PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

HIGHWAY 403 AND OAK PARK ROAD
INTERCHANGE IMPROVEMENTS
GWP 3950-01-00
BRANTFORD, ONTARIO

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TABLE OF CONTENTS

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1. INTRODUCTION

This report provides the results of the preliminary foundation investigation for the upgrade of the existing Highway 403 underpass at Oak Park Road near Brantford, Ontario and preliminary recommendations for the foundation design and construction component of the project. The study was carried out for the Ministry of Transportation of Ontario (MTO) on behalf of Stantec Consulting Ltd. (Stantec).

The interchange improvement project includes the construction of new ramps for the contemplated Parclo A-4 interchange configuration that will replace the existing diamond-type interchange ramps. The construction of the Parclo A-4 loop ramps that pass under the Oak Park Road structure requires the partial or full removal of the embankment foreslopes that cover the north and south abutments of the structure. Details of the Preferred Plan, which included plan and cross-section drawings were provided by Stantec on July 8, 2005. This preliminary foundation study was carried out for the treatment of the existing abutment foundations only.

We have reviewed the factual data of the previous foundation investigation report (reference WP No. 57-60-00) prepared in 1972 for the structure (Geocres No. 40P1-56) and construction drawings for the Contract No. 75-132 dated April 1975. The documents refer to County Road 27, which was later named Oak Park Road.

The purpose of the preliminary foundation investigation component of this study was to determine the geologic and subgrade conditions at the location of the two existing structure abutment foundations based on a review of the previous investigation report and construction drawings and available literature. No field drilling work was included in the scope of work for this study. The preliminary foundation design assessment, discussion and recommendations are based on these findings.

2. SITE DESCRIPTION AND GEOLOGY

Representative site photographs of the north and south abutments obtained by Stantec and PML are included in Appendix A. Figure A shows the site location map.

Highway 403 was constructed under the Oak Park Road alignment in 1976. The area near the interchange has a typically flat to rolling topography, and the old farmlands and gravel pits are currently being developed for residential and commercial use.

A list of the documents reviewed for this study is provided in Appendix B. A copy of the previous record of boreholes and dynamic cone penetration tests is enclosed in Appendix C. Based on the reviewed data and the physiography report (*The Physiography of Southern Ontario* by Chapman and Putman 1984 edition) the site is located on the southern section of the Horseshoe Moraines physiographic region where extensive outwash gravel deposits occur. The deposits are relatively deep (up to 8 m) between the Orangeville and Brantford areas. The occurrence of these deposits at the site is confirmed by the other geological maps reviewed and by the results of the previous foundation investigation.

The MTO frost depth for the project area is 1.4 m.

3. INFERRED SUBSURFACE SITE CONDITIONS

We refer to the previous record of borehole sheets from the 1972 foundation investigation report reference WP No. 157-60-00, prepared for the structure and construction drawings for the Contract No. 75-132, dated April 1975. Copies of the relevant data are provided in Appendix C.

The previous boreholes and dynamic cone penetration tests were carried out at five locations and extended to depths ranging from 1.8 to 11.4 m (6.0 to 37.5 ft on the previous sheets), elevations 241.2 to 249.9.

The inferred typical site stratigraphy comprises cohesionless gravelly sand to sandy gravel unit as encountered in the previous boreholes. The previous standard penetration tests indicated typically very dense relative densities throughout the deposit. Typical N values were in excess of 100 blows for less than 300 mm penetration of the split spoon sampler. These soils extend to the 1.8 to 11.4 m termination depths of the boreholes and dynamic cone penetration tests.

The grain size analyses of soils samples indicated on the report ranges from 32 to 50% gravel, 33 to 46% sand and 13 to 22% silt and clay size particles. Grain size charts were not included with the report copies reviewed. It is considered that the material has low frost susceptibility. Previous determinations of the natural moisture content are less than 5% in the very dense zones of the soil units.

The bedrock underlying the general area of the site consists primarily of sedimentary limestone, dolostone and/or shale. The bedrock is encountered at about 43 m depth, elevation 198 as indicated on Map No. 2035, Bedrock Topography of the Brantford Area.

Ground water was not noted on the record of boreholes or drawings and it is anticipated that the current ground water levels are below the level of the Highway 403, based on the previous natural moisture content values of the soil samples. The lowering of the original grades by the construction of the Highway 403 underpass will also contribute to maintain the ground water at levels lower than the roadway. The ground water levels behind the abutments and under the Highway 403 pavement will be affected seasonally and by rainfall patterns. The subject site is about 1 km east of the Grand River, the regional watercourse.

4. PRELIMINARY DISCUSSION AND RECOMMENDATIONS

4.1 General

The construction of the new loop ramps of the interchange require the removal of the earth slopes under and to the east and west of the structure abutments. Conventional design and construction of the earth retaining walls are considered feasible east and west of the abutments. Alternatively, the existing slopes may be cut back to stable 2H:1V configurations where space permits.

Removal of the section of the earth slopes that cover the abutment footings (foreslopes) would expose the footing subgrade and require special design and construction considerations that are discussed in the following paragraphs.

The previous MTO construction drawings and the cross-sections provided by Stantec show that the abutments of the structure are founded on spread footings and that the founding levels of the north and south abutment footings are established at approximate elevations 242.2 and 242.3, respectively. The inferred founding subgrade soils of the spread footings consist of very dense gravelly sand to sandy gravel. The footings are 0.75 m thick, 2.1 m wide and 17.1 m long, as shown of the existing construction drawings.

The road surface of Highway 403 is at elevation 239.9, therefore the highway was cut 8.9 to 12.7 m below the original grades at the previous borehole locations. The centreline grade of the proposed ramp to the Highway 403 westbound adjacent to the north abutment of the structure is shown at elevation 241.0. The proposed ramp to the Highway 403 eastbound adjacent to the south abutment of the structure is shown at elevation 240.9. Therefore, the founding levels of the abutment footings are about 1.2 and 1.4 m above the proposed pavement levels, respectively at the north and south abutments.

We understand that the existing foreslopes will have to be fully removed to install the full width of the new ramps and shoulders. If the earth slopes are not fully removed, the shoulder width of the new ramps under the structure will have to be reduced.

In the partial earth slope removal alternative, a retaining wall should be constructed in front of the abutment footing to resist the lateral loads and provide frost and erosion protection to the founding subgrade of the structure. For the full shoulder width construction alternative, the founding subgrade of the existing abutment footings will be exposed and must be underpinned. Further alternatives that would provide for the full shoulder widths comprise extending the abutment to the required depth below the new road surface level for frost protection or reconstructing the bridge deck and abutment.

Protection of the cohesionless subgrade from frost and potential erosion should be provided.

4.2 Frost and Erosion Protection Considerations

The native soil below the inferred spread footing levels should be provided with a minimum of 1.4 m of earth cover to prevent damage due to frost action in the area of the site. The use of an equivalent thickness of extruded polystyrene (EPS) insulating product horizontally under the new pavement to prevent frost penetration into the subsoil is not recommended under the interchange structure to avoid creating problems related to pavement icing in the fall/winter/spring months. A layer of EPS may, however be used vertically as frost protection for the subgrade soils that are cut below the levels of the existing footings. A 25 mm thick layer of EPS provides a frost protection effect equivalent to a 600 mm layer of earth.

Protection of the cohesionless subgrade materials from potential erosion should be provided. All exposed subgrade below the existing foundation levels should be protected with a cast-in-place or pre-cast concrete cover. The finishing panels may also be required over all of the underpinning for structural or aesthetic reasons.

The embankment slopes cut to allow construction of the ramps east and west of the abutments will require protection from erosion using a vegetation cover similar to the existing grasses on the slope.

4.3 Retaining Wall Considerations

4.3.1 General

As indicated previously the design and construction of conventional retaining wall east and west of the abutments is considered feasible. The subgrade of the existing spread footings (including wing wall footings) should be protected from disturbance to a line drawn down at a 45° angle from a distance of 1.0 m from the east and west edges of the footings. Preliminary design parameters are provided in Section 4.8 of this report.

The horizontal forces induced by removing the earth foreslope and lowering the grade 1.2 to 1.4 m to the level of the surface of the new ramp and shoulder may be supported with permanent retaining walls installed in front of (Highway 403 side) each abutment footing. The construction of these retaining walls will require the installation of temporary shoring to support the exposed material below the existing embankment footings. Alternatively, a permanent shoring system comprising a cantilever secant caisson wall may also be used.

4.3.2 Retaining Wall with Temporary Shoring (Figures 1 and 1A)

The temporary shoring should be in place before the earth foreslopes are cut to the grade levels of the new ramp and shoulder surfaces and the permanent retaining wall is installed. It is considered that the installation of a soldier pile and lagging shoring system is feasible at the site however the operation will require the use of equipment designed to operate within low headroom conditions.

