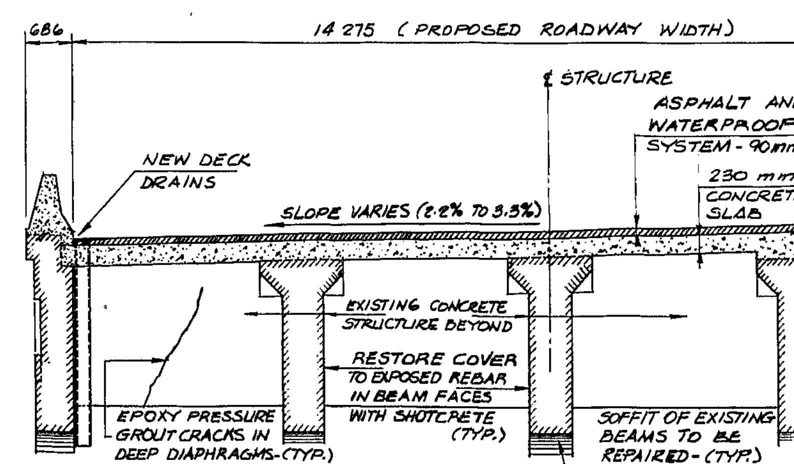
1. RE-CONSTRUCT BEAMS TO PROVIDE CONTINUITY.
2. NEW DECK, DECK DRAINS AND BARRIER WALLS.
3. NEW LIGHT STANDARDS AND EMBEDDED ELECTRICAL WORK.
4. ASPHALT AND WATERPROOFING SYSTEM.

REPAIRS

1. PRESSURE GROUT BEAM AND DEEP DIAPHRAGM CRACKS.
2. APPLY LATEX MODIFIED SHOTCRETE



CROSS-SECTION EXISTING BRIDGE
SCALE 1:50



CROSS-SECTION PROPOSED BRIDGE RECONSTRUCTION
SCALE 1:50

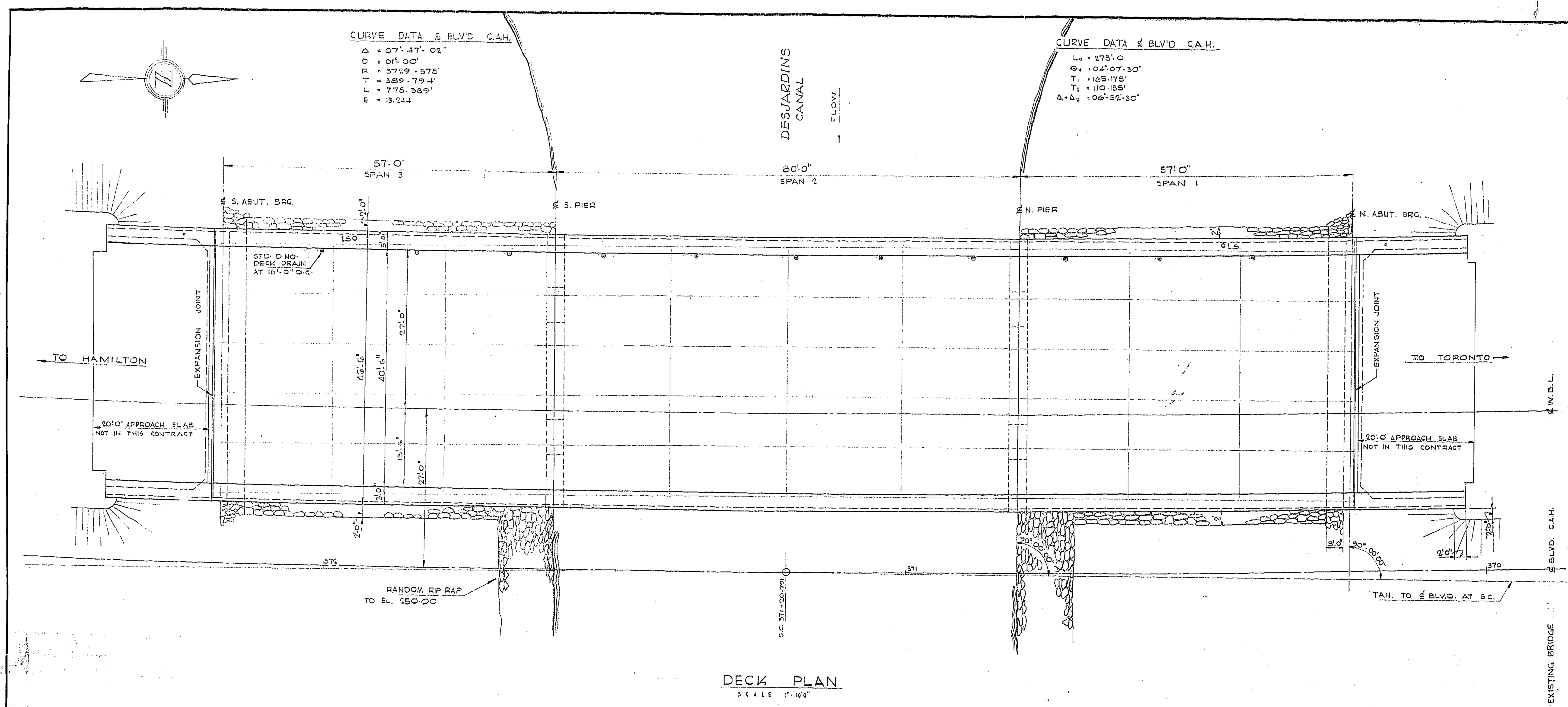
REGISTERED PROFESSIONAL ENGINEER E.P. BRUMFITT PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER J.R. McWHINNIE PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

