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REPORT

REPORT ON

CNR Bridge Rehabilitation and Widening Highway 401, Kingston, Ontario West Pier Micropile Foundation Detail Design G.W.P. 78-99-00

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services as part of the design of the micropile foundation system for support of the new west pier(s) to be constructed as part of the proposed widening of the existing Highway 401/CNR Overhead structure near Kingston, Ontario. Given the proximity of the structure to the adjacent existing CNR line, it is understood that the new foundation for the widened pier(s) must be designed to resist the design collision load from a locomotive (see 2.0 Site Description).

The terms of reference for the scope of work of this component of the assignment are outlined in Golder's proposal letter (08-1111-0044), dated March 4, 2011 that forms part of the Consultant's Agreement for this project.

The latest General Arrangement drawing(s) for the Highway 401, CNR Overhead bridge structure rehabilitation and widening provided to Golder by MRC are dated January 2011. It is our understanding that the existing bridge is an approximately 66 m long, three-span structure supported by a combination of shallow spread footings (west abutment and west pier) and driven piles (east pier and east abutment). The proposed widening, by about 7 m on both the north side and south side of the existing structure, is to be supported on shallow foundations at the west abutment, caissons at the east pier, driven piles at the east abutment, and micropiles at the west pier. This report addresses only the foundation design recommendations for the micropiles to support the north side and south side widening(s) of the existing west pier on micropiles.

2.0 SITE DESCRIPTION

The existing Highway 401/CNR Overhead structure carries Highway 401 over the CNR line (Mile 171.10 of the Kingston Subdivision) and is located about 310 m east of the Montreal Street interchange in Kingston, Ontario.

Through this area, Highway 401 is a four-lane divided highway with a rural cross-section. The highway profile grade on the CNR Overhead bridge structure varies from west to east from about Elevation 87.8 to 86.3 m, respectively (i.e., grade declining eastward). The existing bridge, which was constructed in 1954, consists of a three-span cast-in-place concrete girder structure supported on concrete abutments and piers. Information provided by MTO indicates that the west abutment and west pier are founded on spread footings on bedrock (or bedrock slabs), and that the east abutment and east pier are founded on piles driven to bedrock. This information is consistent with information shown on Department of Highways, Ontario Bridge Office drawings (dated April 1953, originally numbered D3349-1 though D3349-11) which were obtained by MRC and provided to Golder.

The CNR Kingston Subdivision line crosses beneath the Highway 401 structure with top of rail at an elevation of about 78 m. The railway has two tracks at this crossing, with space for a third track on the west side of the existing alignment.

To the west, adjacent to the bridge structure, bedrock outcrops exist that are up to about 9 m high relative to the existing bridge deck. To the east, the existing approach embankments are up to about 10 m high relative to the surrounding natural ground surface and have side slopes of approximately 1.5 horizontal to 1 vertical (1.5H:1V).

The west pier of the existing bridge (which is the focus of this report) is understood to be supported on spread footings founded at a depth of about 6 m below ground surface on an approximately 7.5 m thick layer of limestone slabs that contain voids and could be sensitive to disturbance and settlement from conventional foundation construction techniques (i.e., pile driving or large diameter caisson installation). Further, it is understood that the new foundation for the widened pier(s) must be designed to resist the design collision load of 2,000 kN from a locomotive engine applied at 1.5 m above adjacent ground and along 8.0 m of a protection wall tied into the pier foundation(s)/pile cap(s). Finally, due to the proximity of the existing railway tracks to the footprint of the proposed west pier widening, no excavation for construction of the pile cap(s) below the existing ground surface near the tracks will be permitted, however minor grading of the existing ground surface may be required.

Given the above constraints, micropiles (with a pile cap located close to the existing ground surface) have been selected as the preferred foundation system for support of the new west pier(s) (i.e., for both the north side and south side widening) in order to reduce potential impacts on the existing west pier foundations, structure and adjacent railway tracks during construction of the proposed widening.

3.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

3.1 Regional Geological Conditions

The site is located in the southern portion of the physiographic region known as the Napanee Plain, and just west of the Leeds Knobs and Flats, as delineated in The Physiography of Southern Ontario[1].

The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping[2] indicates that the bedrock within the Napanee Plain consists of grey limestone/dolostone of the Gull River Formation (of the Trenton-Black River Group), which contains some shale partings and seams. The limestone/dolostone is underlain by arkosic sandstone of the Shadow Lake Formation.

The overburden soils within the Napanee Plain generally consist of glacial till, although alluvium is present in river and stream valleys and, in the southern portion of the Plain, low-lying areas are typically covered with deposits of stratified clay. Well records indicate that the average depth to bedrock within the Napanee Plain is approximately 2 m. However, in many areas bedrock outcrops exist at ground surface, while deeper soil deposits (on the order of 10 m) are present in the northern and southern portion of the Plain, and within and adjacent to river valleys throughout the Plain.

The Leeds Knobs and Flats are characterized by knobs of Precambrian rock surrounded by clay flats (i.e., Clay Plain). The clay is grey in colour, and very weakly calcareous.

In particular, the study area lies within the western limits of the Cataraqui River. The Cataraqui River is characterized by a number of lakes joined by the river. This river flows southerly towards Kingston.

3.2 Site Stratigraphy

The detailed subsurface soil, bedrock, and groundwater conditions as encountered in the boreholes advanced at this site in 2009, together with the results of the laboratory testing on select soil and bedrock samples, are described in the report (Golder, 2011) titled, "Foundation Investigation and Design Report, CNR Bridge Rehabilitation and Widening, Highway 401, Kingston, Ontario, G.W.P. 78-99-00", Geocres Number 31C-202, dated January 2011. This information was supplemented by the limited stratigraphic data shown on the Department of Highways, Ontario Bridge Office drawing (original Drawing No. D3349-9) titled, "Grade Separation of CNR and Hwy 401 (Line 'C'), Plan of Borings" dated September 1953. It is noted that the stratigraphy identified as 'Rock' immediately below the 'Clay' in Boreholes #7 and #9 on this drawing is inferred to be limestone slabs.

The subsurface conditions as encountered at the areas of the proposed west pier widenings are summarized below.

West Pier – South Side Widening

The subsurface conditions at the area of the proposed West Pier, South Side widening (based on Borehole B2 (Golder, 2011) and Borehole #9 (DHO, Drawing No. D3349-9) included in Appendices A and B), consist of up to about 5.1 m of fill (comprised of a heterogeneous mixture of rock fill, gravel, silty clay and sand) overlying

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

[2] Map 2544, Ministry of Northern Development and Mines, 1991.

about 2.2 m to 3.8 m of a stiff to very stiff silty clay underlain by limestone slabs. The top of the limestone slabs (as encountered in the boreholes) varies from about Elevation 72.3 m to 73.1 m (or a depth of about 4.5 m to 5.3 m below underside of proposed pile cap at Elevation 77.6 m) and the limestone slab layer was found to be about 6.9 m thick at Borehole B2 (it was not fully penetrated in Borehole #9) and contain numerous zones of voids and/or loose soil infill, some up to about 0.2 m in thickness. The top of the limestone/dolomitic bedrock was encountered at Elevation 66.2 m in Borehole B2, however the Rock Quality Designation (RQD) values measured on recovered core samples from the upper portion of the bedrock were variable ranging from 13% to 71%, indicating a rock classification of very poor to fair quality to about Elevation 63.2 m. Below Elevation 63.2 m, the RQD of the bedrock in Borehole B2 was measured to be 83%, indicating a rock classification of good quality at this elevation. It is noted that Arkosic Sandstone and Precambrian bedrock were encountered below Elevation 62.6 m in the boreholes adjacent to Borehole B2.

West Pier – North Side Widening

The subsurface conditions at the area of the proposed West Pier, North Side widening (based on Borehole B7 (Golder, 2011) and Borehole #7 (DHO, Drawing No. D3349-9) included in Appendices A and B), consist of up to about 5.3 m of fill (comprised of a heterogeneous mixture of sand and gravel, rock fill, silt and silty sand containing cobbles and boulders) overlying about 1.0 m to 1.5 m of a very stiff silty clay underlain by limestone slabs. The top of the limestone slabs (as encountered in the boreholes) varies from about Elevation 72.9 m to 73.4 m (or a depth of about 4.2 m to 4.7 m below underside of proposed pile cap at Elevation 77.6 m) and the limestone slab layer was found to be about 5.8 m thick at Borehole B7 (it was not fully penetrated in Borehole #7) and contain numerous zones of voids and/or loose soil infill, some up to about 0.6 m in thickness. In addition, an approximately 1.2 m thick layer of cobbles, boulders and silty clay was encountered below the limestone slabs (at Elevation 67.6 m) in Borehole B7. The top of the limestone/dolomitic bedrock was encountered at Elevation 66.3 m in Borehole B7, however the Rock Quality Designation (RQD) values measured on recovered core samples from the upper portion of the bedrock were variable ranging from 0% to 67%, indicating a rock classification of very poor to fair quality to about Elevation 64.0 m. Below Elevation 64.0 m, the RQD of the bedrock in Borehole B7 was measured to be 84%, indicating a rock classification of good quality at this elevation. It is noted that Arkosic Sandstone and Precambrian bedrock were encountered below Elevation 63.2 m in the boreholes adjacent to Borehole B7.

The details of the surface elevations of the limestone slabs and bedrock at the borehole locations are summarized below.

Location of West Pier Widening	Borehole(s)	Ground Surface Elevation (m)	Top of Limestone Slabs Elevation (m)	Est. Depth to Top of Limestone Slabs, below underside of Pile Cap* (m)	Bedrock Surface Elevation (m)	Est. Depth to Top of Bedrock, below underside of Pile Cap* (m)
South Side	B2 and #9	80.4	72.3 to 73.1	4.5 to 5.3	66.2	11.4
North Side	B7 and #7	79.7	72.9 to 73.4	4.2 to 4.7	66.3	11.3

* Note: Based on Underside of Pile Cap at estimated Elev. 77.58 m.

Laboratory unconfined compressive strength (UCS) testing carried out on three (3) samples of the limestone bedrock from the site indicate UCS values ranging from about 40 MPa to 64 MPa. In addition, UCS interpreted from point load index testing carried out on nine (9) samples of the limestone and dolomitic core indicate strengths ranging from 14 MPa to 199 MPa.

Based on examination of the samples from Boreholes B2 and B7, the sub-horizontal bedding of the rock core recovered from the limestone slabs layer suggests the possibility that the approximately 6 m to 7 m thick section may have been displaced as a block during glacial activities at the contact between the limestone bluffs/rock outcrops to the west and the lowlands to the east. If bedding orientations had been more variable, it would have suggested that the material was a talus deposit of blocks which could have fallen from the bluffs/rock outcrops to the west. As described above, the limestone slab layer includes zones of voids and/or loose soil infill. These zones in the limestone slabs layer varied from up to about 0.2 m thick at Borehole B2 to up to about 0.6 m thick at Borehole B7.

The water level in the piezometer installed in Borehole B2 as measured on September 29, 2009 was found to be at about 4.3 m below ground surface (Elevation 76.1 m). It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.

For more details regarding the subsurface conditions at this site, the reader is referred to the above noted Golder (2011) report.

4.0 MICROPILE FOUNDATION DESIGN RECOMMENDATIONS

This section of the report provides recommendations on the design of the micropile foundation aspects for the proposed north and south side, west pier widening at the existing Highway 401/CNR Overhead structure. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes advanced during the subsurface investigation at the site by Golder (2011) as well on the limited stratigraphic information shown on the Department of Highways, Ontario Bridge Office drawing (original Drawing No. D3349-9) titled, "Grade Separation of CNR and Hwy 401 (Line 'C'), Plan of Borings" dated September 1953.

The interpretation and recommendations presented are intended only to provide the designers with a micropile foundation system which is capable to withstand the loads provided by MRC and with sufficient information to carry out the design of the proposed structure widening foundations and tie-ins between the old and new foundations. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the construction of the west pier foundation of the project. Those requiring information on the aspects of construction should make their own interpretation of the available factual information as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

4.1 General

The existing Highway 401/CNR Overhead structure is an approximately 66 m long, three-span structure and it is our understanding that it is supported by a combination of shallow spread footings (west abutment and west pier) and driven piles (east pier and east abutment). The proposed widening, by about 7 m on both the north side and south side of the existing structure, is to be supported on shallow foundations at the west abutment, caissons at the east pier and east abutment, and micropiles at the west pier. This report addresses only the foundation design recommendations for the micropiles to support the north side and south side widening(s) of the existing west pier on micropiles. The other foundation design aspects for the project (i.e., foundation design recommendations for abutments and east pier and approach embankment stability and settlement) are addressed in a separate report (Golder, 2011).

4.2 Design Methods

The micropile design was carried out in accordance with the FHWA/NHI Micropile Design and Construction Reference Manual, Publication No. FHWA NHI-05-039 (FHWA/NHI 2005) and the American Railway Engineering and Maintenance-of-way Association (AREMA) Manual for Railway Engineering (AREMA 2009). The geotechnical aspects of the foundation design were evaluated using the Canadian Foundation Engineering Manual, 3rd Edition (CFEM 1992) and 4th Edition (CFEM 2006) as well as the Recommendations for Prestressed Rock and Soil Anchors (PTI 2004).

4.3 Design Procedures

The detailed foundation design procedures were carried out considering the space restrictions at the site, the foundation conditions at the site (including the sensitivity of the existing structure foundations to settlement), the current state-of-practice for micropile design and the practicality and cost effectiveness of different micropile types/sizes from a construction/installation perspective.

The proposed micropile foundation was analysed considering the following load capacity cases (AREMA 2009, Section 4.2.2):

- Case A: The capacity of an individual pile as a structural member;
- Case B: The capacity of the pile to transfer its load to the ground; and,
- Case C: The capacity of the ground to support the load from the pile or piles.

The soil-structure interaction analysis of the proposed micropile arrangement was carried out using GROUP 8.0 (3D) from Ensoft, Inc. The program models the non-linear response of the soil and/or rock by means of "p-y" curves for lateral loading, "t-z" curves for axial loading, and "t-r" curves for torsional loading. For closely spaced piles, the pile-soil-pile interaction is taken into account by introducing reduction factors (or p-multipliers) for the "p-y" curves used for each single pile.

4.4 Load Combinations

The load combinations, including the factored self-weight of the pile cap, as provided by MRC for the design of the micropile group are presented below.

Load*	Load Combination	
	ULS-1	ULS-8
Axial Force (kN)	8,184.00	3,988.00
Shear Force along X-axis (kN)	429.30	-2,264.30
Shear Force along Y-axis (kN)	-860.10	-168.90
Moment about X-axis (kN·m)	5,751.00	2,012.00
Moment about Y-axis (kN·m)	13,087.00	4,339.20
Moment about Z-axis (kN·m)	-	2,500.00

* Note: X-axis on micropile cap is in the horizontal plane oriented perpendicular to rail track.
Y-axis on micropile cap is in the horizontal plane oriented parallel to rail track.
Z-axis on micropile cap is in the vertical plane.

For the purposes of design, based on discussions with MRC, the forces and moments tabulated above are located at the centroid of the pile cap at a height of 1.2 m above the underside of the pile cap in the micropile group analysis.

4.5 Micropile Group Arrangement and Cap Geometry

Based on preliminary input provided by Golder to MRC on the approximate axial and lateral load capacity of an individual micropile, MRC developed an initial micropile group arrangement for consideration in the design. The initial group micropile arrangement consisted of 35 micropiles; 7 rows of 5 micropiles per row, each spaced at a center-to-center distance of 1.525 m along the length of the pile cap (i.e., along Y-axis parallel to the rail track) with a center-to-center distance of 1 m along the width of the pile cap (i.e., along the X-axis perpendicular to the rail track). The first row of micropiles (in the X-axis) was spaced approximately 0.5 m from the existing bridge foundations. In addition, in the first two rows of micropiles (in the Y-axis) (i.e., closest to the rail track), the piles were vertical while in the remaining 3 rows, the piles were battered at an angle of 1 Horizontal to 3 Vertical (i.e., 1H:3V). The initial micropile cap was 10.2 m long (Y-axis), 5 m wide (X-axis) with a thickness of 1.2 m.

During the detailed micropile design, based on the results of the GROUP analysis and discussions on the structural aspects (i.e., demand/capacity ratio) of the piles with MRC as well as discussions with a Micropile Contractor regarding the construction and the desire to reduce the affects of installation on the existing bridge foundation, the micropile group arrangement was modified as follows:

- The group micropile arrangement was reduced to 30 micropiles; 6 rows of 5 micropiles per row based on a requirement for adequate drilling clearances from the existing foundation and given the very conservative demand/capacity ratios of the 35 micropile group;
- The micropile center-to-center distance in the group was changed to vary from approximately 1.525 m to 1.86 m along the length of the pile cap (Y-axis) with a center-to-center distance of 1 m along the width of the pile cap (X-axis);
- The distance between the first row of micropiles (in the X-axis) and the existing adjacent bridge foundation was increased to almost 1.4 m to provide better drilling clearance; and,
- The size of the pile cap was increased to 10.5 m long (Y-axis), 5 m wide (X-axis) with a thickness of 1.2 m.