Prior to the installation of the shoring, the existing foreslope above the abutment footing founding level will have to be removed and a working platform established for the operation of the drilling equipment. Due to the restricted headroom, the steel piles for the caissons will need to be installed in sections and spliced in place. It will be required that these soldier piles be anchored back or supported with raker footings while the wood lagging is installed between the steel piles to support the earth founding subgrade below the existing footing.

The lagging is typically installed behind the front flange of the I-beams of the soldier piles as depicted on Figure 1. Alternatively, the lagging may be installed behind the rear of the I-beams (Figure 1A) providing a narrower retaining system and consequently a wider space for the shoulders of the new ramps than the typical lagging placement.

The space behind the lagging should be grouted as the excavation and shoring progresses to minimize the loss of ground in front and beneath the spread footings of the abutments. It is considered that the potential for loss of ground is relatively higher for the alternative with lagging installed behind the rear flange of the soldier pile I-beam (Figure 1A). This assessment is based on the reduced amount of native soil remaining between the lagging and the edge of the footing.

The requirement for special ground water control measures should be verified during the detailed design subsurface investigation.

The permanent retaining wall should be installed after the temporary shoring is completed. The design of the shoring and retaining wall should allow for the horizontal earth pressures induced by the existing abutment footings and retained earth mass. The design of these structures should also consider the permissible magnitude of movements of the foundation soil and abutments. If only negligible movements are permissible the shoring may have to be designed with a high coefficient of lateral earth pressure or the shoring designed as a secant caisson wall. Preliminary design parameters are provided in Section 4.8 of this report.

The retaining wall should be founded below the 1.4 m frost depth assessed for the structure site.

4.3.3 Permanent Shoring as Retaining Wall (Figure 2)

It is considered feasible to design and construct the shoring as permanent retaining walls. This alternative would require a secant caisson wall designed to work as vertical cantilever beams after installation. It is anticipated that the use of temporary steel liners is required due to the cohesionless nature of the native soil. This alternative requires the design and installation of steel reinforcement into the secant caisson walls. All of the caissons are installed in a continuous sequence and the new caisson hole intersects the previously concrete filled hole until the full wall is completed. After the concrete is completely set, the earth in front of the caisson wall is removed to the level of the new pavement surface.

The geotechnical design parameters for this alternative are similar to those of the retaining wall with temporary shoring. The exposed concrete faces resulting from this alternative would require a concrete facing for aesthetic reasons and to provide for the minimum frost protection.

4.4 Underpinning Considerations

4.4.1 Underpinning Full Width of the Footing (Figure 3)

The underpinning of the whole 2.1 m footing width should be carried down to the level of the full frost protection about 1.4 m below the top of new ramp shoulder pavement. The approximate preliminary founding levels of the underpinning are estimated at elevation 239.5 at both the north and south abutments. The 17.1 m wide footings should be underpinned in 1.2 m wide panels and in a sequence of 5 panels. A schematic representation of the underpinning procedure and sequence is shown on the attached Figure 3.

The abutment footings of the northbound and southbound lanes should be underpinned separately and traffic over the lanes being underpinned should be diverted to other lane(s) or restricted while the excavations are open.

The underpinning under the full 2.1 m wide spread footings is considered marginally feasible and the least practical in the cohesionless very dense gravelly native soils encountered at the site. The feasibility of this method will depend on the presence of ground water behind the abutment and this factor must be verified for detailed design. The resultant horizontal force acting on the underpinning concrete should be structurally resisted. Expeditious construction methods will be required to minimize loss of ground that could occur beyond the far edge of the abutment footings.

4.4.2 Underpinning Partial Width of the Footings (Figure 4)

This alternative consists of underpinning only 1.2 m of footing width along the front edge (near the Highway 403). The sides of the footings (facing east and west) should also be underpinned. The estimated elevation 239.5 founding levels for partial width are the same as for full width underpinning to provide full frost protection. The width of the panels should be 1.2 m however the underpinning sequence may be made over 4 panels instead of 5. The schematic representation of this underpinning procedure and sequence is shown on the attached Figure 4.

It is considered that the potential loss of ground is lessened with this alternative, however the resultant horizontal force acting on the underpinning concrete should be structurally resisted. Potential reinforcement installed into the underpinning concrete during the underpinning procedure is schematically shown on Figure 4.

4.4.3 Permeation Grouting

This alternative entails injecting the gravel and sand subgrade soil that is inferred below the footings with a flowable grout by a specialist contractor. It is envisaged that the injection of the stabilizing material could be made through inclined holes drilled under the footings from a working earth platform cut into the existing foreslope in front of the embankment.

The treated soil becomes a stabilized mass extending below the 1.4 m frost depth that can be cut vertically above the road surface level. This method provides a higher safety factor than the previous underpinning options however it is considered potentially costly due to the required proprietary products. A specialist Canadian grouting designer (www.ecogrout.com) has used this method for international projects involving underpinning of bridge foundations. The feasibility of this alternative depends on further investigation findings including the grain size distribution of the native soils to be permeated. These items should be investigated during detailed design.

The Permeation Grouting alternative will require the installation of vertical finishing (concrete) panels to prevent erosion of the stabilized gravel and sand subgrade.

4.4.4 Underpinning with Micropiles

Micropiles (or minipiles) use a drilled hole typically 150 to 180 mm diameter to inject a column of grout into the soil creating a small diameter friction pile. The pile strength is derived from the grout/soil interaction and from the hollow steel shaft (typically about 100 mm diameter) that is used to advance the hole and remains in the soil.

The use of micropiles (or minipiles) to transfer the footing loads to an elevation lower than the pavement was not considered practical at this site because the very dense and stony native material would likely stop the installation of these piles before the required depth level was reached. In addition, the installation of the micropiles to support the rear of the footings was not considered feasible.

4. 5 Other Alternatives

Two construction alternatives involving the extension of the abutment to a new spread footing levels designed at 1.4 m below the ramp surface level for frost protection were considered. One method requires shoring the approach embankment longitudinally along Oak Park Road and initially removing the east or west half of the embankments to expose the corresponding half of the existing footings. Temporary support is then provided to the bridge deck at the abutments and to the abutment stem with structural steel frame and underpinning. The footing of each abutment is removed in sections in a method similar to underpinning (or completely as designed) and extended down to the new footing that is also constructed in sections (or in full as designed). The embankment backfill is replaced behind the finished half of the abutment. The whole process is repeated for the opposed (east or west) half of the Oak Park Road and corresponding abutment section.

Extending the abutment to deeper spread footing levels may also be accomplished without the longitudinal shoring along Oak Park Road if the traffic over the structure is interrupted for the duration of the work or diverted over a temporary structure spanning Highway 403.

The last alternative considered is the complete reconstruction of the bridge deck and abutments to extend the span of the structure.

It is anticipated that these alternatives may not be cost effective and/or potentially cause excessive user costs due to traffic interruptions and/or restrictions.

4. 6 Advantages and Disadvantages

The various alternatives are listed below together with an estimated assessment of their advantages and disadvantages based on the inferred subgrade conditions.

<table>
<tr><th>Concept</th><th>Alternative</th><th>Advantages</th><th>Disadvantages</th></tr>
<tr><td rowspan="3">Retaining Wall in front of Abutment Footings</td><td>Temporary Shoring - Lagging behind front flange of I-beam (Figure 1)</td><td>1. Soil below footing is not excavated
2. Relatively low risk
3. Traffic maintained on Oak Park Road
4. Concept used on other MTO projects</td><td>1. Adjustments to ramp alignment are required to achieve the required side clearance
2. Installation of anchors or rakers is required</td></tr>
<tr><td>Temporary Shoring - Lagging behind rear flange of I-beam (Figure 1A)</td><td>1. Soil below footing is not excavated.
2. Provides narrower retaining structure than the front flange of I-beam alternative
3. Traffic maintained on Oak Park Road
4. Concept used on other MTO projects (for piled footing alternatives)</td><td>1. Adjustments to ramp alignment are required to achieve the required side clearance
2. Relatively higher risk than the front flange of I-beam alternative
3. Installation of anchors or rakers is required</td></tr>
<tr><td>Permanent Shoring as Retaining Wall (Figure 2)</td><td>1. Soil below footing is not excavated
2. Relatively low risk
3. Traffic maintained on Oak Park Road
4. Anchoring is not required
5. Separate retaining wall is not required</td><td>1. Adjustments to ramp alignment are required to achieve the required side clearance
2. Difficult installation of reinforcing steel in low headroom is required</td></tr>
</table>