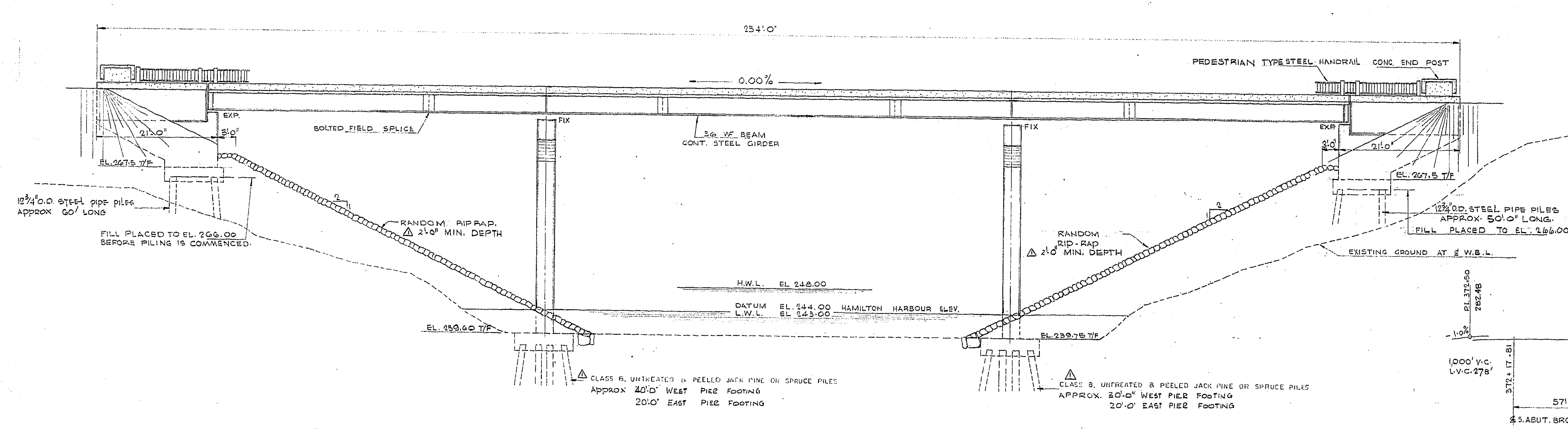
REVISIONS	DATE	BY	DESCRIPTION

DESIGN R.M.	CHECK E.P.B.	LOADING OHBDC-83	DATE DEC 84
DRAWING A.S.	CHECK R.M.	SITE 36-1336-36	DWG 1



DECK PLAN

SCALE 1" = 10'0"



SIDE ELEVATION

SCALE 1" = 10'0"

GENERAL NOTES.