The final micropile group arrangement and geometry is shown on Drawings 1 and 3 for the south and north side widenings, respectively, as well as on the Contract Drawings from MRC included in Appendix C. It is noted that the underside of the pile cap is located at the estimated Elevation 77.58 m.

4.6 Micropile Design - Structural Characteristics

Given the subsurface conditions at the west pier and the design loads for the pile group (in particular the high lateral loads and bending moments), the micropile design considered the most technically feasible for the site includes a partial-length permanent exterior casing (extending through the fills and silty clay and into the top of the limestone slabs) and a full-length, grouted, central reinforcing bar (having the bond zone socketted into the good quality limestone bedrock at depth).

The feasibility of a number of different micropile cross-sections was investigated as part of the foundation design process. A total of four (4) different sections made from a combination of a hollow steel section (i.e., outer casing), grout and either a solid or hollow core central reinforcing bar were analysed in the group pile arrangements. The various micropile sections considered and used in the analyses were comprised of the following components and associated steel section and reinforcing bar properties:

Hollow Steel Section **F_y = 552 MPa (80 ksi)**
178 mm O.D. x 13.8 mm (7" x 0.5")
273 mm O.D. x 16.5 mm (10-3/4" x 0.625")
273 mm O.D. x 13.8 mm (10-3/4" x 0.5")

Central Reinforcing Bar		
Type	**Size**	**Grade**
Solid	57 mm (#18, 2.25")	F_y=520 MPa (75 ksi)
Hollow Core	103 mm O.D., 78 mm I.D. (103/78)	F_y=570 MPa (83 ksi)
Hollow Core	127 mm O.D., 103 mm I.D. (127/103)	F_y=570 MPa (83 ksi)

In the analyses, the micropiles were assumed to be fully laterally supported as per Section 4.3.4 of the AREMA manual (AREMA 2009). A 28-day compressive strength, f'_c, equal to 35 MPa was assumed for the cement grout.

After reviewing the results of the various analyses and following several discussions and a meeting with a Micropile Contractor to consider the practicality of construction of the different alternatives, giving due consideration to the sensitivity of the adjacent existing bridge foundations to movement, the following preferred micropile design was selected:

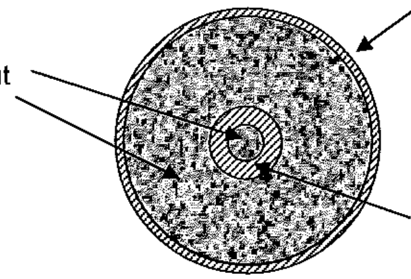
- The upper portion of all micropiles to be surrounded by a 273 mm outside diameter (O.D.) steel casing having a wall thickness of 16.5 mm. The casing will extend from within the micropile cap and terminate at an embedment depth of about 300 mm into the limestone slab layer.
- All micropiles to contain a hollow core steel central bar having an outside diameter of 103 mm and an inside diameter of 78 mm. The hollow core central bars will be drilled through the limestone slab layer and into the bedrock using a continuous thin grout flush (not water flush) with the added benefit of filling voids as encountered during installation. The central bar will extend from within the micropile cap and terminate at a minimum embedment of 3.0 m into the good quality limestone bedrock.
- Below the casing and extending through the limestone slab layer and into the underlying competent bedrock, the uncased diameter of all micropiles is assumed to be at least 175 mm (i.e., the approximate diameter of a 175 mm sacrificial drill bit).
- All micropiles to be completed with the injection of a neat cement structural grout.

The hollow core central reinforcing bar was selected over the solid central bar option for this project after giving due consideration to the subsurface conditions, the sensitivity of the existing adjacent structure and the benefits to be gained by the method of installation of this type of micropile reinforcement. The hollow core central bar will be drilled into the subsurface strata (i.e. through the overburden soils, limestone slabs and bedrock) using a sacrificial drill bit on the end of the hollow bar and employing a continuous grout flush (not water flush) to carry cuttings away from the drill bit. With this installation methodology, voids that are encountered during drilling (e.g. within the limestone slab layer) can undergo some degree of stabilization by the introduction of the continuous grouting during drilling. If a solid central bar was used for the micropiles, the drilling would be advanced using a compressed air or water flush which would have a higher risk of washing out loose materials, in particular within the limestone slab portion of the stratum, with an associated higher risk of ground loss and movement of the adjacent structure.

The steel casing, the hollow core central bar and the grout have the following characteristics:

Steel Casing:

- HSS 273 x 16 (10 ¾" x 0.625")
- API-N80 (threaded)
- F_y = 552 MPa (80 ksi)
- 273 mm outside diameter
- 16.5 mm wall thickness



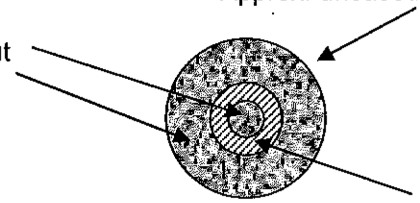
Upper "Cased" Section

Hollow Core Central Bar:

- Titan Hollow-Thread bar 103/78 (or equivalent)
- Fy = 570 MPa (83 ksi)
- 103 mm outside diameter
- 78 mm inside diameter

Final Structural Grout:

- f'c = 35 MPa (minimum at 28 days)
- Water/Cement Ratio (by weight) < 0.45



Lower "Uncased" Section

The allowable stresses used for the steel and cement grout in the assessment of the structural capacity of the micropiles (i.e., Demand/Capacity ratio) under the different Load Combinations were determined as specified in Section 4.2 of AREMA (2009) and in accordance with Sections 5.6 and 5.7 of FHWA/NHI (20005). These allowable stresses are summarize below, along with the assumed elastic properties of the micropile material.

Material	Allowable Stress (AREMA) (MPa)(1)	Allowable Stress (FHWA/NHI) (MPa)(2)	Modulus of Elasticity (GPa)
Hollow Steel Section	304	259	200
Hollow Core Central Reinforcing Bar	314	268	200
Grout	14	14	26.6(3)

Notes:
1. 0.55 x F_y for Structural Steel and 0.4 x f'_c for cement grout.
2. 0.47 x F_y for Structural Steel and 0.4 x f'_c for cement grout.
3. Equal to 4500 x $\sqrt{f'_c}$ (with f'_c in MPa).

4.7 Micropile Design – Foundation Model and Parameter Assessment

The development of the geotechnical model and the associated soil parameters for use in the GROUP analysis was based on the results of the geotechnical investigation carried out by Golder in 2009 (Golder 2011) supplemented by the stratigraphic information shown on the Department of Highways, Ontario Bridge Office drawing (original Drawing No. D3349-9) titled, "Grade Separation of CNR and Hwy 401 (Line 'C'), Plan of Borings" dated September 1953. In particular, the stratigraphy, in situ and laboratory testing as encountered in Boreholes B2 and B7 (Golder 2011) and Borehole #7 and #9 (DHO Drawing No. D3349-9) were utilized.

Although the bridge widening and micropile foundation(s) are to be constructed on both the south side and north side of the existing structure, an examination of the available information indicates the subsurface conditions for these two areas are reasonably similar. The most significant differences appear to be in the composition and consistency of the near surface fills which range from a coarse grained rock fill and very stiff silty clay fill on the south side to a compact silty sand containing cobbles and boulders (fine grained rock fill) on the north side. In order to utilize a single geotechnical model for the micropile group analysis, the conditions on both sides of the bridge were considered and the more conservative conditions selected.

Based on the available information and the considerations noted above, the simplified subsurface stratigraphy selected for the geotechnical model for the analysis comprises a sandy, fine grained rock fill overlying a stiff to very stiff silty clay crust, underlain by a layer of limestone slabs over limestone bedrock.

The details of the geotechnical model and soil and rock parameters used in the analysis are summarized below.

Material	Elevation (m)		Unit Weight (kN/m³)	s_u (kPa)	ε_{50}	Effective Angle of Internal Friction (ϕ')	E (MPa)	Poisson Ratio □'	UCS (MPa)	K (kPa/m)	Ultimate Shaft Resistance (kPa)
	From	To									
Sandy, Fine Grained Rock Fill	77.5	75.4	19	-	-	30	-	-	-	10,000 *(above GWT)* 7,500 *(below GWT)*	5 to 15
Stiff to Very Stiff Silty Clay (weathered crust)	75.4	73.1	20	80	0.007	-	-	-	-	135,000	40
Limestone Slabs	73.1	66.2	19	-	-	35	30	-	-	-	60 to 75
Limestone Bedrock	< 66.2	-	25	-	-	-	9,400	0.25	42	-	1,250

The following additional considerations as they relate to the selection of the soil and rock parameters for the model should be noted:

- Due to the spatial variability of the fill material(s) in the area, the effective angle of internal friction and the gradient of the horizontal coefficient of subgrade reaction with depth (K) are assessed to be considerably lower than the typical values for rock fill but higher than the lower bound value of loose sands.

- Given the variability in the consistency of the weathered silty clay crust, the soil parameters for this layer are selected to be more representative of a stiff clay.
- Due to the presence of voids in the Limestone Slabs (as noted during the drilling and coring investigation), the Limestone Slabs are conservatively modeled as a rock fill.
- The p-y curves for the Limestone Slabs (modeled as rock fill) and for the Limestone Bedrock have been developed based on the model proposed by St. John and Zahrah (1987). In this procedure the initial tangent of the p-y curve is assessed by means of the solution of the theory of elasticity for a line load in a infinite medium while the maximum value of the p-y curve is assessed by means of the limit resistance provided by the Limestone Bedrock or by the Limestone Slabs to the pile.
- The Young's modulus for the Limestone Slabs (modeled as rock fill) is assessed considering the hyperbolic function proposed by Duncan and Chang (1970) as in Marachi, Chan and Seed (1972).
- The Young's modulus for the Limestone Bedrock rock mass is assessed considering the intact rock Young's modulus as proposed by Deere (1968), the Rock Mass Rating Index (RMR) and the equations proposed by Hoek and Diederichs (2006).
- The Ultimate Shaft Resistance (or grout-to-ground bond value) in the bedrock has been estimated using the results of the UCS tests performed on specimens of the bedrock core and considering the recommendations in the Canadian Foundation Engineering Manual (CFEM, 2006) Section 18.6.4.2. The value calculated was then checked with the recommended typical values for limestone bedrock found in Recommendations for Prestressed Rock and Soil Anchors (PTI 2004) and the Micropile Design and Construction Reference Manual (FHWA/NHI 2005).

When a conventional pile group is subjected to a vertical or lateral load, the group resistance is not necessarily equal to the sum of the individual single pile resistances. The conventional group pile efficiency factor is dependent on the pile type, spacing, installation method and soil or rock conditions which, in some cases, can significantly affect (i.e., reduce) the group capacity.

For the proposed micropiles at this site (bonded into the good quality limestone bedrock at depth), an axial group efficiency factor equal to unity was utilized in the analyses. For group efficiency under lateral load conditions, the approach recommended by Brown et al. (1987) was employed in the GROUP analysis

4.8 Micropile Design – General Assumptions

It is our understanding that to facilitate construction adjacent to CNR tracks, no excavation below the elevation of the adjacent rail track and only minimal grading will be permitted for construction of the micropile cap. In order to satisfy frost protection requirements, it will be necessary to install a sufficient thickness of rigid extruded polystyrene insulation below and adjacent to the cap. As such, the micropile design considers a gap of 125 mm between the underside of the pile cap and the ground surface to accommodate a sufficient thickness of rigid insulation with an equivalent frost protection thickness of 1.5 m. Given this, the micropile design is based on the premise that all foundation loads (vertical and lateral) will have to be fully supported by the micropiles and that there is no contribution of the soil immediately below or adjacent to the pile cap to resist the lateral or vertical loads. The length of the bond zone (in good quality bedrock) required for the micropiles to support the axial loads from the pier cap, has been calculated based on the maximum axial load in a single pile as estimated from the micropile group analysis. Further, for gravity grouted (Type A) micropiles with a bond zone in competent bedrock, it is assumed that a group reduction factor for axial capacity is not required.

A Factor of Safety of 2.5 for the geotechnical capacity of the micropiles has been used based on the recommendations in Section 24.3.2.5 of the AREMA (2009) manual, considering the micropile bond zone will be formed in the competent medium strong limestone bedrock and assuming that at least one verification pile load test (axial and lateral) will be conducted prior to commencing the production micropile installation.

Corrosion testing results on a combined sample of the fill collected from Borehole B7 (Samples #5 and #7) indicate a medium to high corrosion potential. In this regard, a 3 mm section loss (all around) has been accounted for in the design of the outer casings of the micropiles based on the recommendations in FHWA/NHI (2005).

4.9 Micropile Group Analysis Results

The proposed pile layout and cross-sections for the micropile foundation(s) are shown on Drawings 1 and 3 for the south and north side widenings, respectively, and incorporate 30 – 273 mm (10-3/4") diameter micropiles. The analyses indicate that the use of micropiles smaller than 273 mm (10-3/4") in diameter would be inefficient and impractical given the geometry, soil conditions and load combinations at the pier widening.

In the design, each micropile consists of the following two (2) cross-sections as shown on Drawings 1 and 3:

- Upper – fully grouted HSS 273 x 16.5 exterior casing with a 103/78 hollow core thread bar
- Lower – fully grouted (approximately 175 mm diameter socket) with a 103/78 hollow core thread bar

In order to satisfy the load combinations and the requirements of AREMA (2009), the required total embedded length of each micropile in the foundation strata is estimated to range from about 17 m to 19 m, extending through the fill(s), silty clay crust, limestone slabs and a minimum of 3 m into the good quality bedrock with a tip elevation of about 60.2 m (south side) and 61.0 m (north side). The underside of the proposed new micropile cap was assumed to be at Elevation 77.6 m.

The detailed results including shear force, moment and demand/capacity ratio for each of the micropiles in the group for load combinations ULS-1 and ULS-8 are summarized in Table 1. The pile numbers and positions as well as the calculated maximum and minimum resultant bending moment, shear force and deflection diagrams for the micropile group arrangement are shown on Drawings 2 and 4 for the south and north side widenings, respectively.

The results of the analysis indicate that the maximum internal forces and deflections in the micropiles take place under Load Combination ULS-8 for the bending moment and shear forces, but take place under Load Combination ULS-1 for the axial forces. The pile cap displacements and the calculated maximum internal forces induced in a single, critical micropile within the proposed group are summarized below.

Load Combination	Max. Axial Force (kN)		Max. Resultant Bending Moment (kN·m)	Max. Resultant Shear Force (kN)	Max. Cap Displacement (mm)			Maximum Demand/Capacity[3]	
	Compression	*Tension*			*Vert.*	*Lat*[1]	*Long*[2]	FHWA	AREMA
ULS-1	787	-157	42.2	41.9	3.4	2.9	2.0	0.60	0.64
ULS-8	328	-28	82.8	68.8	1.4	3.9	3.4	0.54	0.58

Notes: 1. Perpendicular to adjacent rail track.

2. Parallel to adjacent rail track.

3. Demand over Capacity Ratio considering the combination of internal bending moments and axial forces in a micropile.

In the micropile group analysis, the pile cap is considered to be rigid and the micropiles are considered to be fixed to the pile cap. The results of the analysis indicate that the maximum resultant shear forces and bending moments occur at the contact between the pile cap and the micropiles. The results also indicate that the torsional moments along individual micropiles vertical axes are not significant (less than 0.04 kN·m);

4.9.1 Axial Geotechnical Resistance

The axial geotechnical resistance of the micropiles will be primarily developed within the bond zone of the micropile socketed into the bedrock.

The ultimate axial geotechnical capacity of a single micropile in compression for the above noted embedded lengths, assuming a diameter of the bond zone in the bedrock of 175 mm, is estimated to be 1,970 kN. This ultimate capacity satisfies the requirement that for the purpose of geotechnical analysis (i.e., Cases B and C from AREMA (2009) as described in Section 4.3 of this report), the micropiles are to be designed using a factor of safety equal to 2.5 in accordance with Section 24.3.2.5 of AREMA (2009). The factored (0.4) axial geotechnical resistance at ultimate limit states (ULS) for a single micropile is 788 kN and it is noted that the geotechnical capacity of a single pile at ULS is the governing factor in the design. The geotechnical reaction at Serviceability Limit States (SLS) for the length of piles required at this site will be greater than the factored axial resistance at ULS, since the micropiles are socketted into the limestone bedrock which is considered to be an unyielding material.