Concept	Alternative	Advantages	Disadvantages
Underpinning of Abutment Footing	Full Width Underpinning (Figure 3)	1. Space created for full new shoulder width	1. Relatively high risk 2. Traffic on Oak Park Road restricted to one lane each way during construction 3. Possible sloughing of fill to structure
	Partial Width Underpinning (Figure 4)	1. Full new shoulder width 2. Better control of fill to structure than using full width underpinning	1. Relatively high risk 2. Traffic on Oak Park Road restricted to one lane each way during construction
	Permeation Grouting	1. Full new shoulder width 2. Relatively low risk 3. Traffic maintained on Oak Park Road 4. Anchoring not required	1. Specialized construction methods 2. High cost of proprietary products and techniques
	Underpinning with Micropiles	Not considered feasible due to subgrade conditions	Not considered feasible due to subgrade conditions
Other Alternatives	Extend Abutment Footing Deeper	1. Full new shoulder width 2. Most work done with conventional equipment 3. Traffic on Oak Park Road is not interrupted	1. High construction costs 2. Complex construction details required 3. Traffic on Oak Park Road is restricted or diverted over a temporary structure
	Replace bridge Deck and Abutments	1. Full new shoulder width	1. High construction costs 2. High user costs due to interruption of traffic on Oak Park Road 3. Uninterrupted traffic flow requires a temporary structure

4. 7 Preferred Alternative

The preferred alternative from the geotechnical standpoint is based on the currently inferred subgrade soil and ground water conditions. The preferred alternative is the cantilever secant caisson wall concept that uses the shoring as the permanent retaining structure, therefore the construction of a separate retaining wall is not required.

The feasibility of this alternative must be validated during detailed design by carrying out a site specific investigation.

4. 8 Design and Construction Considerations

It is inferred that the native soils comprise very dense gravel and sand. For preliminary design purposes, the geotechnical resistances for a new retaining wall footing or a 1.2 or 2.1 m wide underpinning footing constructed on the very dense gravelly sand or sandy gravel deposits are as follows:

Geotechnical Resistance at ULS	750 kPa
Bearing Resistance at SLS	500 kPa

The resistance at SLS normally allows for 25 mm compression of the founding medium of new footings east and west of the abutments. However, the settlements of the new retaining wall footings or underpinning under the structure are considered to be negligible since no increase of loading condition is expected. Consequently differential settlement is expected to be negligible. A footing embedment depth of 1.4 m was assumed for computation of the ULS resistance.

For preliminary purposes, the lateral earth pressure may be computed using the equivalent fluid pressure for the at-rest condition (angle of internal friction greater than 35º) presented in Section 6.9 of the CHBDC. The loads induced by the existing structure must also be considered.

It is anticipated that the typical underpinning construction will be accomplished with methods appropriate to the low headroom and space available under the deck of the bridge structure.

The inferred very dense gravelly sand and sandy gravel soils are considered Type 3 soils under the Occupational Health and Safety Act (OHSA). Ground water problems are not anticipated during construction based on existing data and nature of subsoil, however the ground water condition should be assessed during detailed design. It is anticipated that perched ground water present behind the abutments from infiltration through the Oak Park Road pavement may influence the site conditions during construction. The ground water levels will depend on the seasons and on rainfall patterns.

5. ADDITIONAL STUDIES

The preliminary assessments in this report are based on reviews of existing foundation reports and literature and a site reconnaissance only. It is recommended that confirmatory geotechnical data be obtained by conducting a site foundation investigation to confirm the data inferred during this preliminary study, since most boreholes were terminated at elevations 241.2 to 249.9 that are 2.7 to 11.4 m above the final base level (about elevation 238.5) of the new footings or contemplated underpinning. The studies should reveal the soil stratigraphy and ground water conditions at and below the founding levels of the new temporary and permanent structures.

6. CLOSURE

This report was prepared by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer and reviewed by Mr. B. R. Gray, M.Eng, P.Eng., MTO Designated Contact.

Yours very truly

Peto MacCallum Ltd.

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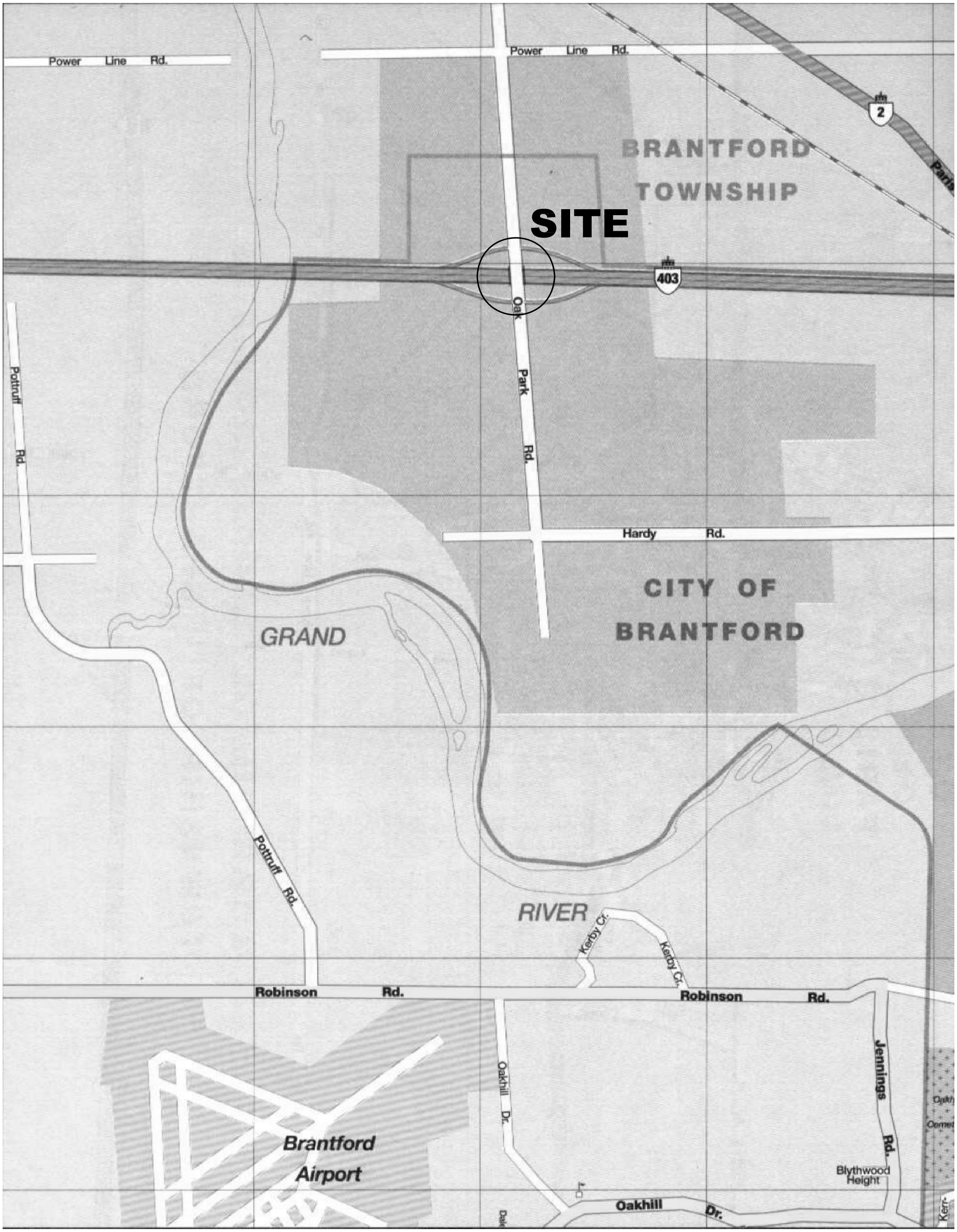