NOTE TO DISTRICT 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

NOTE TO STRUCTURAL STEEL CONTRACTOR — FOR NOTES CONCERNING STRUCTURAL STEEL SEE DRG. D-4563-6

NOTE TO GENERAL CONTRACTOR - STRUCTURE TO BE BUILT IN ACCORDANCE WITH FORM No. 9 (LATEST REVISION) AND THE SPECIAL PROVISIONS, EXTRA COPIES OF WHICH MAY BE OBTAINED FROM THE DISTRICT ENGINEER

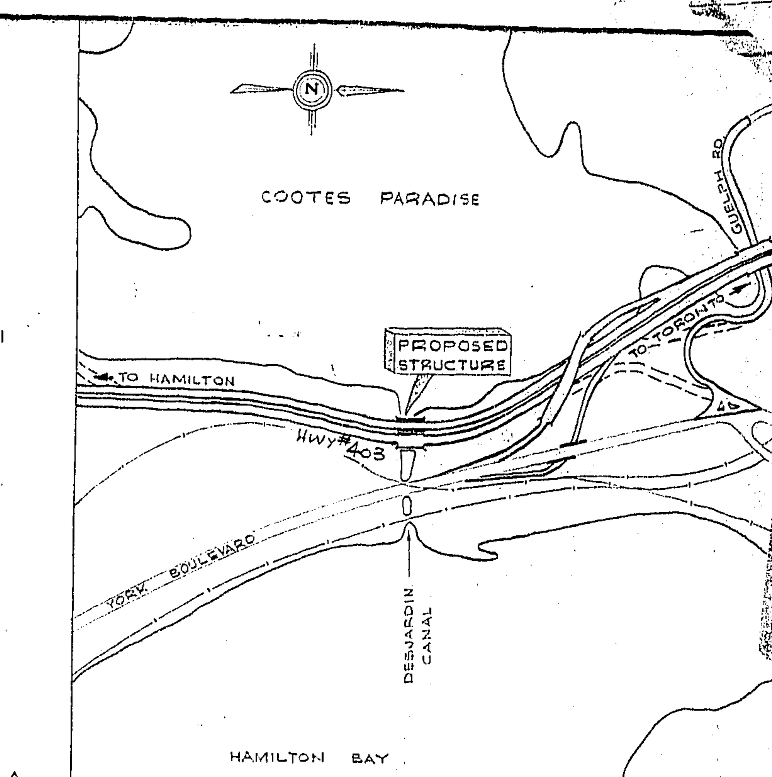
CONCRETE MIX. — MINIMUM STRENGTH AT 28 DAYS COMPLETE STRUCTURE 3000 P.S.I. APPROVED ADMIXTURES SUPPLIED BY CONTRACTOR WILL BE ADDED TO ALL CONC., AS SPECIFIED BY THE MATERIALS AND RESEARCH SECTION - D.H.O.

CLEAR COVER ON REINFORCING STEEL. FOOTINGS - 3"
ABUTMENTS I.F. 3" O.F. 2" WING WALLS I.F. 3" O.F. 2" PIER BEAMS 2"
PIER COLUMNS - 4" TO MAIN STEEL DECK TOP 1½" BOTT. 1"

CONSTRUCTION NOTES - ALL EXPOSED CONCRETE EDGES TO HAVE 1"×1" CHAMFER EXCEPT AS OTHERWISE NOTED. CONSTRUCTION JOINTS SHALL BE MADE ONLY WHERE LOCATED ON THE DRAWINGS UNLESS OTHERWISE APPROVED BY THE ENGINEER.
NO CONCRETE TO BE PLACED BEFORE MATERIALS HAVE BEEN APPROVED AND A MIX ESTABLISHED TO THE SATISFACTION OF THE ENGINEER.
FOR NOTES CONCERNING FOUNDATION CONSTRUCTION SEE DRG. D-4563-2.
NO CONCRETE TO BE PLACED BEFORE FORMWORK, FALSEWORK, AND REINFORCING STEEL HAVE BEEN CHECKED AND APPROVED BY THE ENGINEER.
NO CONCRETE TO BE PLACED ABOVE BRIDGE SEAT ELEVATION UNTIL
△ DECK CONCRETE HAS BEEN PLACED.