4.9.2 Demand/Capacity Ratio

The Demand/Capacity Ratio of the proposed micropile sections were calculated manually following the procedures recommended in FHWA/NHI (2005), which utilizes the guidelines of AASHTO (2002), as well as those recommended in AREMA (2009), Section 1.3.14. Given the composite nature of the proposed micropiles, the assumption is made in the calculations that the applied bending moments are resisted by only the steel casing, while the applied axial forces are resisted by the entire composite section. This assumption is necessary because AREMA (2009) and FHWA/NHI (2005) do not including bending analysis of composite sections.

Considering the combined axial compression and bending of the micropiles, the structural Demand/Capacity ratio varied from about 0.19 to 0.60 with an average of 0.42 according to FHWA/NHI (2005). The Demand/ Capacity in accordance with AREMA (2009) varied from about 0.20 to 0.64 with an average value of 0.45.

It is noted that the Demand/Capacity ratios are significantly less than 1.0. This approach to the design provides an additional Factor of Safety to account for unexpected variations in the field, in the geotechnical subsurface conditions and/or adjustments to foundation elevations during construction and/or variations in the impact load. In this regard, a Demand/Capacity ratio of 0.65 is considered appropriate for the potential unforeseen conditions at this site.

4.9.3 Buckling Potential

The potential for buckling of the micropile in the limestone slabs layer was also considered following the recommendations in FHWA/NHI (2005). For the maximum thicknesses of voids encountered in Boreholes B2 and B7 (i.e., 0.6 m), an adequate factor of safety was achieved and therefore the potential for buckling is considered low. The risk of buckling within a voided section of the limestone slabs is further mitigated by the fact that a grout flush will be used during the drilling/advancement of the hollow core central bars for the construction of the micropiles.

4.10 Micropile Dimensions

Based on the results of the micropile group analysis considering the load combinations as described in Section 4.4, the material properties outlined in Section 4.6, the soil and bedrock properties outlined in Section 4.7 and the length of the uncased (bond) zone discussion in Section 4.9, the dimensions shown below are recommended for the micropiles.

Location (on West pier)	Orientation of Micropile	Number Required	Average Cased Length(1) (m)	Average Uncased Length (m)	Plunge Length (of casing into limestone slabs) (m)	Estimated Central Bar Length(1) (m)
South Side	Vertical	12	5.38 ± 0.4	12.50 ± 0.4	0.3	17.88 ±
	Battered(2)	18	5.67 ± 0.4	13.18 ± 0.4	0.3	18.85 ±
North Side	Vertical	12	4.93 ± 0.25	12.15 ± 0.25	0.3	17.08 ±
	Battered(2)	18	5.20 ± 0.25	12.8 ± 0.25	0.3	18.00 ±

Notes: 1. Based on the assumption of a 0.5 m embedment length of the micropile central reinforcing bar and casing into the pile cap (to be confirmed by the structural engineer).
2. Based on a batter of 1 Horizontal to 3 Vertical (1H:3V).

4.11 Micropile Installation/Technical Specifications

All micropiles should be installed in accordance with the NSSP "Technical Specification for Grouted Micropiles" included in Appendix D. This specification was developed for this project in accordance with the guidelines recommended by DFI and ADSC (DFI 2004), the FHWA and the NHI (FHWA/NHI 2005) and prepared following the Ontario Provincial Standards (OPS) format. Due to the presence of voids as encountered within the limestone slabs layer, it is noted that a Low Mobility Grout may be required at some depths and locations in order to limit grout takes during the installation of the micropiles. In addition, the existing bridge structure should be monitored for movement in response to the drilling and grouting operations during construction. The drilling method(s), sequence of pile installation and grouting pressures may have to be modified or controlled depending on the response of the structure to the new construction. In this regard, any variations in the assumed field conditions/parameters shall be submitted to Golder for evaluation.

4.11.1 Micropile Connection at Pile Cap

The design condition considered in the analysis for the connection of the micropiles to the pile cap was that the micropiles would be fixed to the pile cap. Taking this into account, it is recommended that both the outer casing and inner central bar of the micropiles be extended into the pile cap to accommodate the load and moment transfer. The above recommendation can likely be provided by adopting a connection based on the typical detail shown on Figure 1. The actual required dimensions and thicknesses of the embedment, bearing plate and stiffener plates (as well as the need for any additional reinforcement, if required) will have to be determined by the structural engineer.

4.11.2 Joints in Micropile Casing

As noted in Section 4.9, large bending moments are expected to occur within the upper (i.e., cased) section(s) of the micropiles as a result of the high lateral loads and bending moments defined in the design load combinations for the west pier. As such, the bending moment capacity of the threaded casing joint has been evaluated in accordance with the recommendations of Section 5.18.3 of FHWA/NHI (2005).

For the recommended HSS 273 x 16 steel casing (i.e., 273 mm diameter with a 16.5 mm wall thickness), and assuming a 3 mm section loss, as discussed in Section 4.8, the calculated maximum bending moment allowed at a casing joint is 53 kN·m. In order to maintain a Demand/Capacity ratio of about 0.6 for the design, the maximum bending moment at the threaded joints should be about 32 kN·m (i.e., 53 kN·m x 0.6).

The range of resultant bending moments within the micropiles for loading combinations ULS-1 and ULS-8 from the GROUP analysis are show on Figure 2. From this figure it can be seen that to maintain a Demand/Capacity ratio of about 0.6, threaded joints should not be allowed within about the upper 3.0 m of the casing section of the micropiles below the pile cap. Given this, it is recommended that no threaded casing joints be allowed in the micropiles above Elevation 74.58 m.

The above constraint will result in the requirement that single, unthreaded sections of casing up to about 3.5 m (11.5 ft) in length be installed during construction. This requirement could affect the constructability of the micropiles at the site in terms of the longest casing section that can be installed within the headroom restrictions imposed by the existing bridge structure. Based on information provided by MRC, the following approximate clearances (or maximum head room) estimated to be available below the underside of the existing bridge girders are summarized below. The Contractor shall verify these dimensions based on the existing field conditions prior to commencement of the micropile drilling/installation.

Location of West Pier Widening	Elevation of Underside of Existing Bridge Girders (m)	Elevation of Base of Pile Cap / Subgrade for Construction (m)	Estimated Vertical Clearance (m) / (ft.)
South Side	85.41 ±	77.58 ±	7.83 / 25.7 ±

Location of West Pier Widening	Elevation of Underside of Existing Bridge Girders (m)	Elevation of Base of Pile Cap / Subgrade for Construction (m)	Estimated Vertical Clearance (m) / (ft.)
North Side	84.88 ±	77.58 ±	7.30 / 24.0 ±

Following discussions with a micropile contractor, it is our understanding that sufficient vertical clearance is available at the site to facilitate the construction in accordance with the above requirements on locations of the threaded joints in the casing. However, as an alternative to the above restrictions, the option could be provided in the Technical Specification that all casing joints be constructed using full penetration field welds.

4.11.3 Drilling Requirements

The Micropile Contractor must select a drilling method that will minimize the potential for ground loss during the advancement of the micropile through the sequence of limestone slabs and into the bedrock to minimize the risk of surface settlement and movement of the existing pier foundation. In this regard, it is important that hollow core central bars equipped with sacrificial drilling bits and advanced with a thin grout flush (not water flush) are utilized to install the micropiles. The use of compressed air or other method(s) of drilling and installation of micropiles, in particular through the limestone slab layer, that have a high potential for surface settlement and movement of the existing pier foundations should not be allowed.

4.11.4 Grouting Requirements

The uncased section of the micropiles shall be installed by advancing the hollow core central bar (and sacrificial drill bit) using a continuous thin grout flush during drilling. Flushing with water should not be allowed. The water/cement ratio of the thin grout flush (by weight) shall not exceed 0.90.

If large voids are encountered during drilling (in particular within the limestone slab layer), it may be necessary for the Contractor to stop advancement/drilling of the central bar and switch to injection with a Low Mobility Grout (LMG) to minimize grout take within a particular horizon. The LMG grout should be thickened by a suitable admixture (such as a thixotropic agent); thickening by use of an inert filler (such as sand) should be avoided given the potential for difficulties in pumping such a thickened grout through the sacrificial drill bits. Grouting pressures may have to be controlled depending on the response of the structure during micropile installation.

Upon completion of filling of the void (or upon achieving return of grout at surface), advancement of the central bar by drilling and flushing with the thin grout flush may continue. Near completion of drilling (i.e., when the sacrificial drill bit approaches the design tip elevation of the micropile), the Contractor shall inject a final structural grout (with a water/cement ratio (by weight) less than 0.45) from the lowest point of the drill hole until clean, pure structural grout flows from the top of the micropile.

4.11.5 Lateral and Axial Pile Load Tests

Considering the recommendations for load tests on micropiles in FHWA/NHI (2005), Section 5.9.2 and on piles in general in AREMA (2009), Section 4.3.6.2, and given the scale and importance of this project, it would be appropriate to carry-out load testing on pre-production micropiles to verify the axial and lateral performance. Axial load tests on selected production micropiles (minimum one (1) per group) should also be carried out.

For the pre-production load tests, it is recommended that at least one (1) vertical micropile be subjected to axial load testing and two (2) vertical micropiles be subjected to lateral load testing at the outset of the construction, prior to the installation of any of the production micropiles. The purpose of the axial load test is to confirm the axial geotechnical resistance at ultimate limit states (ULS), while the purpose of the lateral load test is confirm the assumptions regarding the horizontal coefficient of sugbrade reaction of the upper soil layers.

It is recommended that the pre-production test micropiles be installed on the north side of the existing bridge near the west pier widening area as the near surface overburden soil (in particular the silty sand fill) in this area is considered to be more critical for the design and performance of the micropiles subjected to lateral loading. The pre-production test micropiles shall be installed at a minimum distance of 2 m from the location of the production micropiles.

During production micropile installation, it is recommended that a minimum of one vertical micropile be subjected to proof load testing (under axial compression conditions) within each pile group on the north and south side pier widening.

A proposed load test pile arrangement for the pre-production micropile testing is shown on Figure 3. The details of the pile load tests (in terms of load increments and maximum applied loads) are included in the NSSP "Technical Specification for Grouted Micropiles" in Appendix D.

4.11.6 Frost Protection

Based on the information provided in the Ontario Provincial Standard Drawing, OPSD-3400.010, a frost penetration depth of approximately 1.5 m is expected at the location of the site. In order to avoid excavation adjacent to the rail track and the embedment of the micropile cap for frost protection (as described in Section 4.8), it is recommended that a 125 mm thickness of rigid extruded polystyrene insulation (i.e., DOW Highload or equivalent) be installed below and around the perimeter of the pile cap, extending to a distance of 1.5 m in all directions.

4.11.7 Monitoring of the Existing Foundation

It is our understanding that the west pier of the existing bridge is founded on a spread footing supported on the limestone slab layer. Given the likelihood of sensitivity of the existing foundation to movement and considering the presence of voids within the limestone slab layer, it is recommended that a monitoring program be set-up to continuously monitoring the performance and response of the existing bridge to the new construction.

Monitoring of the existing structure could include checking for settlement and tilt of the existing pier. The details of the structural monitoring program, along with the Review and Alert Levels appropriate for the existing bridge structure must be defined by the structural engineer.

In addition, it is recommended that the sequence of the installation of the micropiles within the group(s) be specified in the Technical Specification so that the micropiles furthest from the existing foundation (i.e., Piles #1 to #5 as shown on Drawings 2 and 4) be installed first, while the micropiles closest to the existing foundation (i.e., Piles #26 to #30) be installed last. In this manner, given the type of micropiles recommended for construction (i.e., hollow core central bars drilled with a continuous grout flush through the limestone slab layer), the voided limestone slab layer that currently supports the existing west pier foundation, should undergo a level of ground improvement as each micropile is advanced within the group.

5.0 CLOSURE

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TABLE 1: Detailed Results of Micropile GROUP Analysis - Load Combinations ULS-1 and ULS-8

ULS case	Pile	P (kN)	Shear Force Y (kN)	Shear Force Z (kN)	Moment x (kNm)	Moment y (kNm)	Moment z (kNm)	Resultant Moment (kNm)	Resultant Shear Force (kN)	Displacement X, (mm)	Displacement Y, (mm)	Displacement Z, (mm)	Demand/Capacity according to FHWA	Demand/Capacity according to AREMA (2009)
1	1	-149.78	-6.51	-25.38	0.04	27.90	1.65	27.9	26.2	-0.7	-1.3	-1.5	0.23	0.23
1	2	16.99	-10.65	-27.03	0.04	29.87	-3.07	30	29.1	0.1	-1.6	-1.7	0.19	0.20
1	3	183.64	-14.70	-28.66	0.04	31.80	-7.73	32.7	32.2	0.9	-1.9	-1.8	0.28	0.31
1	4	523.16	-30.65	8.46	0.04	-1.08	-34.70	34.7	31.8	2.3	-1.9	1.5	0.44	0.41
1	5	717.72	-34.20	9.17	0.04	-1.60	-37.99	38	35.4	3.1	-2.0	1.5	0.54	0.51
1	6	-151.01	-9.25	-24.81	0.04	27.51	-1.53	27.5	26.5	-0.7	-1.5	-1.5	0.23	0.23
1	7	15.76	-13.19	-26.32	0.04	29.35	-6.10	30	29.4	0.1	-1.8	-1.7	0.19	0.22
1	8	182.42	-17.09	-27.88	0.04	31.23	-10.65	33	32.7	0.9	-2.1	-1.8	0.28	0.32
1	9	535.65	-29.85	10.84	0.04	-3.97	-34.13	34.4	31.8	2.3	-1.9	1.7	0.44	0.43
1	10	730.21	-33.53	11.79	0.04	-4.66	-37.53	37.8	35.5	3.2	-2.0	1.7	0.55	0.53
1	11	-152.26	-12.34	-24.83	0.04	27.50	-4.92	27.9	27.7	-0.7	-1.7	-1.5	0.23	0.25
1	12	14.52	-16.23	-26.34	0.04	29.34	-9.46	30.8	30.9	0.1	-2.0	-1.7	0.19	0.24
1	13	181.17	-20.07	-27.87	0.04	31.20	-13.96	34.2	34.3	0.9	-2.3	-1.8	0.29	0.34
1	14	548.35	-29.90	13.68	0.04	-7.15	-34.15	34.9	32.9	2.4	-1.9	1.9	0.45	0.45
1	15	742.92	-33.53	14.76	0.04	-7.94	-37.52	38.3	36.6	3.2	-2.0	1.9	0.56	0.55
1	16	-153.78	-16.43	-25.23	0.04	27.74	-9.27	29.2	30.1	-0.7	-1.9	-1.5	0.24	0.28
1	17	13.00	-20.34	-26.78	0.04	29.60	-13.81	32.7	33.6	0.1	-2.2	-1.7	0.2	0.27
1	18	179.66	-24.18	-28.32	0.04	31.47	-18.31	36.4	37.2	0.9	-2.5	-1.8	0.3	0.37
1	19	563.84	-30.45	17.52	0.04	-11.28	-34.51	36.3	35.1	2.5	-1.9	2.1	0.47	0.49
1	20	758.41	-33.98	18.70	0.04	-12.15	-37.80	39.7	38.8	3.3	-2.0	2.1	0.57	0.58
1	21	-155.02	-18.98	-24.65	0.04	27.31	-12.30	29.9	31.1	-0.7	-2.1	-1.5	0.25	0.30
1	22	11.75	-22.69	-26.10	0.04	29.11	-16.71	33.6	34.6	0.1	-2.4	-1.7	0.21	0.28
1	23	178.41	-26.36	-27.58	0.04	30.93	-21.09	37.4	38.2	0.9	-2.7	-1.8	0.3	0.38
1	24	576.54	-29.68	19.72	0.04	-14.05	-33.96	36.7	35.6	2.5	-1.9	2.3	0.47	0.50
1	25	771.11	-33.28	21.14	0.04	-15.09	-37.31	40.2	39.4	3.4	-2.0	2.3	0.58	0.60
1	26	-156.54	-23.13	-25.02	0.04	27.52	-16.66	32.2	34.1	-0.8	-2.4	-1.5	0.26	0.32
1	27	10.23	-26.86	-26.52	0.04	29.36	-21.07	36.1	37.7	0.0	-2.6	-1.7	0.22	0.31
1	28	176.89	-30.41	-27.93	0.04	31.16	-25.40	40.2	41.3	0.9	-2.9	-1.8	0.32	0.41
1	29	592.04	-30.15	23.58	0.04	-18.18	-34.26	38.8	38.3	2.6	-1.9	2.5	0.49	0.54
1	30	786.60	-33.63	25.05	0.04	-19.27	-37.54	42.2	41.9	3.4	-2.0	2.5	0.6	0.64
8	1	-28.12	68.76	-3.30	0.03	4.01	82.70	82.8	68.8	-0.1	3.8	-0.2	0.51	0.53
8	2	83.65	59.56	-4.13	0.03	5.13	75.01	75.2	59.7	0.4	3.6	-0.3	0.49	0.51
8	3	195.39	57.50	-5.31	0.03	6.50	72.47	72.8	57.7	0.9	3.4	-0.3	0.52	0.55
8	4	80.15	-6.86	-58.35	0.03	73.58	-8.60	74.1	58.7	0.4	-0.4	-3.5	0.48	0.53
8	5	210.61	-8.02	-58.50	0.03	73.78	-9.96	74.4	59	0.9	-0.5	-3.5	0.54	0.58
8	6	-18.13	66.98	-3.32	0.03	4.03	80.47	80.6	67.1	-0.1	3.6	-0.2	0.49	0.52
8	7	93.63	57.90	-4.15	0.03	5.14	72.89	73.1	58.1	0.5	3.4	-0.3	0.48	0.51
8	8	205.37	55.84	-5.35	0.03	6.52	70.34	70.6	56.1	1.0	3.3	-0.3	0.52	0.54
8	9	101.47	-6.89	-56.91	0.03	71.76	-8.62	72.3	57.3	0.5	-0.4	-3.4	0.48	0.52
8	10	231.93	-8.06	-57.05	0.03	71.96	-9.99	72.6	57.6	1.0	-0.5	-3.4	0.54	0.58
8	11	-7.97	65.19	-3.35	0.03	4.05	78.22	78.3	65.3	0.0	3.5	-0.2	0.48	0.50
8	12	103.78	56.36	-4.19	0.03	5.17	70.83	71	56.5	0.5	3.3	-0.3	0.47	0.50
8	13	215.52	54.28	-5.41	0.03	6.56	68.26	68.6	54.5	1.0	3.1	-0.3	0.51	0.53
8	14	123.15	-6.94	-55.58	0.03	70.02	-8.66	70.6	56	0.5	-0.4	-3.3	0.48	0.52
8	15	253.60	-8.12	-55.72	0.03	70.20	-10.03	70.9	56.3	1.1	-0.5	-3.3	0.54	0.58
8	16	4.41	63.01	-3.38	0.03	4.07	75.48	75.6	63.1	0.0	3.3	-0.2	0.46	0.48
8	17	116.15	54.39	-4.24	0.03	5.20	68.26	68.5	54.6	0.6	3.1	-0.3	0.46	0.49
8	18	227.90	52.22	-5.46	0.03	6.60	65.63	66	52.5	1.1	2.9	-0.3	0.5	0.52
8	19	149.58	-7.01	-53.90	0.03	67.84	-8.70	68.4	54.3	0.7	-0.4	-3.1	0.48	0.52
8	20	280.03	-8.19	-54.03	0.03	68.01	-10.08	68.8	54.6	1.2	-0.5	-3.1	0.54	0.58
8	21	14.55	61.13	-3.41	0.03	4.09	73.16	73.3	61.2	0.1	3.1	-0.2	0.45	0.47
8	22	126.30	52.65	-4.27	0.03	5.23	66.06	66.3	52.8	0.6	3.0	-0.3	0.45	0.48
8	23	238.04	50.38	-5.50	0.03	6.63	63.40	63.7	50.7	1.1	2.8	-0.3	0.49	0.51
8	24	171.26	-7.05	-52.44	0.03	66.00	-8.73	66.6	52.9	0.8	-0.4	-3.0	0.48	0.51
8	25	301.71	-8.25	-52.57	0.03	66.16	-10.12	66.9	53.2	1.3	-0.5	-3.0	0.54	0.57
8	26	26.93	58.81	-3.45	0.03	4.11	70.33	70.4	58.9	0.1	3.0	-0.2	0.44	0.46
8	27	138.67	50.50	-4.32	0.03	5.26	63.40	63.6	50.7	0.7	2.8	-0.3	0.44	0.47
8	28	250.42	48.15	-5.55	0.03	6.66	60.70	61.1	48.5	1.2	2.6	-0.3	0.48	0.50
8	29	197.69	-7.11	-50.67	0.03	63.78	-8.77	64.4	51.2	0.9	-0.4	-2.9	0.47	0.51
8	30	328.14	-8.32	-50.80	0.03	63.94	-10.17	64.7	51.5	1.4	-0.5	-2.9	0.53	0.57
	Max.	786.60	68.76	25.05	0.04	73.78	82.70	82.80	68.80	3.4	3.8	2.5	0.60	0.64
	Min.	-156.54	-34.20	-58.50	0.03	-19.27	-37.99	27.50	26.20	-0.8	-2.9	-3.5	0.19	0.20