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SITE MAP OAK PARK ROAD AND HIGHWAY 403 INTERCHANGE IMPROVEMENTS BRANTFORD, ONTARIO	AUG. 2005	GWP 3950-01-00 HWY 403
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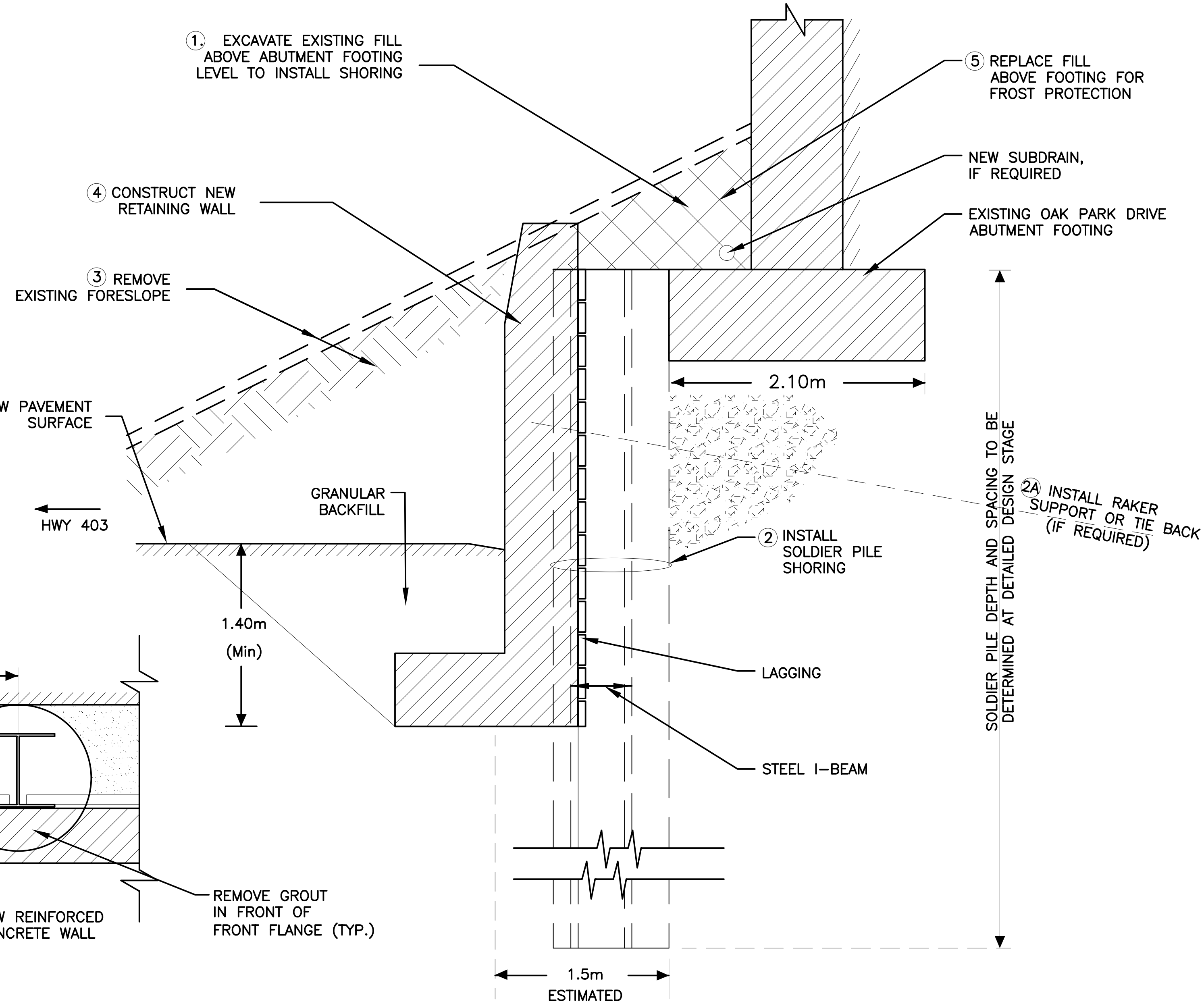
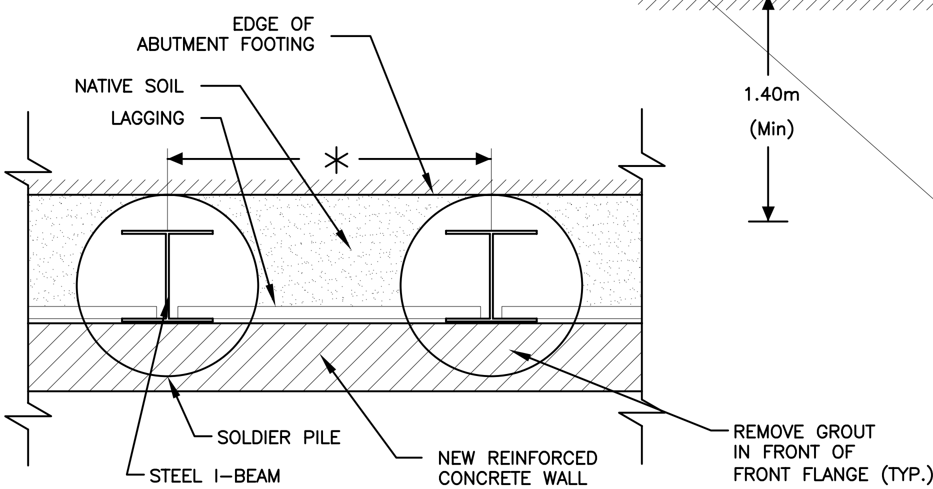
	DRAWN BY: N. A.	





PLAN DETAIL

* DISTANCE TO BE DETERMINED AT DETAILED DESIGN STAGE.

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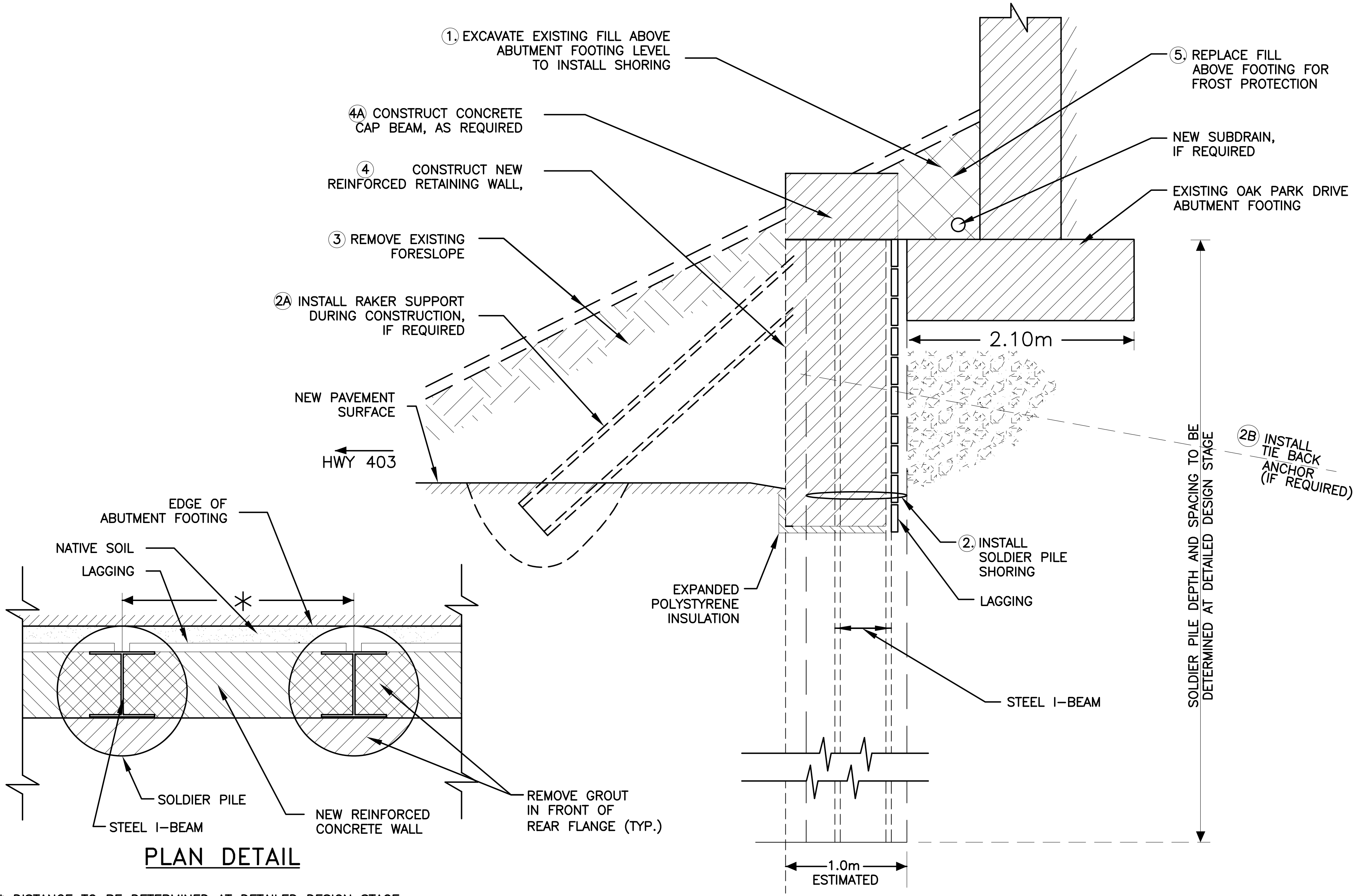


PLAN DETAIL

* DISTANCE TO BE DETERMINED AT DETAILED DESIGN STAGE.