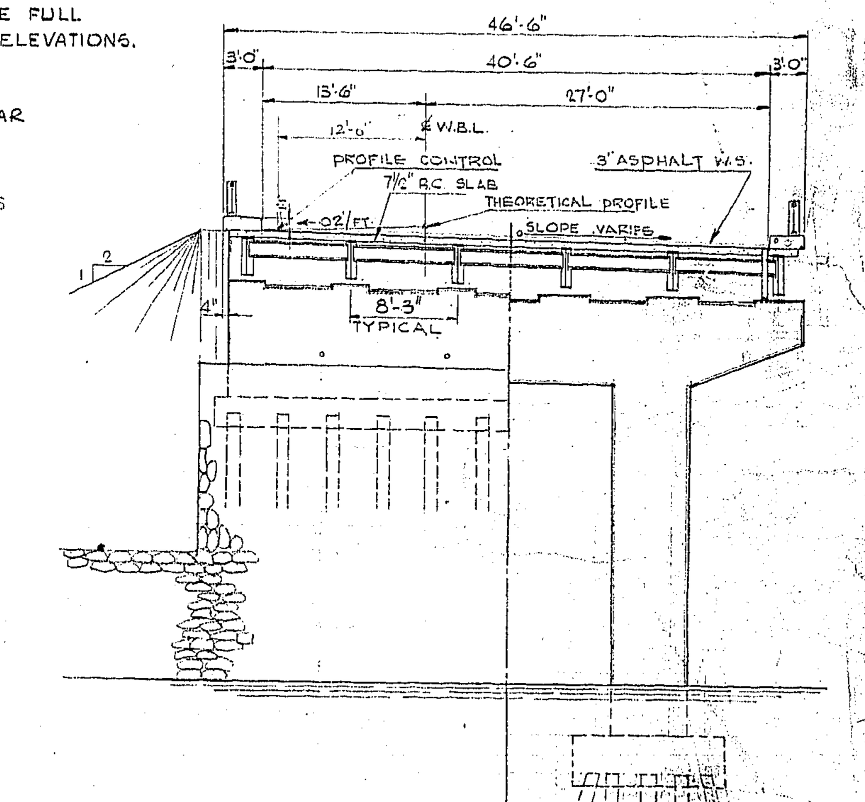
THE PAINTING OF THE STRUCTURAL STEEL SHALL BE IN ACCORDANCE WITH SPECIFICATIONS FOR STRUCTURES D.H.O FORM No. 9 AND THE SPECIAL PROVISIONS △
THE GENERAL CONTRACTOR SHALL BE RESPONSIBLE FOR FINISHING THE BRIDGE SEATS DEAD LEVEL TO THE PROPER ELEVATION FOR THE STRUCTURAL STEEL. BRIDGE SEATS SHALL BE FINISHED TO THE SPECIFIED ELEVATIONS TO A TOLERANCE OF PLUS OR MINUS 1/8 INCH. IF THEY ARE CAST TOO HIGH THEY SHALL BE BUSH HAMMERED DOWN BY THE GENERAL CONTRACTOR. IF THEY ARE CAST TOO LOW THE GENERAL CONTRACTOR SHALL PROVIDE FULL BEARING STEEL SHIM PLATES TO BRING THEM UP TO THE CORRECT ELEVATIONS. THE USE OF GROUT IS PROHIBITED I.F. — DENOTES INSIDE FACE O.F. - OUTSIDE FACE E.F. - EACH FACE.
ALL REINFORCING BAR SPLICES TO BE LAPPED 35 DIAMETERS OF BAR (MIN.) UNLESS OTHERWISE NOTED.