Notes: **X-Axis coincides with the axis of the each pile**

Y-Axis and Z-Axis are perpendicular to the X-Axis and orthogonal to each other

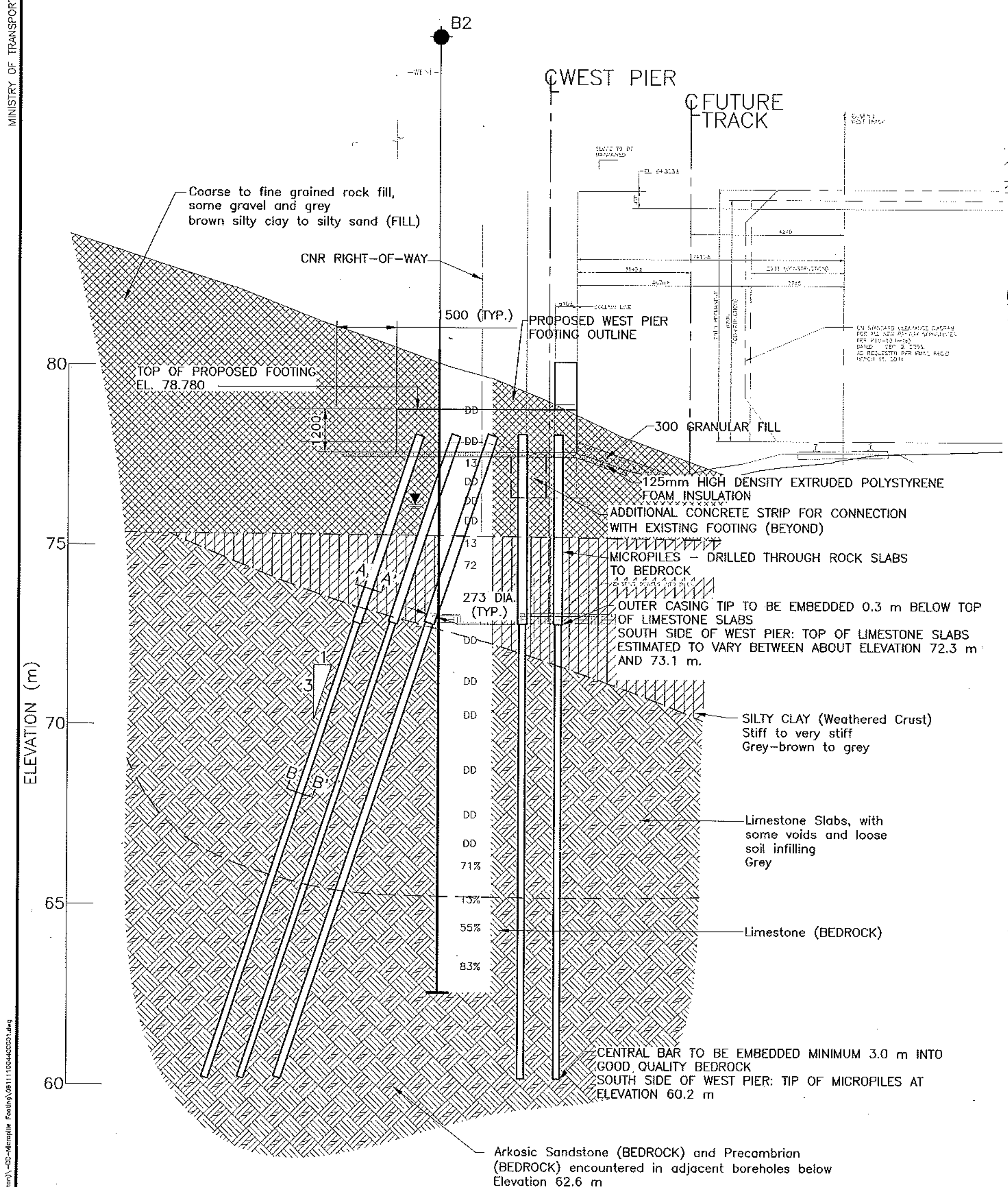
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No.

CNR BRIDGE REHABILITATION AND WIDENING–HIGHWAY 401
MICROPILE GROUP ARRANGEMENT
WEST PIER–SOUTH SIDE

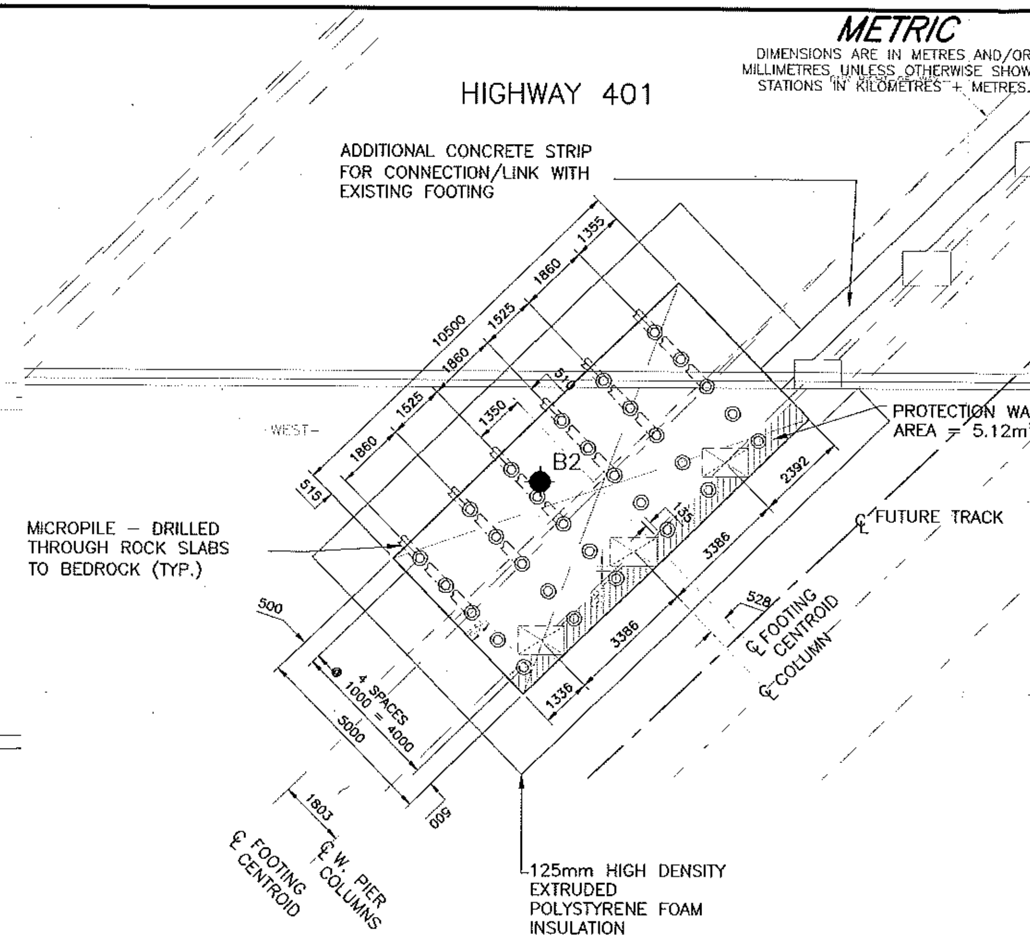
SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



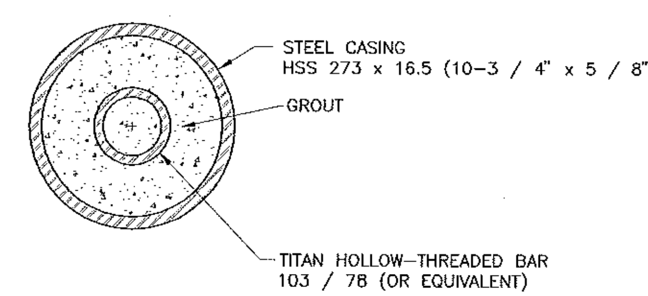
ELEVATION – PROPOSED WEST PIER – SOUTH SIDE

SCALE
1.5 0 1.5 3 m



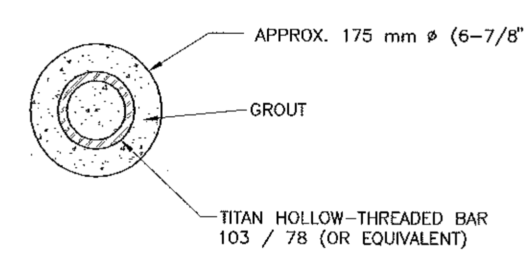
PART PLAN – WEST PIER – SOUTHERN END

SCALE
2 0 2 4 m



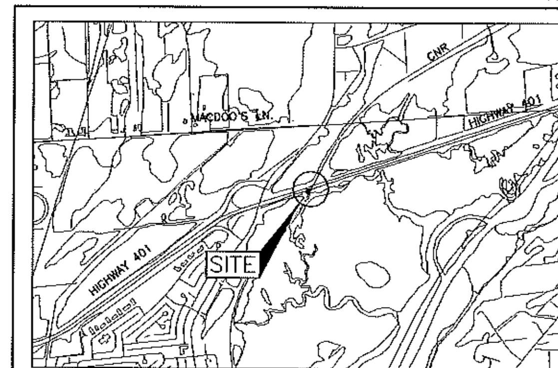
MICROPILE SECTION WITH STEEL CASING A–A'

SCALE
0.1 0 0.1 0.2 m



MICROPILE SECTION WITHOUT STEEL CASING B–B'

SCALE
0.1 0 0.1 0.2 m



KEY PLAN
SCALE
500 0 500 1000m

LEGEND

Symbol	Description
●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on Sept. 29, 2009

NOTES

Elevations are in metres.

The 28–day compressive strength of grout is assumed to be 35MPa.

The yield strength of steel casing and hollow–threaded bar is assumed to be 552 MPa and 570 MPa, respectively.

LEGEND

- Coarse to fine grained rock fill, some gravel and grey brown silty clay to silty sand (FILL)
- SILTY CLAY (Weathered Crust) Stiff to very stiff Grey–brown to grey
- Limestone Slabs, with some voids and loose soil infilling Grey
- Limestone (BEDROCK)

REFERENCE

Base plans provided in digital format by MRC, drawing file nos. 7437–302–Clearance Envelope II.dwg and 7437–302–TMP–PROPOSED–FOOTING–PLAN–WPIER–Aug_09_11.dwg, received Sept. 26, 2011.

NO.	DATE	BY	REVISION

Geocres No. 31C–212

HWY. 401		PROJECT NO. 08–1111–0044	DIST.
SUBM'D. GA	CHKD. JPD	DATE: 11/19/2012	SITE:
DRAWN: JFC	CHKD. GA	APPD. JPD	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

REFERENCE

Base plan provided in digital format by MRC, drawing file no. 7437-302-Clearance Envelope II.dwg and received Sept. 26, 2011.