PML

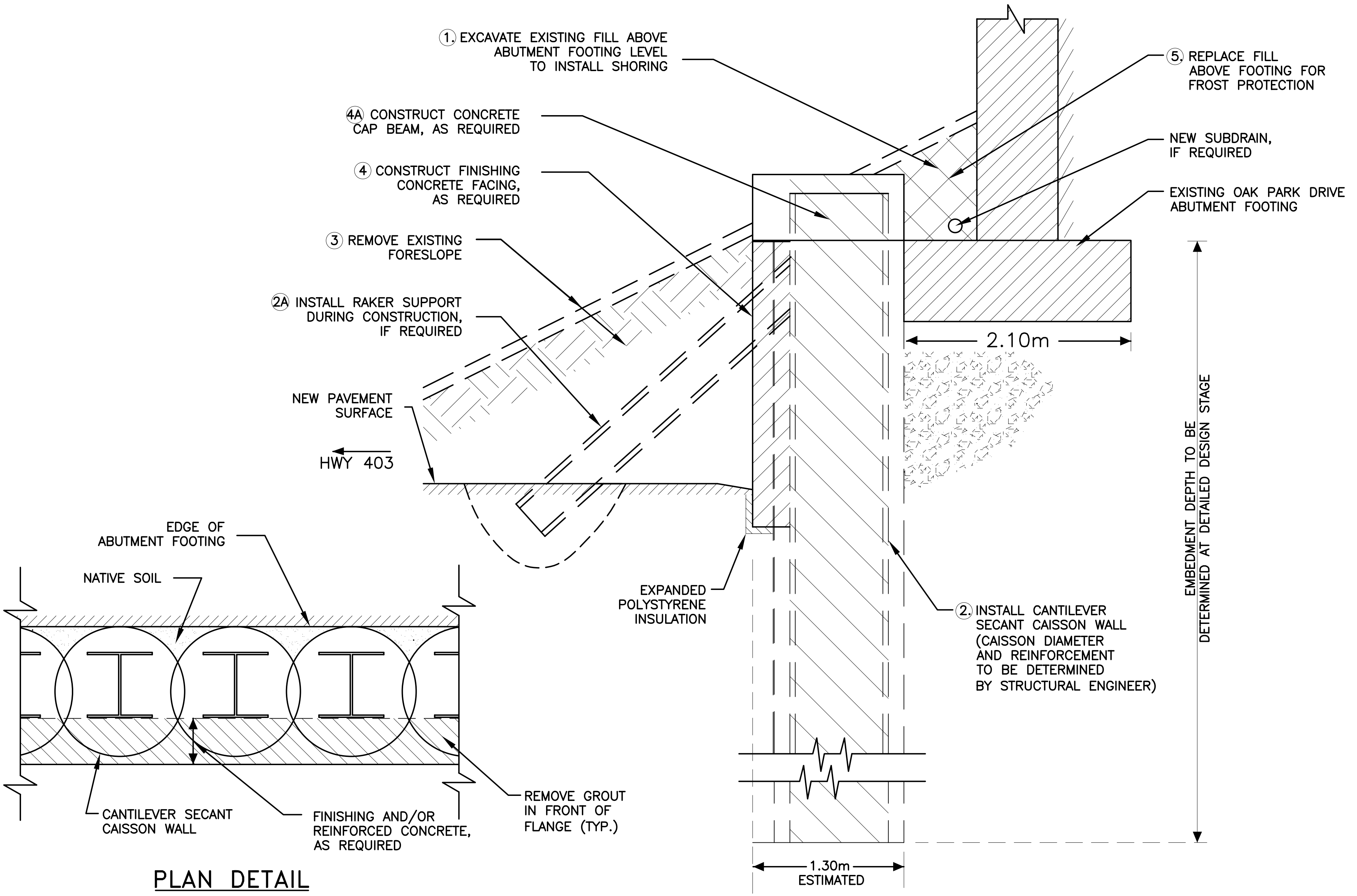
TITLE:

RETAINING WALL WITH TEMPORARY SHORING

(LAGGING BEHIND REAR FLANGE OF I-BEAM)

OAK PARK ROAD AND HIGHWAY 403
INTERCHANGE IMPROVEMENTS
BRANTFORD, ONTARIO

DATE:	OCT. 2005
SCALE:	1 : 500
DRAWN BY:	N. A.
PROJECT REF:	GWP 3950-01-00 HWY 403
FIGURE NO:	1A

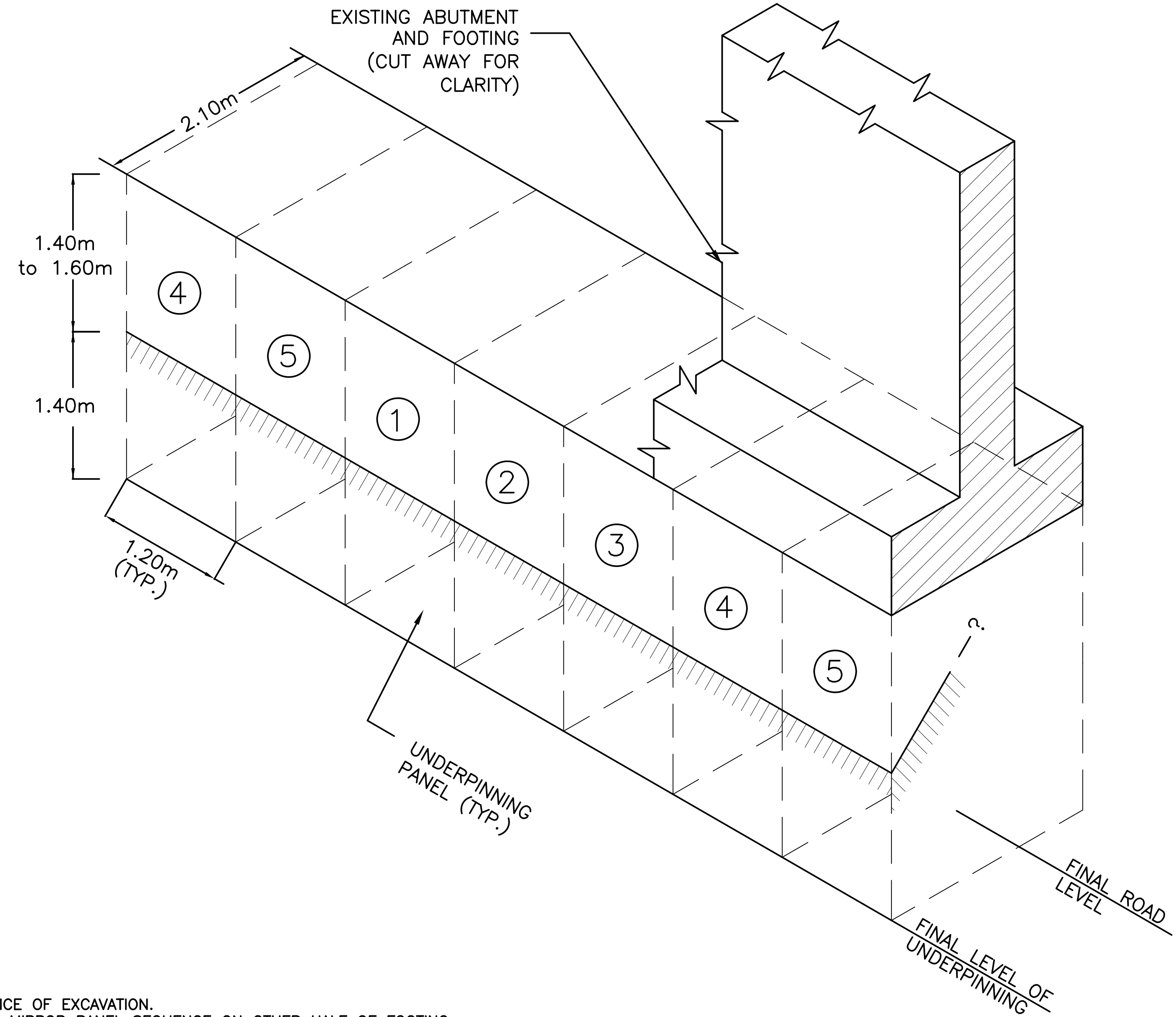


TITLE:

CANTILEVER SECANT CAISSON WALL

OAK PARK ROAD AND HIGHWAY 403
INTERCHANGE IMPROVEMENTS
BRANTFORD, ONTARIO

DATE:	OCT. 2005
SCALE:	1 : 500
DRAWN BY:	N. A.
PROJECT REF:	GWP 3950–01–00 HWY 403
FIGURE NO:	2



NOTES:
1. PANEL NUMBERS SUGGEST SEQUENCE OF EXCAVATION.
2. HALF OF FOOTING LENGTH SHOWN. MIRROR PANEL SEQUENCE ON OTHER HALF OF FOOTING.