△2 THE GENERAL CONTRACTOR IS RESPONSIBLE THAT THE FINAL DECK ELEVATIONS CONFORM WITH THE ELEVATIONS SHOWN ON THE PLANS.



KEY PLAN

SCALE 1" = 800'

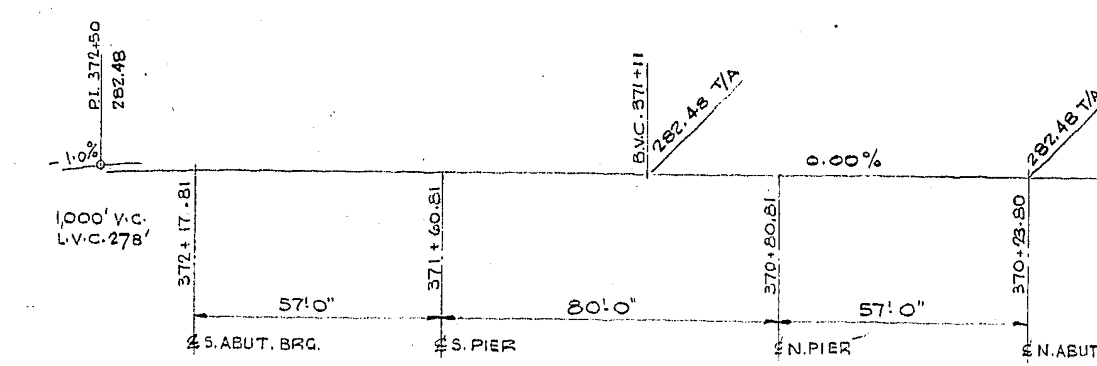


HALF SECTION AT ABUTMENT HALF SECTION

TYPICAL CROSS SECTION

SCALE 1" = 10'0"

LIST OF DRAWINGS
1 - GENERAL ARRANGEMENT
2 - FOUNDATION PLAN
3 - PIER DETAILS & REINF'G. PIER & ABUT' PILE CAP REINF'G
4 - NORTH & SOUTH ABUT' REINF'G
5 - DECK SLAB REINFORCING, DECK ELEV'S JOINT LOCAT'S. POST SPACING.
6 - STRUCTURAL STEEL DETAILS & TYP'L SECTIONS
7 - BEARING DETAILS.
8 - MISCELLANEOUS DETAILS.
9 - HANDRAIL DETAILS
10 - ELECTRICAL DETAILS
11 - APPROACH SLAB DETAILS.
12 - 13 - 14. REINFORCING STEEL SCHEDULES



THEORETICAL PROFILE ALONG ℄ W.B.L.

ELEVATIONS SHOWN ARE TOP OF ASPHALT
N.T.S.

REVISIONS:		
OCT. 3/66	A.E.S.	REVISED AS-CONSTRUCTED.
10:10:61	J.G.S.	△ NOTES ALTERED. △ NOTE ADDED.
OCT. /61	G.R.M.	ELEVATIONS ALONG THEORETICAL PROFILE AMENDED
DATE	BY	DESCRIPTION

REGISTERED PROFESSIONAL ENGINEER CIVIL C. K. MOULSON PROVINCE OF ONTARIO

69791

W.P. 193-60-1

C. C. PARKER & PARSONS, BRINCKERHOFF LIMITED
HAMILTON CONSULTING ENGINEERS ONTARIO

DEPARTMENT OF HIGHWAYS:-ONTARIO-
BRIDGE OFFICE:-TORONTO

BRIDGE AT DESJARDINS CANAL-W.B.L.

THE KING'S HIGHWAY No. 403 DIST. No. 4
CO. OF WENTWORTH CHEDOKE BRIDGE No. 13
TWP. OF WEST FLAMBOROUGH LOT 29 CON. 1

GENERAL ARRANGEMENT.