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

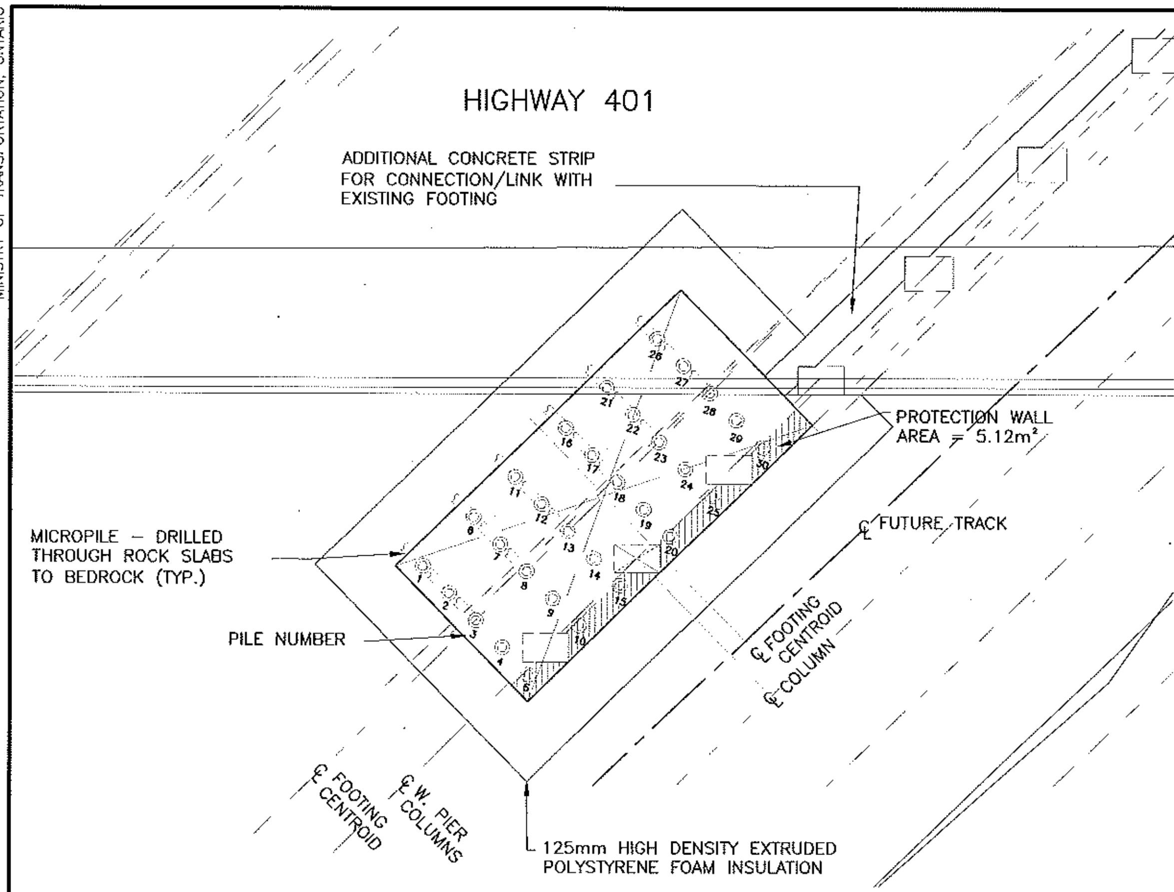
CONT No.
WP No.

SHEET

CNR BRIDGE REHABILITATION AND WIDENING–HIGHWAY 401
BENDING MOMENT, SHEAR FORCE AND DISPLACMENTS–WEST PIER–SOUTH SIDE

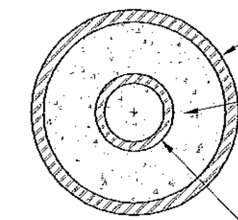
Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



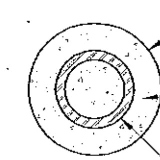
MICROPILE GROUP ARRANGEMENT – WEST PIER – SOUTHERN END

SCALE 2 0 2 4 m



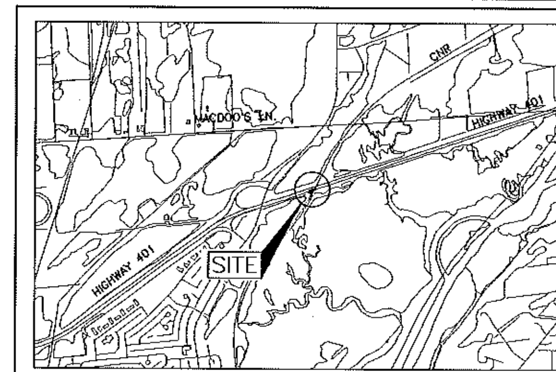
MICROPILE SECTION WITH STEEL CASING A–A'

SCALE 0.1 0 0.1 0.2 m



MICROPILE SECTION WITHOUT STEEL CASING B–B'

SCALE 0.1 0 0.1 0.2 m



KEY PLAN

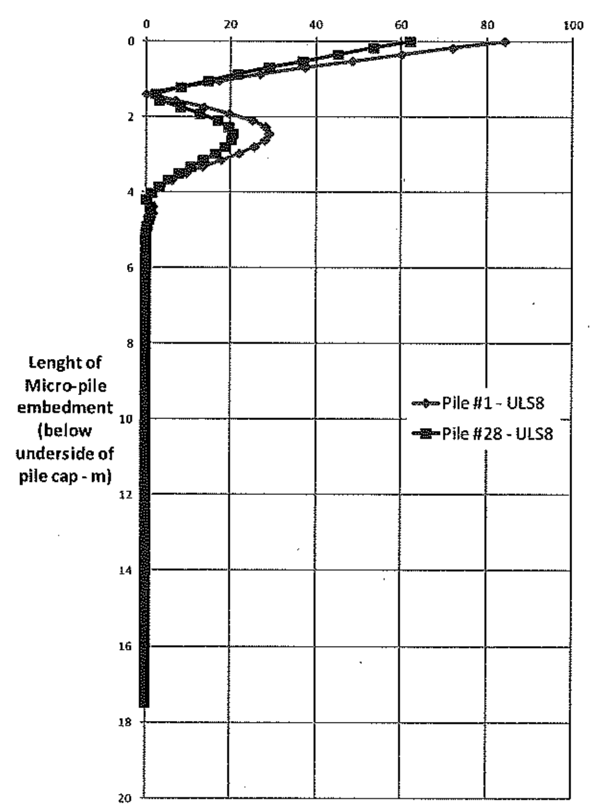
SCALE 500 0 500 1000m

NOTES

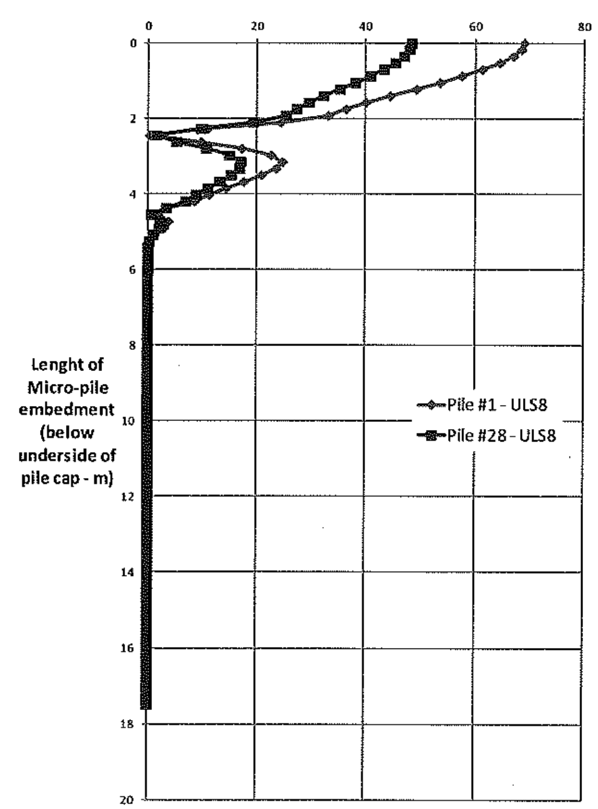
Shear Force and Bending Moments shown represent maximum and minimum values within group.

Maximum Demand over Capacity Ratio calculated in the upper cased section (Section A–A') in accordance with AREMA (2009) is 0.54 Maximum Demand over Capacity Ratio calculated in the lower uncased section (Section B–B') in accordance with AREMA (2009) is 0.64.

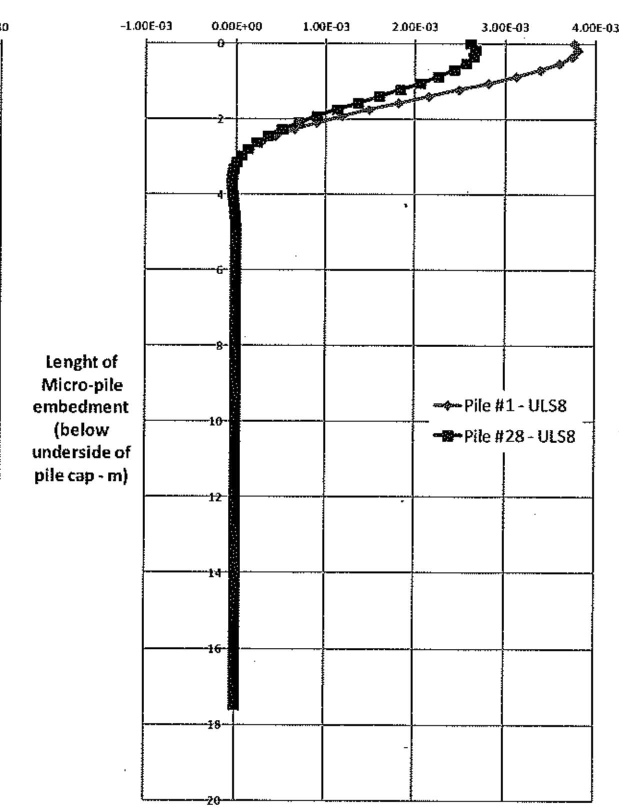
Maximum Axial Force is 787 kN and occurs on Pile #30 for ULS1.



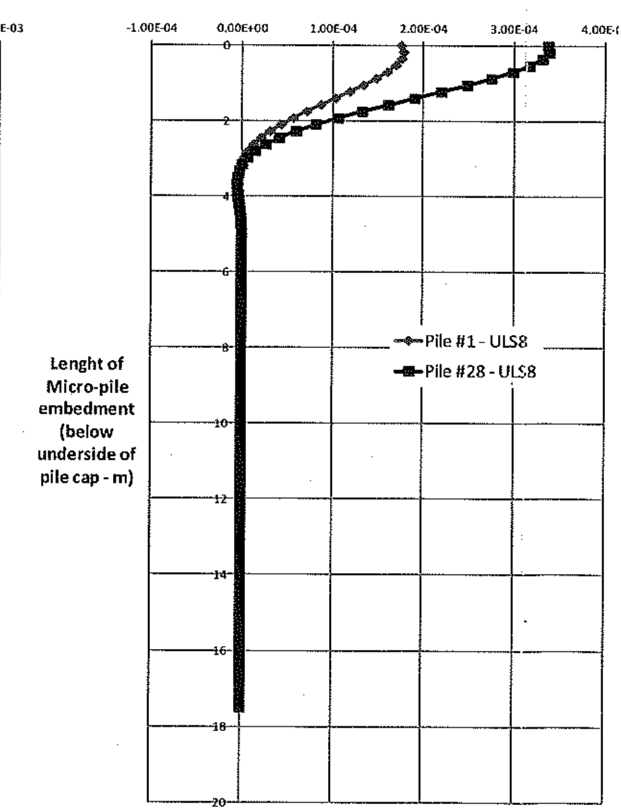
RANGE OF RESULTANT BENDING MOMENT (kNm)



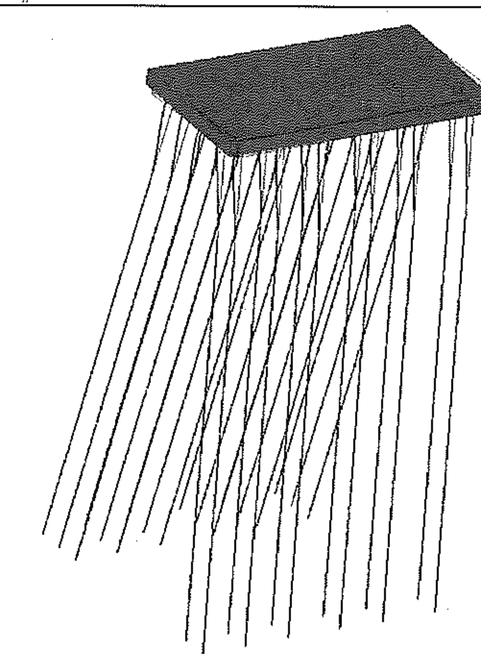
RANGE OF RESULTANT SHEAR FORCE (kN)



RANGE OF LATERAL DISPLACEMENT (m)



RANGE OF LONGITUDINAL DISPLACEMENT (m)



MICROPILES AND MICROPILE CAP DISPLACEMENT – ULS8

** LONGITUDINAL = PARALLEL TO TRACK

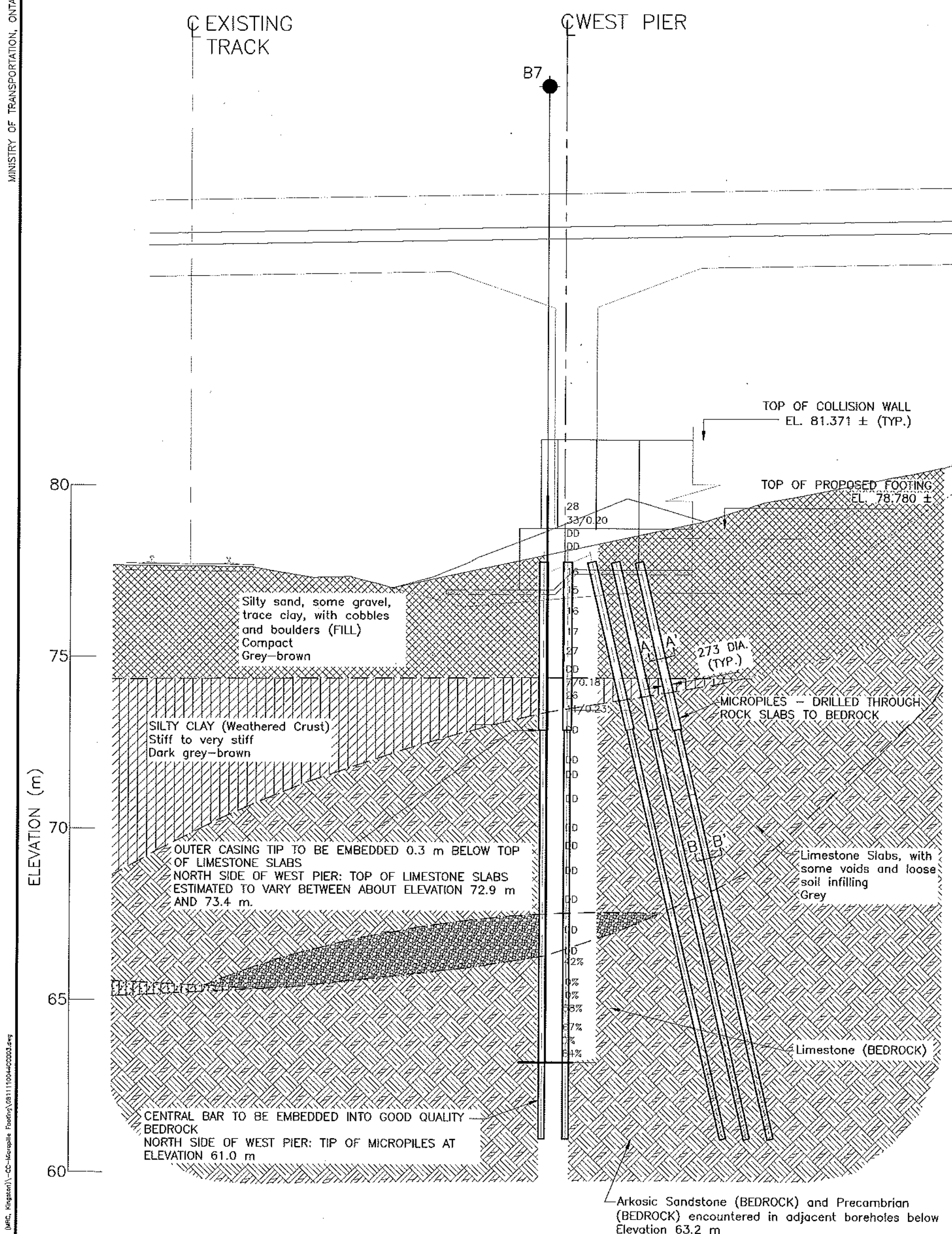
** LATERAL = PERPENDICULAR TO TRACK

NO.	DATE	BY	REVISION

Geocres No. 31C–212

HWY. 401		PROJECT NO. 08–1111–0044	DIST.
SUBM'D. GA	CHKD. JPD	DATE: 11/19/2012	SITE:
DRAWN: JFC	CHKD. GA	APPD. JPD	DWG. 2