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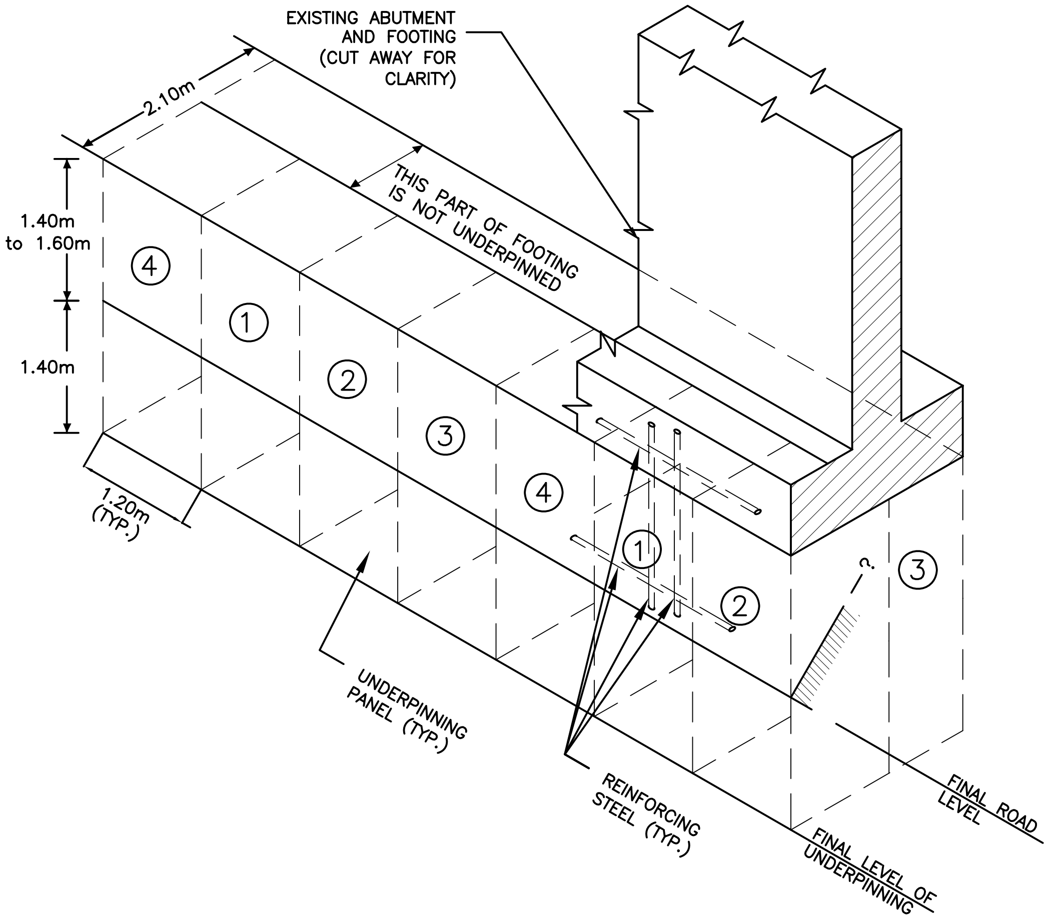
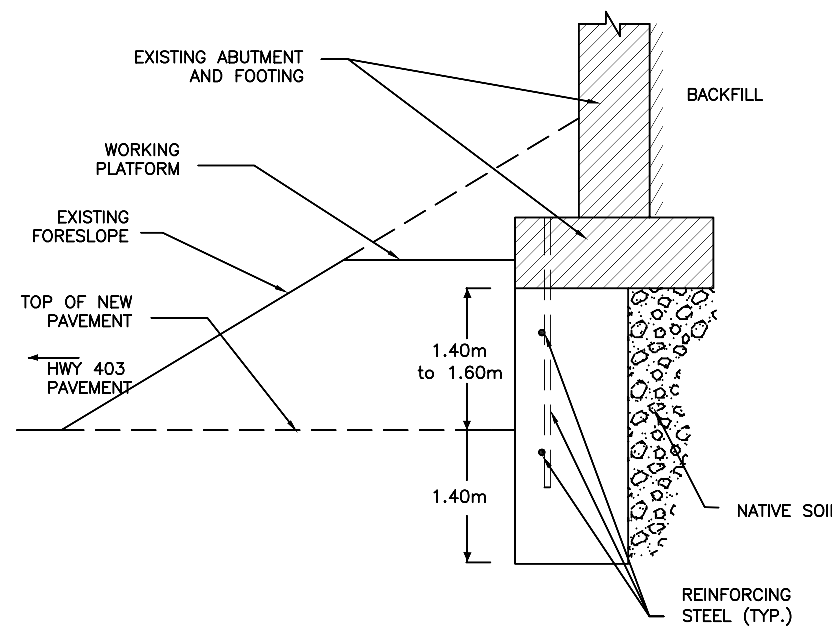
		SCALE: 1 : 600	FIGURE NO: 3
		DRAWN BY: N. A.	





SECTION THROUGH FORESLOPE AND ABUTMENT

NOTES:
1. PANEL NUMBERS SUGGEST SEQUENCE OF EXCAVATION.
2. HALF OF FOOTING LENGTH SHOWN. MIRROR PANEL SEQUENCE ON OTHER HALF OF FOOTING.
3. REINFORCING STEEL TO BE DESIGNED BY STRUCTURAL ENGINEER, IF REQUIRED.

PML	TITLE: UNDERPINNING PARTIAL WIDTH OF FOOTING OAK PARK ROAD AND HIGHWAY 403 INTERCHANGE IMPROVEMENTS BRANTFORD, ONTARIO	DATE: OCT. 2005 SCALE: 1 : 800 DRAWN BY: N. A.	PROJECT REF: GWP 3950-01-00 HWY 403 FIGURE NO: 4

APPENDIX A

SITE PHOTOGRAPHS 1 TO 4

Photograph 1: Looking west along Highway 403 at north abutment of Oak Park Road structure. Abutment foreslope will be removed to allow construction of new on-ramp under structure.

Photograph 2: Looking east at wing wall and slope of north abutment of Oak Park Road structure.

Photograph 3: Looking north from median of Highway 403 at foreslope of Oak Park Road north abutment. Note slope protection with concrete slabs and water seepage below deck on vertical abutment wall.

Photograph 4: Looking south from median of Highway 403 at foreslope of Oak Park Road south abutment. Note slope protection with concrete slabs.

APPENDIX B

LIST OF REFERENCE DOCUMENTS

LIST OF REFERENCE DOCUMENTS

Maps and Publications

- Pleistocene Geology of the Brantford Area (West Half), Preliminary Geological Map No. P. 582 from Ontario Department of Mines, issued 1970. Scale 1:50,000
- Bedrock Topography of the Brantford Area, Map 2035 from Ontario Department of Mines, Published 1953. Scale 1:63,360
- Pleistocene Geology of the Brantford Area, Map 2240 from Ontario Department of Mines and Northern Affairs. Scale 1:63,360
- Physiography of Southern Ontario, 3rd Edition by L.J. Chapman and D.F. Putnam, Published 1984 from Ministry of Natural Resources
- Physiography of Southern Ontario, Map P. 2715 from Ontario Geological Survey, Published 1984. Scale 1:600,000

MTO Documents

- Foundation Investigation Report for Proposed Underpass of Highway 403, Line "K" at Brant County Road No. 27, W.O. No. 71-11111, W.P. 157-60-00
- Drawings 1 to 17 for Contract No. 75-132 for Brant County Road No. 27 Interchange Underpass, Highway 403, Line "K", Site No. 1-139.

APPENDIX C

RELEVANT SUBSURFACE DATA

- Record of Boreholes 1 to 5, W.P. No. 157-60-00
- Borehole Locations & Soil strata, W.P. No. 157-60-00
- General Plan, Contract No. 75-132

FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION S

JOB 71-11111 LOCATION Sta. 20 + 11 26' Rt. (Co.Rd. 27) ORIGINATED BY PK

W.P. 157-60-00 BORING DATE Jan. 12, 1972 COMPILED BY AKB

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY S.R.

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE • QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REM GR SA
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE				
816.3	Ground Level									
0.0	Silty sand, some gravel									
812.3	& organics.		1	SS	5					
4.0	Sandy gravel, some silt		2	SS	100/10"	81				
			3	SS	100/8"					47 40
806.8	Very Dense									
9.5	End of Borehole									
						800				

FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION S

JOB 71-11111 LOCATION Sta. 19 + 76 27' Rt. (Co. Rd. 27) ORIGINATED BY FK

W.P. 157-60-00 BORING DATE Jan. 12, 1972 COMPILED BY AKB

DATUM Geodetic BOREHOLE TYPE Cone Test Only CHECKED BY S.R.

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE • QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REM
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE				GR,SA
817.4	Ground Level									
0.0										
6.0	End of Cone Test					810				

FORM OD–MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION

JOB 71-11111	LOCATION Sta. 18 + 97 28' Rt. (Co. Rd. 27)	ORIGINATED BY PK	
W.P. 157-60-00	BORING DATE Jan. 12, 1972	COMPILED BY AKB	
DATUM Geodetic	BOREHOLE TYPE Hollow Stem Auger	CHECKED BY S.R.	