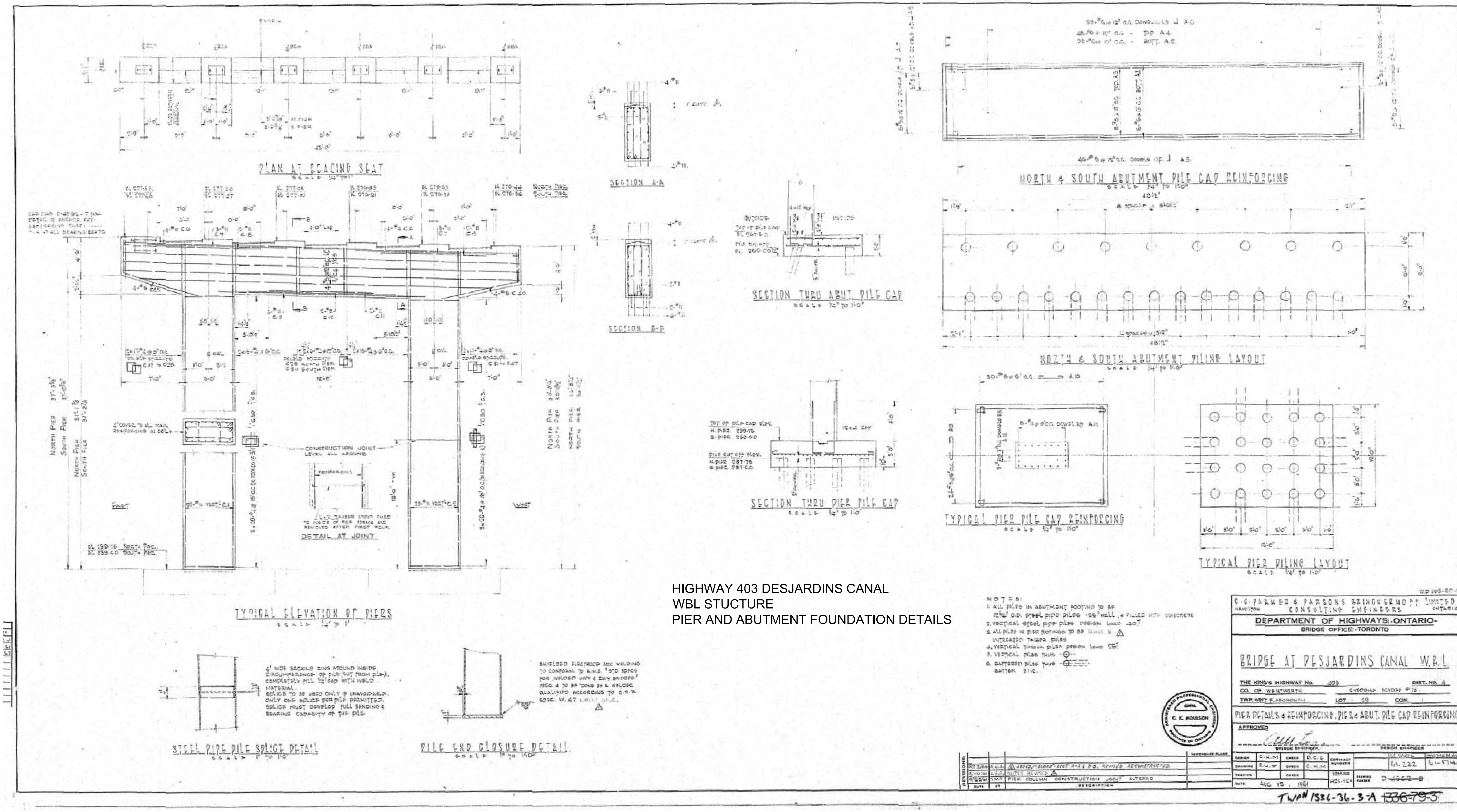
APPROVED

BRIDGE ENGINEER DESIGN ENGINEER

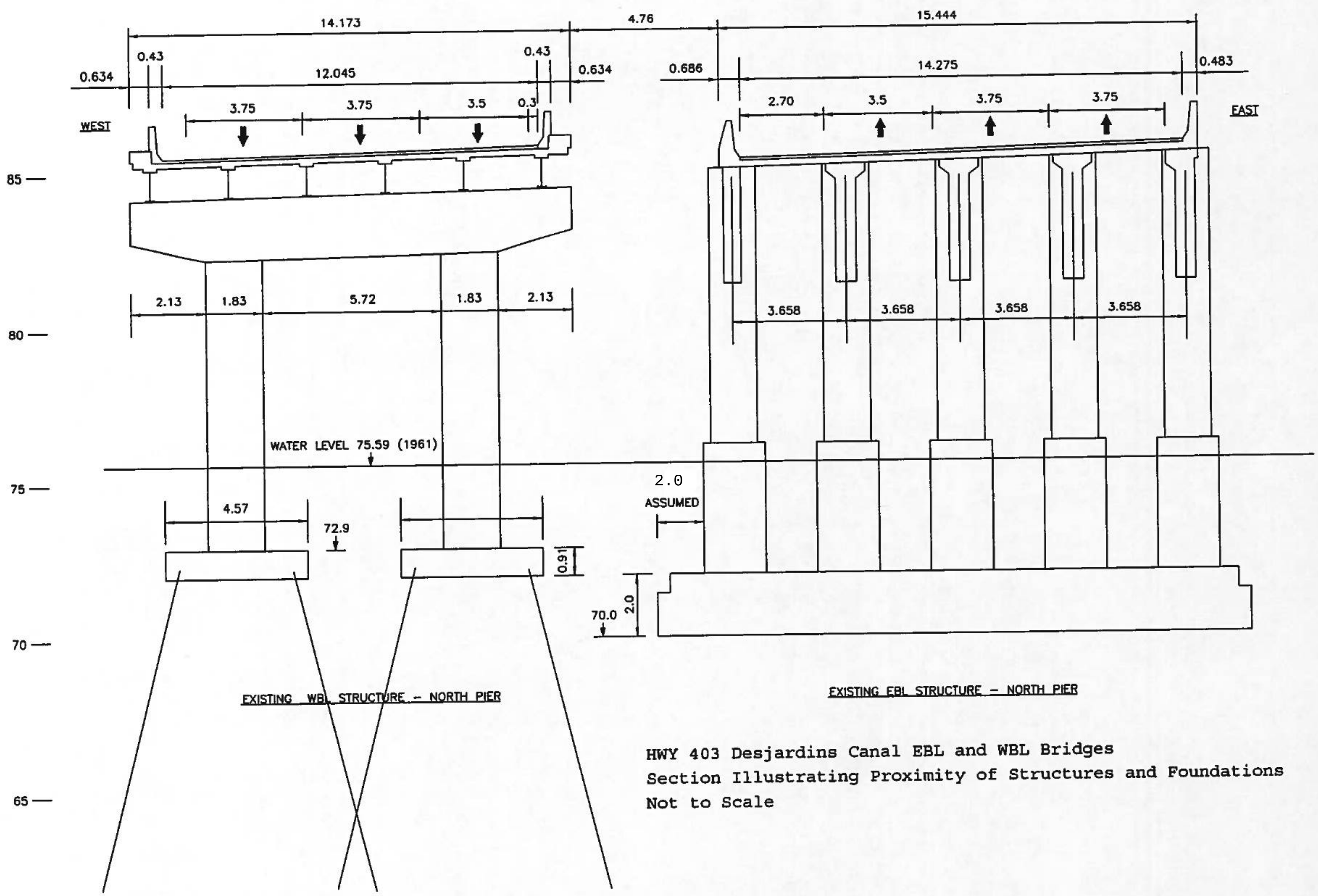
DESIGN	K.	CHECK	D.C.C.	CONTRACT NUMBERS		ST. STEEL	GENERAL
DRAWING	R.M.T.	CHECK	K.			61-222	61-174
TRACING		CHECK		LOADING	DRAWING NUMBER	D-4563-1	
DATE	AUG. 15, 1961			H25-S20			