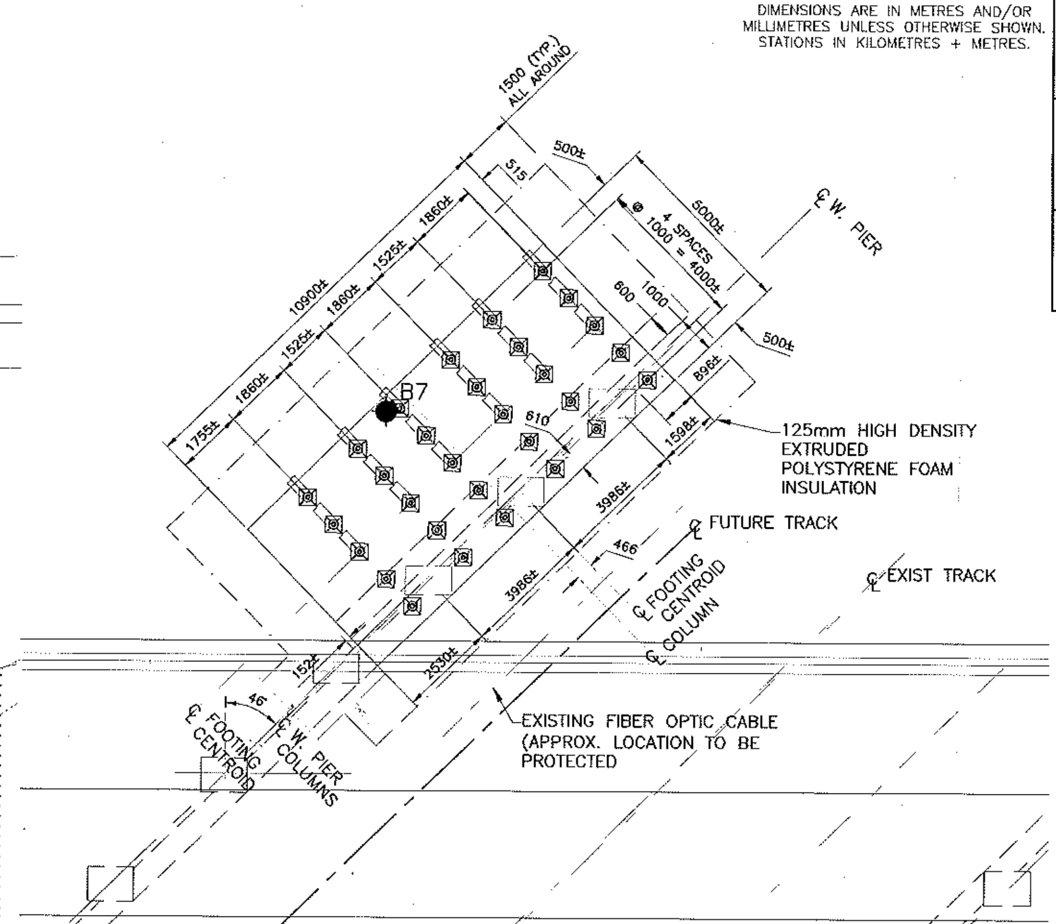
MINISTRY OF TRANSPORTATION, ONTARIO



ELEVATION – PROPOSED WEST PIER – NORTH SIDE

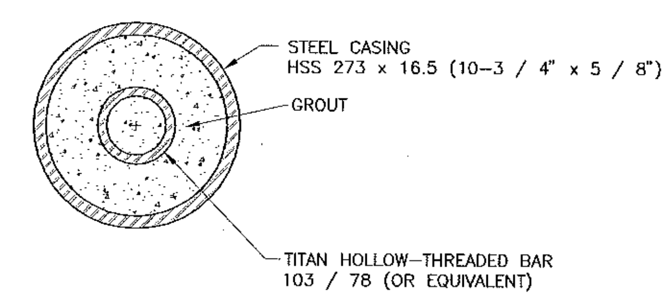
SCALE 1.5 0 1.5 3 m

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



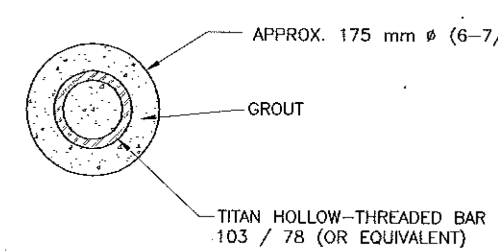
PART PLAN – WEST PIER – NORTHERN END

SCALE 2 0 2 4 m



MICROPILE SECTION WITH STEEL CASING A–A'

SCALE 0.1 0 0.1 0.2 m



MICROPILE SECTION WITHOUT STEEL CASING B–B'

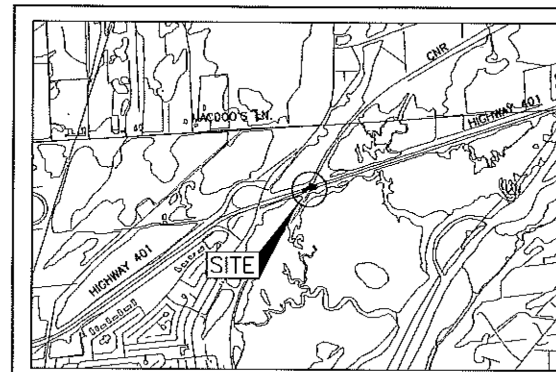
SCALE 0.1 0 0.1 0.2 m

CONT No.
WP No.

CNR BRIDGE REHABILITATION AND WIDENING–HIGHWAY 401
MICROPILE GROUP ARRANGEMENT WEST PIER–NORTH SIDE

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE 500 0 500 1000m

LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on Sept. 29, 2009

NOTES

Elevations are in metres.

The 28–day compressive strength of grout is assumed to be 35MPa.

The yield strength of steel casing and hollow–threaded bar is assumed to be 552 MPa and 570 MPa, respectively.

LEGEND

- Coarse to fine grained rock fill, some gravel and grey brown silty clay to silty sand (FILL)
- SILTY CLAY (Weathered Crust) Stiff to very stiff Grey–brown to grey
- Limestone Slabs, with some voids and loose soil infilling Grey
- Limestone (BEDROCK)

REFERENCE

Base plans provided in digital format by MRC, drawing file nos. 7437–302–Clearance Envelope II.dwg and 7437–302–TMP–PROPOSED–FOOTING–PLAN–WPIER–Aug_09_11.dwg, received Sept. 26, 2011.

NO.	DATE	BY	REVISION

Geocres No. 31C–212

HWY. 401	PROJECT NO. 08–1111–0044		DIST.
SUBM'D. GA	CHKD. JPD	DATE: 11/19/2012	SITE:
DRAWN: JFC	CHKD. GA	APPD. JPD	DWG. 3

PLOT DATE: November 19, 2012

REFERENCE
Base plan provided in digital format by MRC, drawing file no. 7437-302-Clearance Envelope II.dwg and received Sept. 26, 2011.

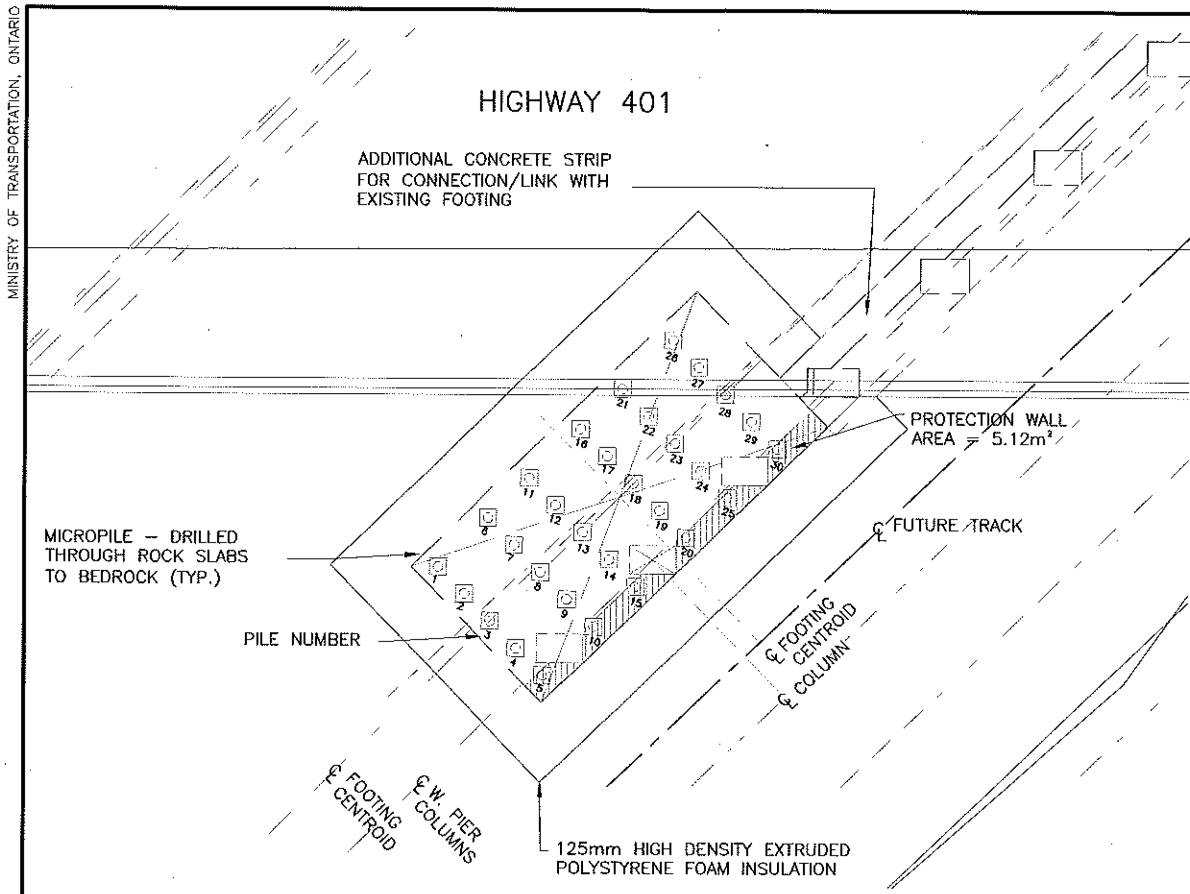
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No.

CNR BRIDGE REHABILITATION AND WIDENING–HIGHWAY 401
BENDING MOMENT, SHEAR FORCE AND DISPLACMENTS–WEST PIER–NORTH SIDE

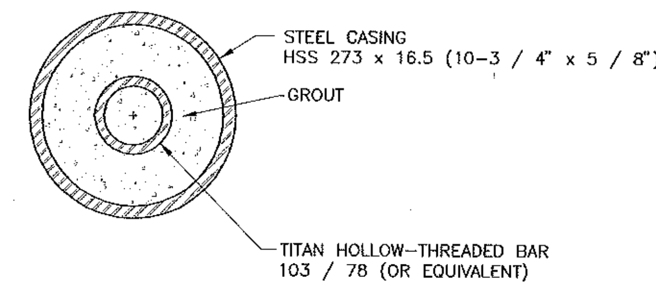
SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



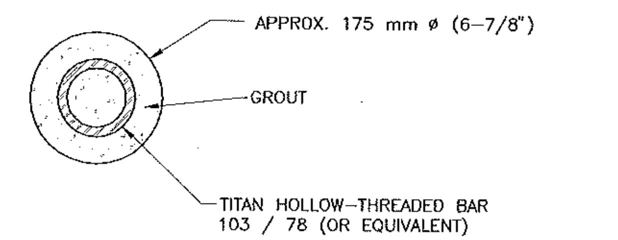
MICROPILE GROUP ARRANGEMENT – WEST PIER – NORTHERN END

SCALE 2 0 2 4 m



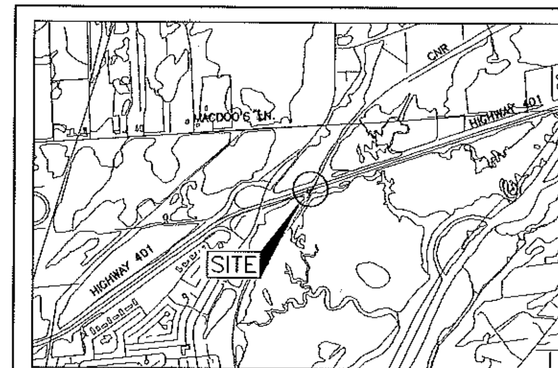
MICROPILE SECTION WITH STEEL CASING A–A'

SCALE 0.1 0 0.1 0.2 m



MICROPILE SECTION WITHOUT STEEL CASING B–B'

SCALE 0.1 0 0.1 0.2 m



KEY PLAN

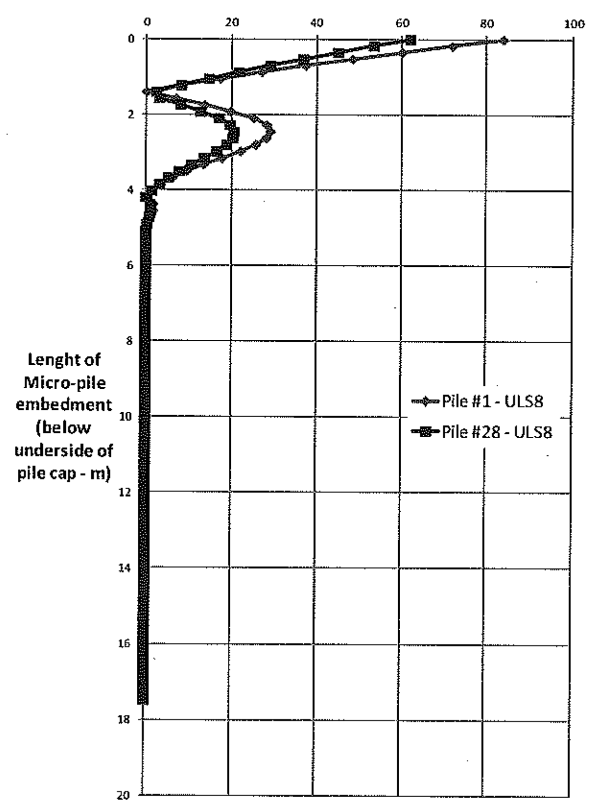
SCALE 500 0 500 1000m

NOTES

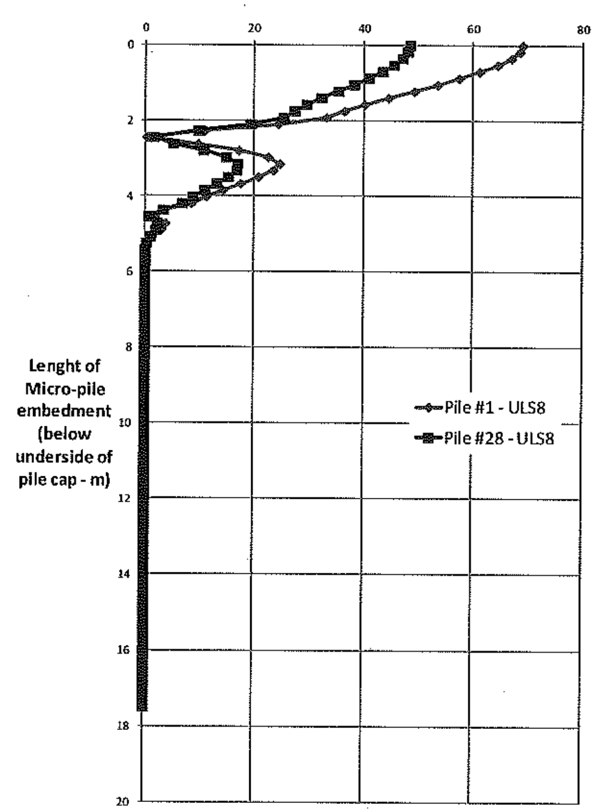
Shear Force and Bending Moments shown represent maximum and minimum values within group.

Maximum Demand over Capacity Ratio calculated in the upper cased section (Section A–A') in accordance with AREMA (2009) is 0.54 Maximum Demand over Capacity Ratio calculated in the lower uncased section (Section B–B') in accordance with AREMA (2009) is 0.64.

Maximum Axial Force is 787 kN and occurs on Pile #30 for ULS1.