ELEV. DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT. PLOT	NUMBER (SAMPLES)	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. O UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB. VANE	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REM GR SA
822.3	Ground Level									
0.0	Silty sand, some gravel & organics.		1	SS	40	820		o		
818.5										
3.8	Sandy gravel, some silt Very Dense		2	SS	101			o		50 33
811.8										
10.5	End of Borehole					810				

FORM OD-MT - 126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

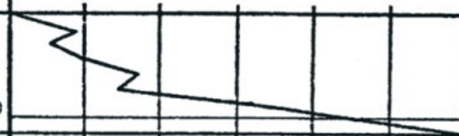
RECORD OF BOREHOLE No. 4

FOUNDATION S

JOB 71-11111 LOCATION Sta. 18 + 18 28' Rt. (Co.Rd.27) ORIGINATED BY PK

W.P. 157-60-00 BORING DATE Jan 12, 1972 COMPILED BY AKB

DATUM Geodetic BOREHOLE TYPE Cone Test Only CHECKED BY S.R.

SOIL PROFILE			SAMPLES					DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE • QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REM GR.SA
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE					
826.9	Ground Level										
0.0						820					
7.0	End of Cone Test					810					

FORM OD-MT - 126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION S

JOB 71-11111 LOCATION Sta. 17 + 82 28' Rt. (Co. Rd.27) ORIGINATED BY PK

W.P. 157-60-00 BORING DATE Jan 12-13, 1972 COMPILED BY AKB

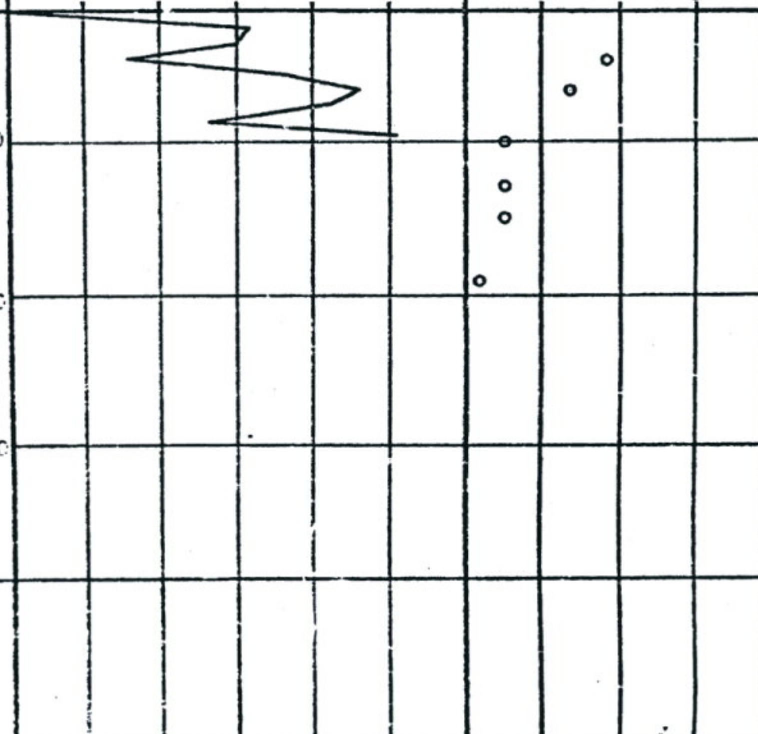
DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger CHECKED BY J.R.

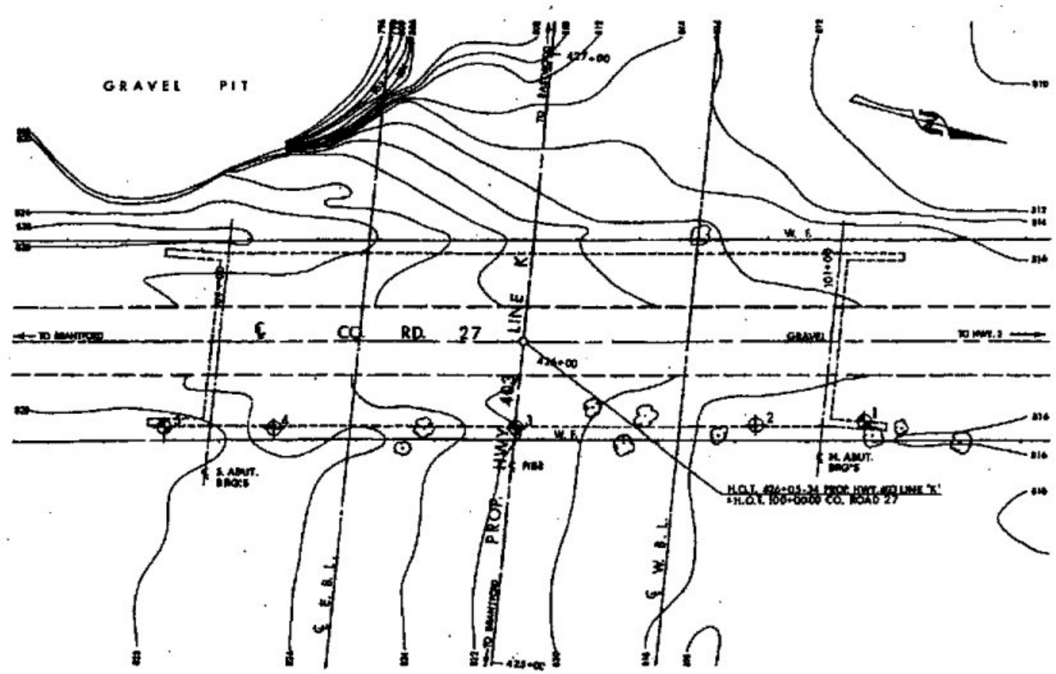
SOIL PROFILE: ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REM. GR. SA.
828.7	Ground Level							
0.0	Silty sand, some gravel & organics.		1	SS	5			
822.9 / 5.8	Gravelly sand to sandy gravel, some silt, traces of clay. Very Dense Brown		2	SS	77			
			3	SS	10[illegible]	820		
			4	SS	107/8"			32 46
			5	SS	1[illegible]/11"			
			~~6~~	~~SS~~	~~[illegible]~~/5"	810		
			~~7~~	~~SS~~	~~75~~/3"			
			~~8~~	~~SS~~	~~1[illegible]~~/6"	800		
			~~9~~	~~SS~~	~~[illegible]~~/2"			
791.2			~~10~~	~~SS~~	~~[illegible]~~/2"			
37.5	End of Borehole							

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w

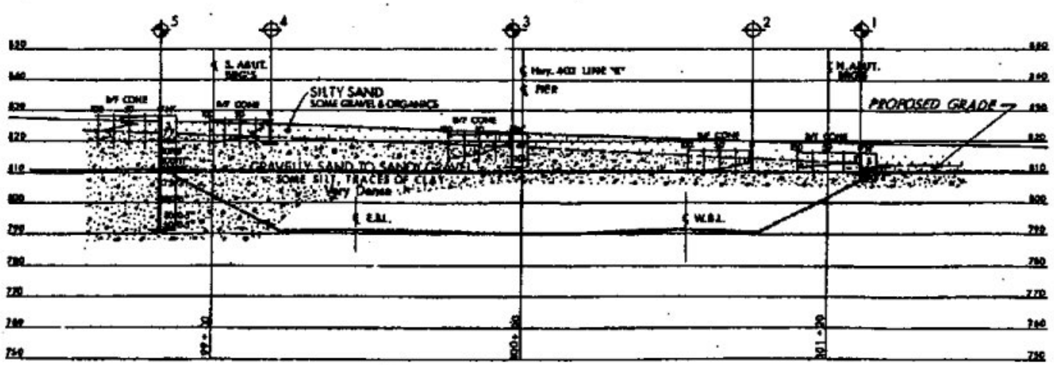
WATER CONTENT % 10 20 30





PLAN

SCALE 20 10 0 20 40 FT



℄ PROFILE CO. RD. 27

SCALE 20 10 0 20 40 FT.

KEY PLAN

SCALE IN MILES

98

LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation.

NO.	ELEVATION	STATION	OFFSET
1	816.3	101+11	26' RT
2	817.4	100+76	27' RT
3	822.3	99+97	20' RT
4	826.9	99+18	28' RT
5	828.7	98+82	28' RT

-NOTE-
BOREHOLE LOCATIONS REVISED TO COMPLY WITH CHAINAGE REVISION.
NO WATER ENCOUNTERED IN BORE HOLES - 15 JAN. 1972

NOTE FOR CONTRACT DOCUMENT
The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the HAMILTON District Office.