TWP# 1336-36-1-A 33679-1

PRINT RECORD		
NO.	FOR	DATE
9	[illegible]	26.9.61
36	[illegible]	10.10.61
3	[illegible]	18.1.62



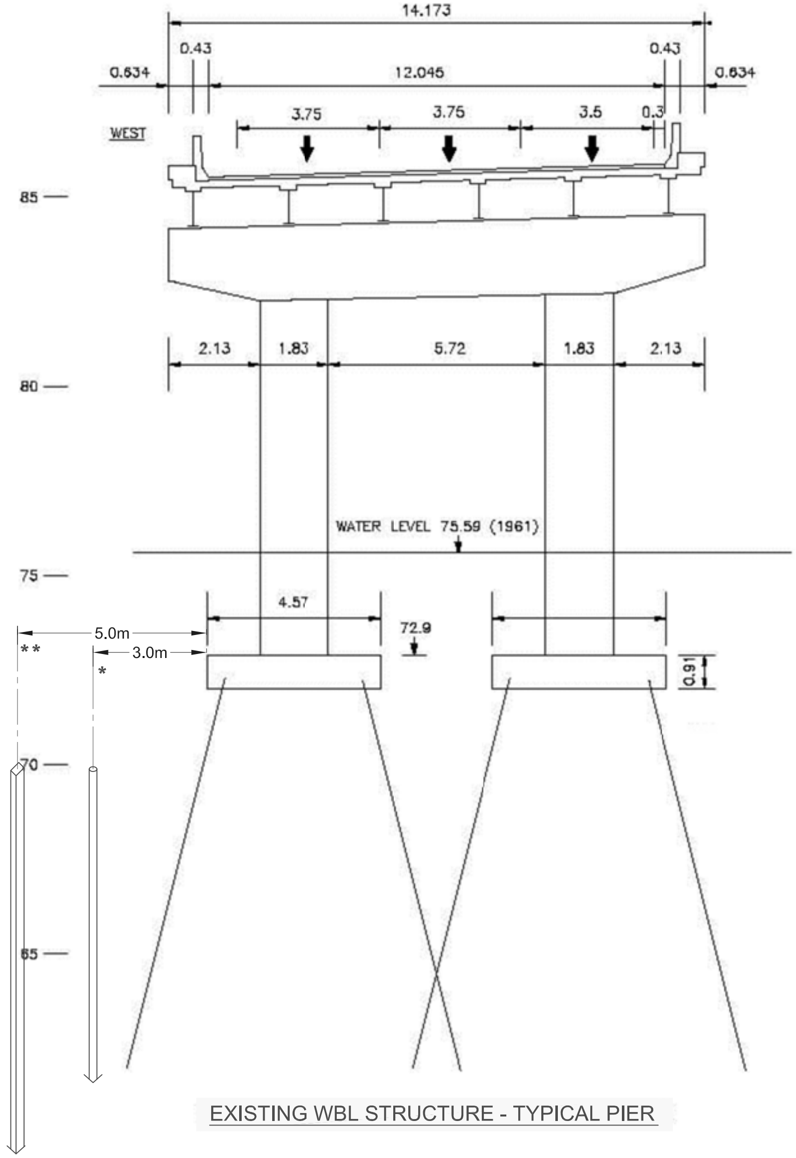
HIGHWAY 403 DESJARDINS CANAL
WBL STUCTURE
PIER AND ABUTMENT FOUNDATION DETAILS



HWY 403 Desjardins Canal EBL and WBL Bridges

Section Illustrating Proximity of Structures and Foundations

Not to Scale



EXISTING WBL STRUCTURE - TYPICAL PIER

NOTES:

* MINIMUM CLEARANCE FOR MICROPILES:
- PHYSICAL INTERFERENCE
- LOSS OF GROUND

** MINIMUM CLEARANCE FOR DRIVEN PILES:
- PHYSICAL INTERFERENCE
- VIBRATION

HIGHWAY 403 DESJARDINS CANAL BRIDGE

MINIMUM CLEARANCE FOR NEW DEEP FOUNDATIONS ADJACENT TO WBL PIERS