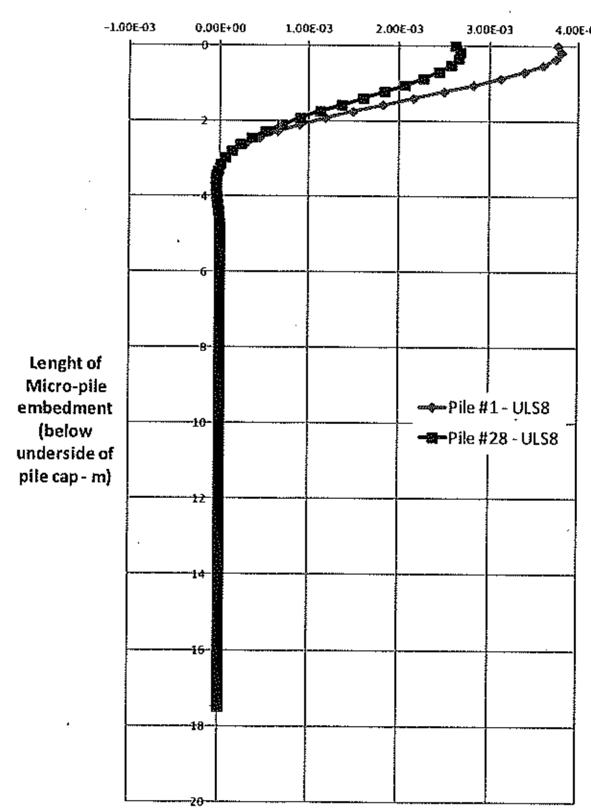


RANGE OF RESULTANT BENDING MOMENT (kNm)

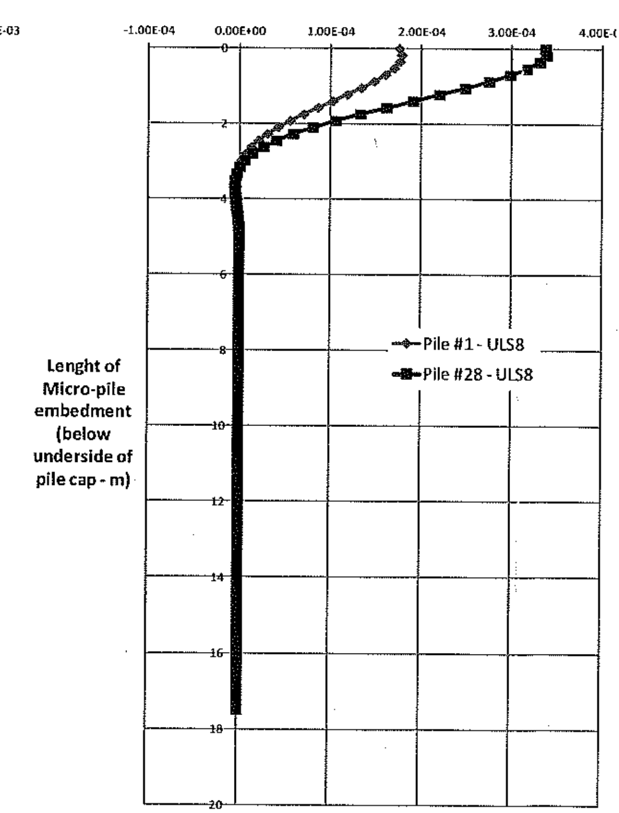


RANGE OF RESULTANT SHEAR FORCE (kN)

** LONGITUDINAL = PARALLEL TO TRACK

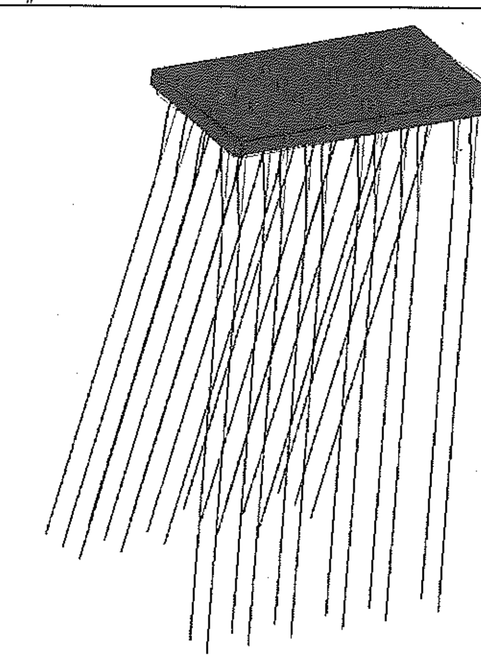


RANGE OF LATERAL DISPLACEMENT (m)



RANGE OF LONGITUDINAL DISPLACEMENT (m)

** LATERAL = PERPENDICULAR TO TRACK



MICROPILES AND MICROPILE CAP DISPLACEMENT – ULS8

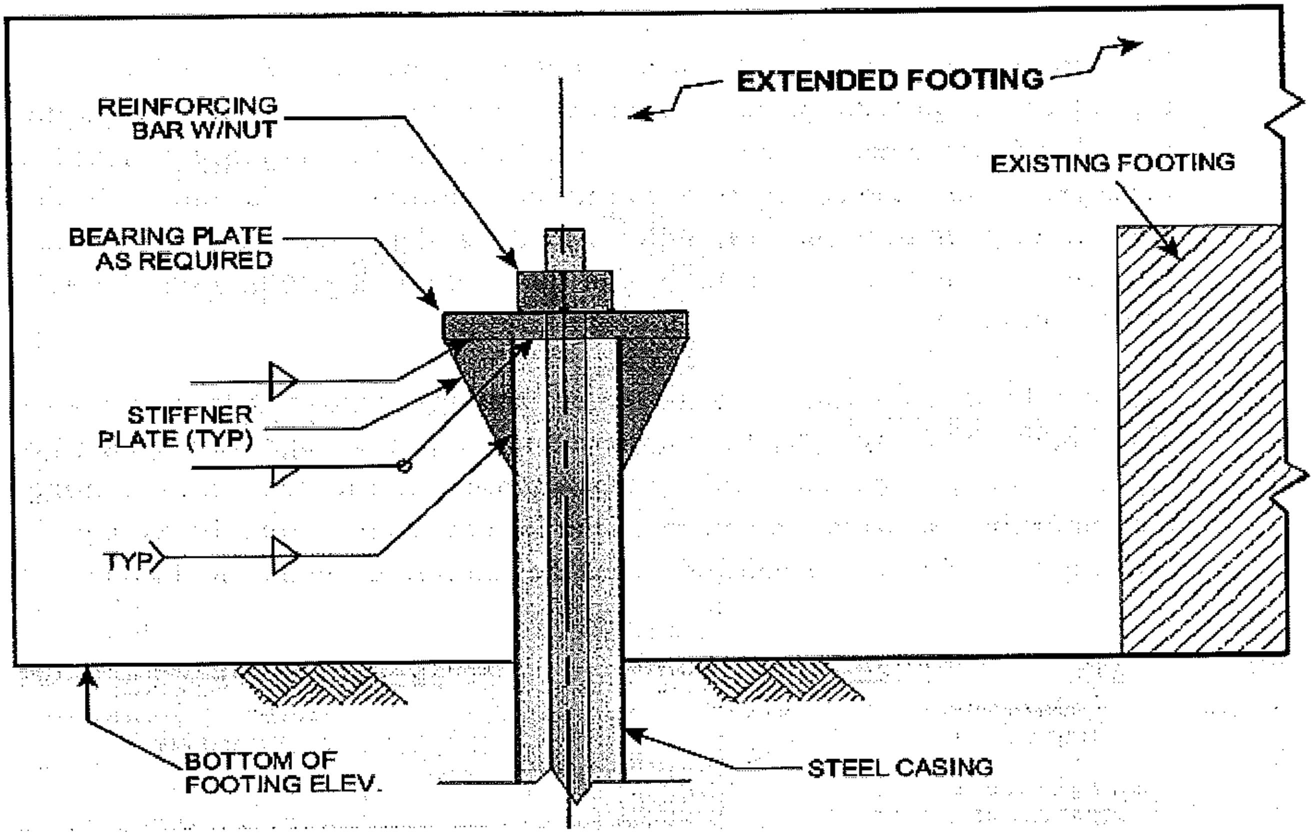
NO.	DATE	BY	REVISION

Geocres No. 31C-212

HWY. 401		PROJECT NO. 08-1111-0044	DIST.
SUBM'D. GA	CHKD. JPD	DATE: 11/19/2012	SITE:
DRAWN: JFC	CHKD. GA	APPD. JPD	DWG. 4

SCHEMATIC OF POSSIBLE MICROPILE CONNECTION DETAIL
CNR BRIDGE REHABILITATION AND WIDENING

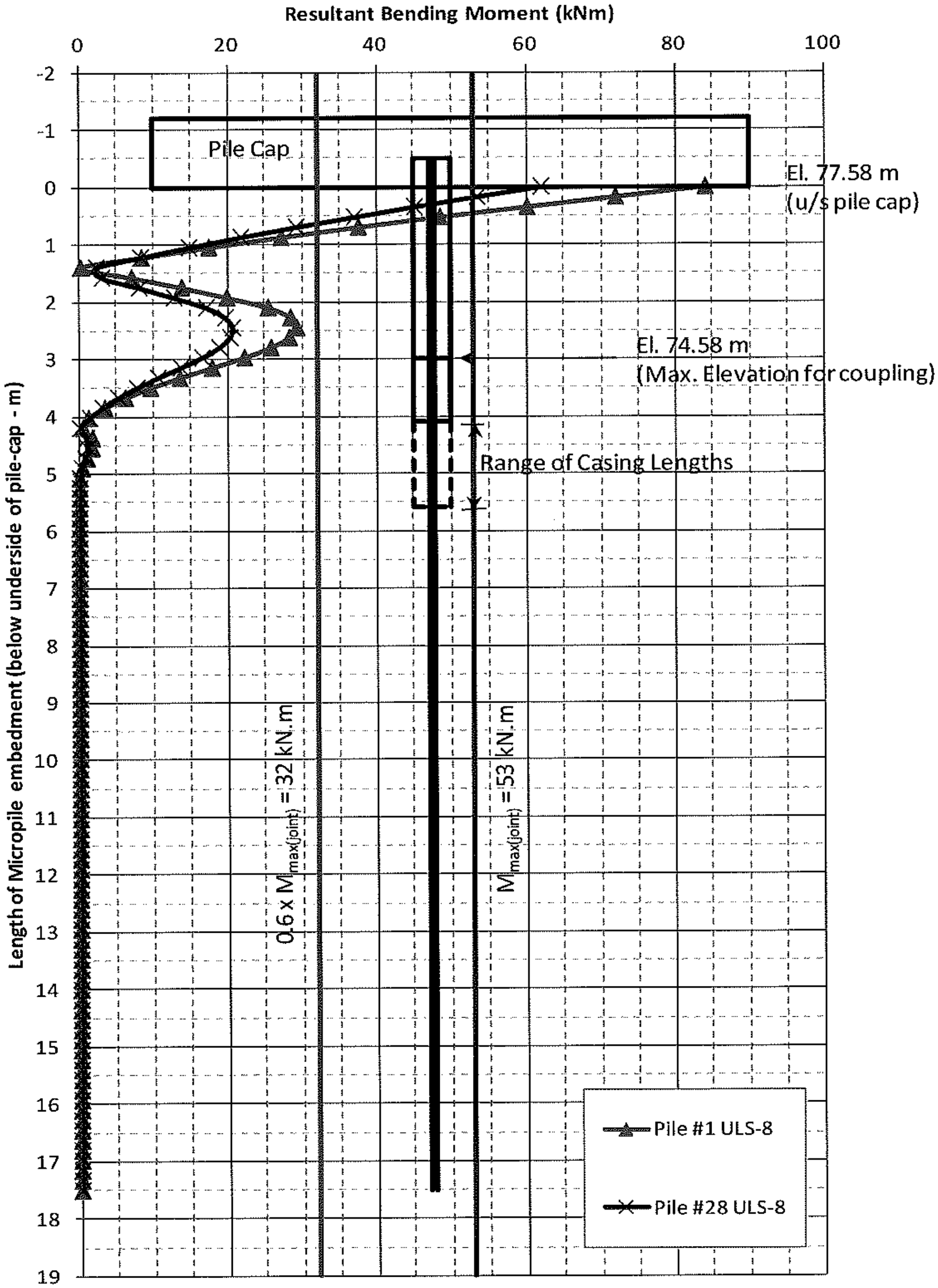
Figure 1



(Adapted from FHWA/NHI 2005)

Date:	November 2012		Drawn:	JPD
Project:	08-1111-0044	**Golder Associates**	Checked:	FJH

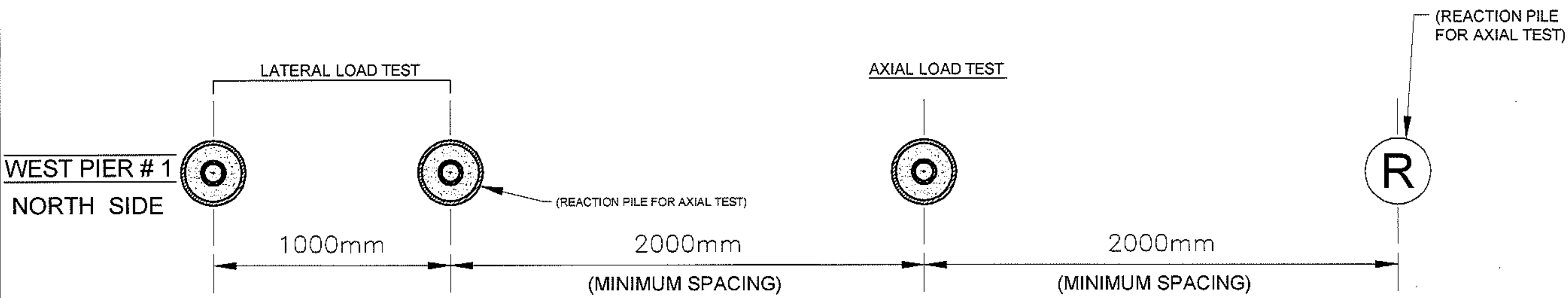
FLUSH JOINT CASING - THREAD LOCATION DETAILS CNR BRIDGE REHABILITATION AND WIDENING	Figure 2



Date: November 2012
Project: 08-1111-0044

Golder Associates

Drawn: JPD
Checked: FJH



NOTE:

1. ALL TEST MICROPILES ARE 273mm DIA. WITH HSS 273 X 16 STEEL CASING AND 103/78 HOLLOW CENTRAL REINFORCING BAR.

0.5 0 0.5 1
SCALE 1:25 METRES

Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN
DATE	Nov. 19, 2012
DESIGN	
CAD	JFC
CHECK	JPD
REVIEW	JPD
FILE No.	0811110044CC03.dwg
PROJECT No.	08-1111-0044 (4050)
REV.	C

TITLE

PROPOSED LOAD TEST PILE ARRANGEMENT

CNR BRIDGE REHABILITATION AND WIDENING - HIGHWAY 401

FIGURE 3

APPENDIX A

Record of Borehole Sheets

Golder Associates

RECORD OF BOREHOLE No B2

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01
LOCATION N 4904273.3 ;E 307205.4
ORIGINATED BY HEC
DIST HWY 401
BOREHOLE TYPE Portable Equipment, Continuous Sampling, NW, BW, AW, EW Casing, Wash Boring
COMPILED BY JM
DATUM Geodetic
DATE August 26 - September 1, 2009
CHECKED BY KSL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
80.4	GROUND SURFACE				
0.0	Rock FILL				
79.2 / 1.2	Coarse grained rock fill, some gravel and grey-brown silty clay (FILL) Moist	1	BW RC	DD	
		2	BW RC	DD	
77.5	Silty clay (FILL) Very stiff Grey-brown Moist	3	SS	13	
77.1 / 3.4	Fine grained rock fill, trace sand, some grey-brown silty clay (FILL) Moist	4	AW RC	DD	
		5	AW RC	DD	
		6	AW RC	DD	
75.4 / 5.1	SILTY CLAY (Weathered Crust) Stiff to very stiff Grey-brown to grey Moist	7	SS	13	
		8	SS	72	
		9	SS	28	
73.1 / 7.3	LIMESTONE SLABS, with some voids and soil infilling Grey	10	RC	DD	
		11	RC	DD	
70.4	Void or loose soil	12	RC	DD	
	Void or loose soil				
10.3	LIMESTONE SLABS, with some voids and soil infilling Grey				
69.3	Void or loose soil				
11.3	LIMESTONE SLABS, with some voids and soil infilling Grey	13	RC	DD	
		14	RC	DD	
		15	RC	DD	
66.2 / 14.3		C1	RC	REC 97%	RQD = 71%

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (80 to 66); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; GR SA SI CL

Continued Next Page

+ ³, × ³ Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No B2

 METRIC

PROJECT 08-1111-0044
G.W.P. 78-99-01
LOCATION N 4904273.3 ;E 307205.4
ORIGINATED BY HEC
DIST HWY 401
BOREHOLE TYPE Portable Equipment, Continuous Sampling, NW, BW, AW, EW Casing, Wash Boring
COMPILED BY JM
DATUM Geodetic
DATE August 26 - September 1, 2009
CHECKED BY KSL

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.2 15.2	LIMESTONE (BEDROCK) Fresh Thinly bedded Weak Greenish-grey	C1	RC			RQD = 71%
		C2	RC	REC 71%	65	RQD = 13%
64.3 16.1	LIMESTONE (BEDROCK) Fresh to weathered Thinly bedded Weak Greenish-grey and reddish-grey	C3	RC	REC 83%	64	RQD = 55%
63.2 17.2	LIMESTONE (BEDROCK) Fresh Thinly bedded Weak Reddish-grey	C4	RC	REC 100%	63	RQD = 83%
62.6 17.9	LIMESTONE (BEDROCK) Fresh Thinly bedded Medium strong Grey Note: Bedrock cored between 14.3 m and 17.9 m depth. For bedrock coring details refer to Record of Drillhole B2. End of Borehole Note: Water level in well screen at 4.3 m depth (Elev. 76.1) on Sept. 29, 2009.					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: B2