— NOTE —
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.

REVISIONS		
MAR/72	G.P.	STATIONS CHANGED ON PLAN & PROFILE
DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH — FOUNDATION OFFICE

COUNTY ROAD 27

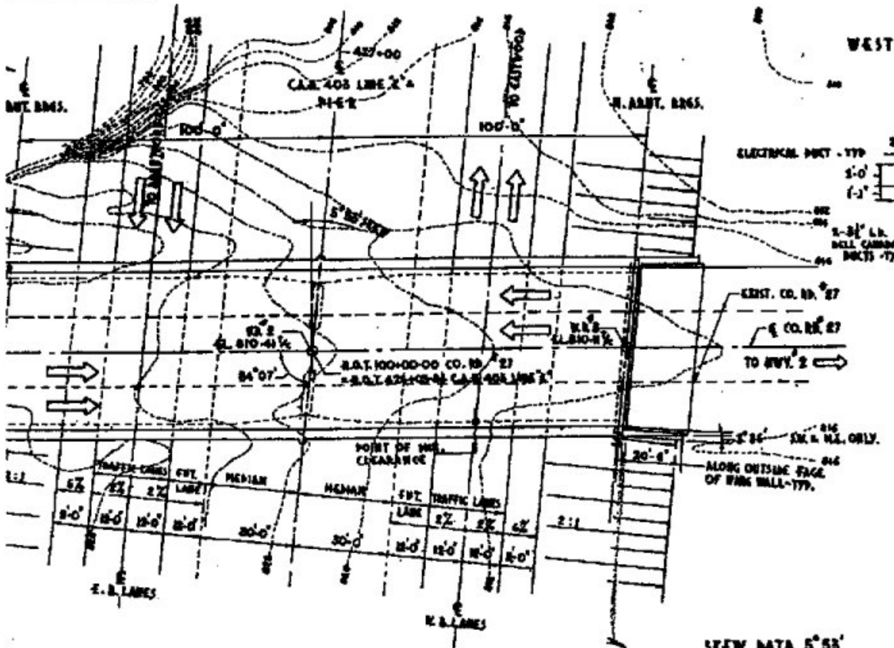
HIGHWAY NO. 403 LINE 'K' DIST. NO. 4
CO. BRANT
TWP. BRANTFORD LOT 18 & 19 CON. 2

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. P. K.	CHECKED	W.R. NO. 157-60-06	DRAWING NO. 71-11111 A
DRAWN	CHECKED	JOB NO. 71-11111	
DATE FEB. 16, 1972		SITE NO. 1-139	BRIDGE DRAWING NO. 1-139-2
APPROVED		CONT. NO. 72-422	

REF NO 2-4862-1

97

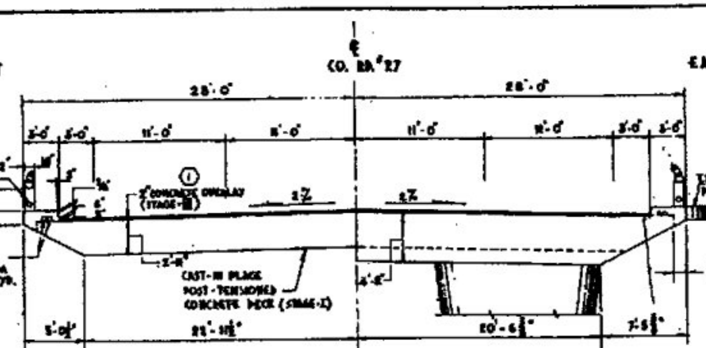


P L A N

SCALE 1"=20'

SKEW DATA 5°53'

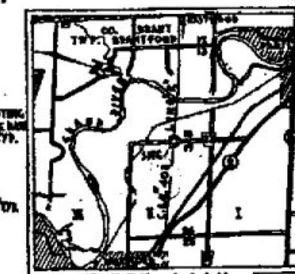
- SIN 0.102 506
- COS 0.994 785
- TAN 0.103 066
- SEC 1.005 295



AT ABUT. & SPAN — AT PIER

TYP. DECK SECT.

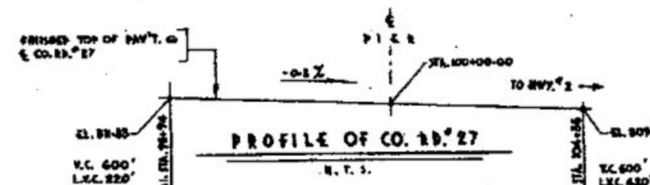
SCALE $\frac{3}{8}$" = 1'-0"



KEY PLAN

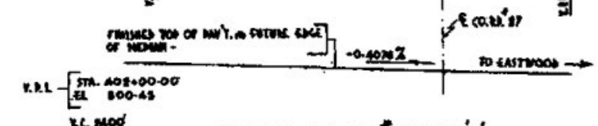
SCALE 1 IN = 1 MI.

B. M. 850-78
GEODETIC DATUM
N. & W. IN W. ROOT OF 2-3' OAK 192' RT. 425+77 LINE 'K'.



PROFILE OF CO. RD. 27

N. T. S.



PROFILE OF C.A.H. 408 LINE 'K'

CLASS OF CONCRETE

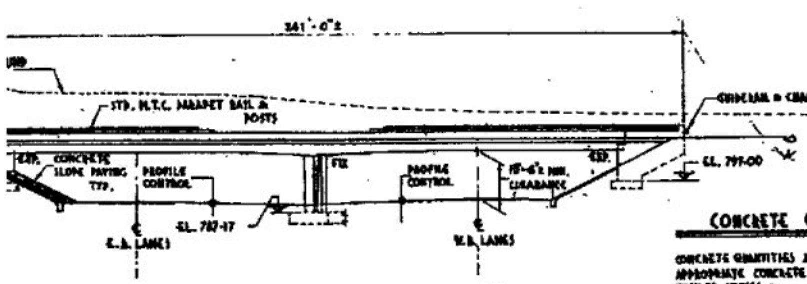
DECK & CURBS OVER DECK .... 5000 P.S.I.
PARAPET WALLS, CURBS OVER WING WALLS & PIER SHAFT .... 4000 P.S.I.
REMAINDER .... 3000 P.S.I.

CLEAR COVER TO REINF. STEEL

FOOTINGS & ABUTMENTS ..... 3"
PIER .... 2½"
TOP OF DECK .....1", BOTTOM ... 1½"; CURBS ..... 2"
PARAPET WALLS .... 1½"
OR AS NOTED ON DRAWINGS.

CONSTRUCTION NOTES

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± ⅛".
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED, STRESSED AND GROUTED.



E L E V A T I O N

SCALE 1"=20'

CONCRETE QUANTITIES

CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONCRETE LUMP SUM TENDER ITEMS :

	C.Y.	P.S.I.
• CONCRETE IN PIER, ABUTMENTS	177	3000
• WING WALLS	86	4000
● • PRESTRESSED CONCRETE BRIDGE DECK ...	1232	5000
• CONCRETE IN PARAPET WALLS	30	4000
• CONCRETE IN APPROACH SLABS	64	3000
• CONCRETE IN SLOPE PAVING	50	3000

● EXCLUSIVE OF CONCRETE OVERLAY.

LIST OF DRAWINGS

SHEET #1 - GENERAL PLAN.
2 - BORE HOLE LOCATIONS & SOIL STRATA.
3 - FOUNDATION LAYOUT, PIER & REINF.
4 - ABUTMENTS & BEARINGS.
5 - WING WALLS
6 - DECK DETAILS.
7 - CABLE DETAILS.
8 - DECK REINF. I
9 - DECK REINF. II
10 - PARAPET WALL DETAILS.
11 - STEEL PARAPET RAILING (DOUBLE TUBE)
12 - 20 FOOT APPROACH SLAB.
13 - DETAILS OF CONC. SLOPE PAVING.
14 - STANDARD DETAIL I
15 - STANDARD DETAIL II
16 - BRIDGE ELECTRICAL DETAILS - TYPE 'B'
SHEET #17 - AS CONSTRUCTED ELEV. & DIM.

FOR REDUCED PLAN
← 2 INCHES ON ORIGINAL PLAN →

REVISIONS
DATE | BY | DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

BRANT CO. RD. #27 INTERCH. U'PASS

1-8 MILES WEST OF BRANTFORD WEST LIMITS

KING'S HIGHWAY No. 403 LINE 'K' — DIST. No. 4
CO. OF BRANT
TWP. OF BRANTFORD — LOT 18 & 19 — CON. 2

GENERAL PLAN

APPROVED				CONTRACT No.	75 - 152
DESIGN	R. K.	CHECK	F.F.	W.R. No.	157-60-OC
DRAWN	C. C.	CHECK		DWG No. 1-139	SHEET 1
DATE	1/75	LOADING			