PROJECT: 08-1111-0044
LOCATION: N 4904273.3 ;E 307205.4
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 26 - September 1, 2009
DRILL RIG: Portable
DRILLING CONTRACTOR: OGS

SHEET 1 OF 1
DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No
	Continued from Record of Borehole B2	66.10	
15	LIMESTONE (BEDROCK) Fresh Thinly bedded Weak Greenish-grey	14.30 / 65.20	C1
16	LIMESTONE (BEDROCK) Fresh to weathered Thinly bedded Weak Greenish-grey and reddish-grey	15.20 / 64.30	C2
17	LIMESTONE (BEDROCK) Fresh Thinly bedded Weak Reddish-grey	16.10 / 63.20	C3
18	LIMESTONE (BEDROCK) Fresh Thinly bedded Medium strong Grey	17.20 / 62.50 / 17.90	C4
	End of Drillhole		

Other column headings: DRILLING RECORD; SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); RQD %; FRACT INDEX PER 0.3; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec; DIAMETRAL POINT LOAD INDEX (MPa); NOTES WATER LEVELS INSTRUMENTATION

Depth scale: 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29

MIS-RCK 001 08-1111-0044 (ROCK).GPJ GAL-MISS.GDT 12/3/10 DD

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: HEC
CHECKED: KSL

Golder Associates

Foundation Design

RECORD OF BOREHOLE No B7

 METRIC

PROJECT 08-1111-0044

G.W.P. 78-99-01

LOCATION N 4904318.2 ;E 307227.7

ORIGINATED BY DG

DIST ____ HWY 401

BOREHOLE TYPE Portable Equipment, Continuous Sampling, AW, BW Casing, Wash Boring

COMPILED BY JM

DATUM Geodetic

DATE June 19, 2009

CHECKED BY KSL

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
79.7	GROUND SURFACE				
0.0	Sand and gravel, trace silt (FILL) Compact Grey-brown Moist	1	SS	28	
		2	SS	33/0.20	
78.9	Rock FILL Grey	3	AW RC	DD	
1.0	Silty sand, some gravel, trace clay, with cobbles and boulders (FILL) Compact Grey-brown Moist to wet	4	AW RC	DD	
		5	SS	16	
		6	SS	15	
		7	SS	16	
75.9 / 3.8	Silt, some clay, trace sand (FILL) Grey-brown Wet	8	SS	17	0 9 67 24
75.2 / 4.6	Silty sand, with cobbles and boulders (FILL) Grey Wet	9	SS	27	
		10	AW RC	DD	
74.4 / 5.3	SILTY CLAY, trace gravel (Weathered Crust) Very stiff Grey-brown	11	SS	7/0.18	
		12	SS	26	
		13	SS	11/0.23	
73.4	LIMESTONE SLABS				
73.1	VOID or loose soil	14	AW RC	DD	
72.7	LIMESTONE SLAB				
72.3 / 7.4	VOID or loose soil	15	AW RC	DD	
71.7	LIMESTONE SLAB	16	AW RC	DD	
71.4	VOID or loose soil				
71.0	LIMESTONE SLABS	17	AW RC	DD	
70.6	VOID or loose soil				
70.3	LIMESTONE SLABS				
	VOID or loose soil	18	AW RC	DD	
	LIMESTONE SLABS				
69.6	VOID or loose soil	19	AW RC	DD	
	VOID or loose soil				
10.5	From 6.3m to 12.2m depth: LIMESTONE SLABS, with numerous voids and occasional inclined bedding planes Grey	20	AW RC	DD	
		21	AW RC	DD	
67.6 / 12.2	Grey COBBLES, BOULDERS and red brown SILTY CLAY	22	EW RC	DD	
66.3		23	EW RC	DD	
13.4	LIMESTONE (BEDROCK) Fractured Medium strong Grey to reddish brown	C1	EW RC	REC 84%	RQD = 42.3%
65.7 / 14.1		~~C2~~	~~EW RC~~	REC 100%	~~RQD = 0%~~
		C3	EW RC	REC 76.3%	RQD = 0%
		C4	EW RC	REC 100%	RQD = 58.1%

Elevation scale: 79, 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100

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

WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No B7

 METRIC

PROJECT 08-1111-0044

G.W.P. 78-99-01

LOCATION N 4904318.2 ;E 307227.7

ORIGINATED BY DG

DIST HWY 401

BOREHOLE TYPE Portable Equipment, Continuous Sampling, AW, BW Casing, Wash Boring

COMPILED BY JM

DATUM Geodetic

DATE June 19, 2009

CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
	LIMESTONE (BEDROCK) Fresh Thinly bedded Medium strong Grey to reddish brown		C4	EW RC	REC 100%						RQD = 58.1%
			C5	EW RC	REC 100%						RQD = 66.7%
			C6	EW RC	REC 100%		64				RQD = 0%
	Note: Bedrock cored between 13.4 m and 16.5 m depth. For bedrock coring details refer to Record of Drillhole B7.		C7	EW RC	REC 100%						RQD = 83.9%
63.2 16.5	End of Borehole										

MIS-MTO 001 08-1111-0044.GPJ GAL-MISS.GDT 12/3/10 DD

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

PROJECT: 08-1111-0044

LOCATION: N 4904318.2 ;E 307227.7

INCLINATION: -90°   AZIMUTH: ---

RECORD OF DRILLHOLE: B7

DRILLING DATE: June 19, 2009

DRILL RIG: Portable

DRILLING CONTRACTOR: OGS

SHEET 1 OF 1

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
	Continued from Record of Borehole B7	66.30		
	LIMESTONE (BEDROCK) Fractured Medium strong Grey to reddish brown	13.40	C1	
14		65.60 / 14.10	C2	
	LIMESTONE (BEDROCK) Fresh Thinly bedded Medium strong Grey to reddish brown		C3	
15			C4	
			C5	
			C6	
16			C7	
		63.20 / 16.50		
	End of Drillhole			
17				
18				
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				

Other column headings: DRILLING RECORD; SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); RQD %; FRACT. INDEX PER 0.3; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa)

DEPTH SCALE
1 : 75

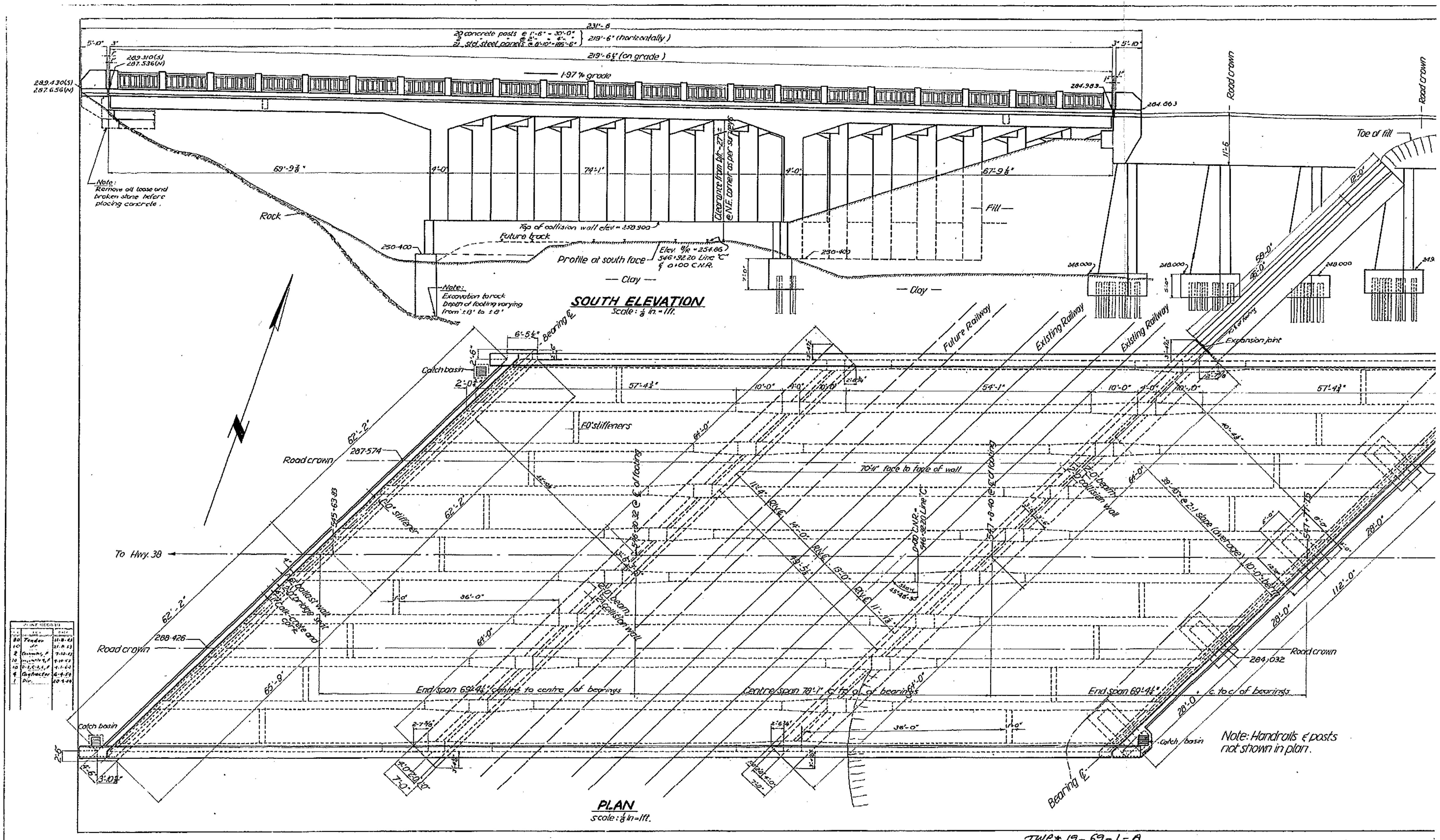
Golder Associates

LOGGED: DG
CHECKED: KSL

MIS-RCK 001 08-1111-0044 (ROCK).GPJ GAL-MISS.GDT 12/3/10 DD

APPENDIX B

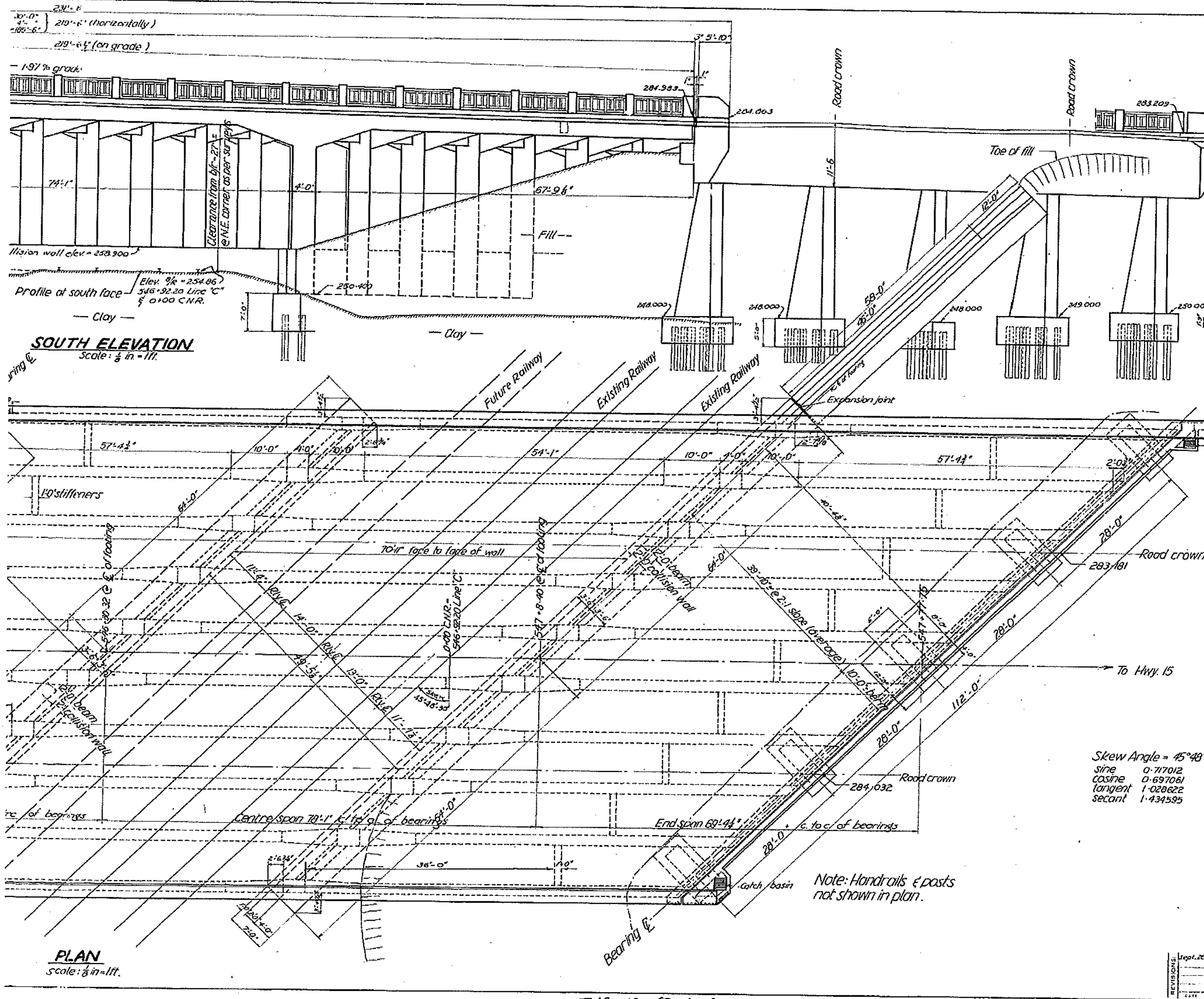
Original Foundation Drawings (from Department of Highways, Ontario)

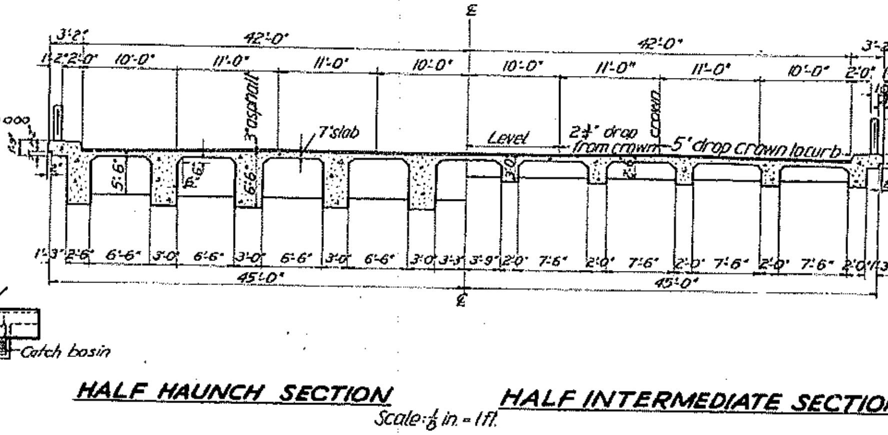


1

NEGATIVE DUE

Notes:
All footings excavation to be neat to given dimensions. Footings to be poured tight against natural ground.
All exposed edges are to be chamfered.
Curbs and handrail posts are not to be poured until the falsework for the deck has been removed.





Note to Division Engineer
Concrete work on this structure must not be commenced until monuments to fix control points have been erected and checked by the Division Engineer.

Note to Contractor
Structure to be built in accordance with the "General Specifications for Highway Bridges, Ontario, 1935," Forms No. 9, and the Special Specifications attached to the "Information to Bidders" sheet, extra copies of which may be obtained from the Division Engineer.

During construction the Contractor shall not erect any falsework within 6'-0" of the rails in the horizontal direction nor within 22'-6" of the base of the rail in the vertical direction unless written permission has been obtained from the Railway Company involved.

Concrete Mix
All concrete to be $1:1\frac{3}{4}:3\frac{1}{2}$.
Add $\frac{1}{4}$ lb. Pozzolith "2A" per bag of cement.

KINGSTON TWP. BR. #20

DEPARTMENT OF HIGHWAYS - ONTARIO
BRIDGE OFFICE - TORONTO

GRADE SEPARATION
OF C.N.R. AND HWY. 401 (Line 'C')
NEAR KINGSTON

THE KING'S HIGHWAY No. 401 (C.A.H.) DIV. No. 8
CO. FRONTENAC
TWP. KINGSTON LOT 12 CON FRONTING CATARAQUI R.

PLAN AND [redacted]
7-69
D3349
1 to 11

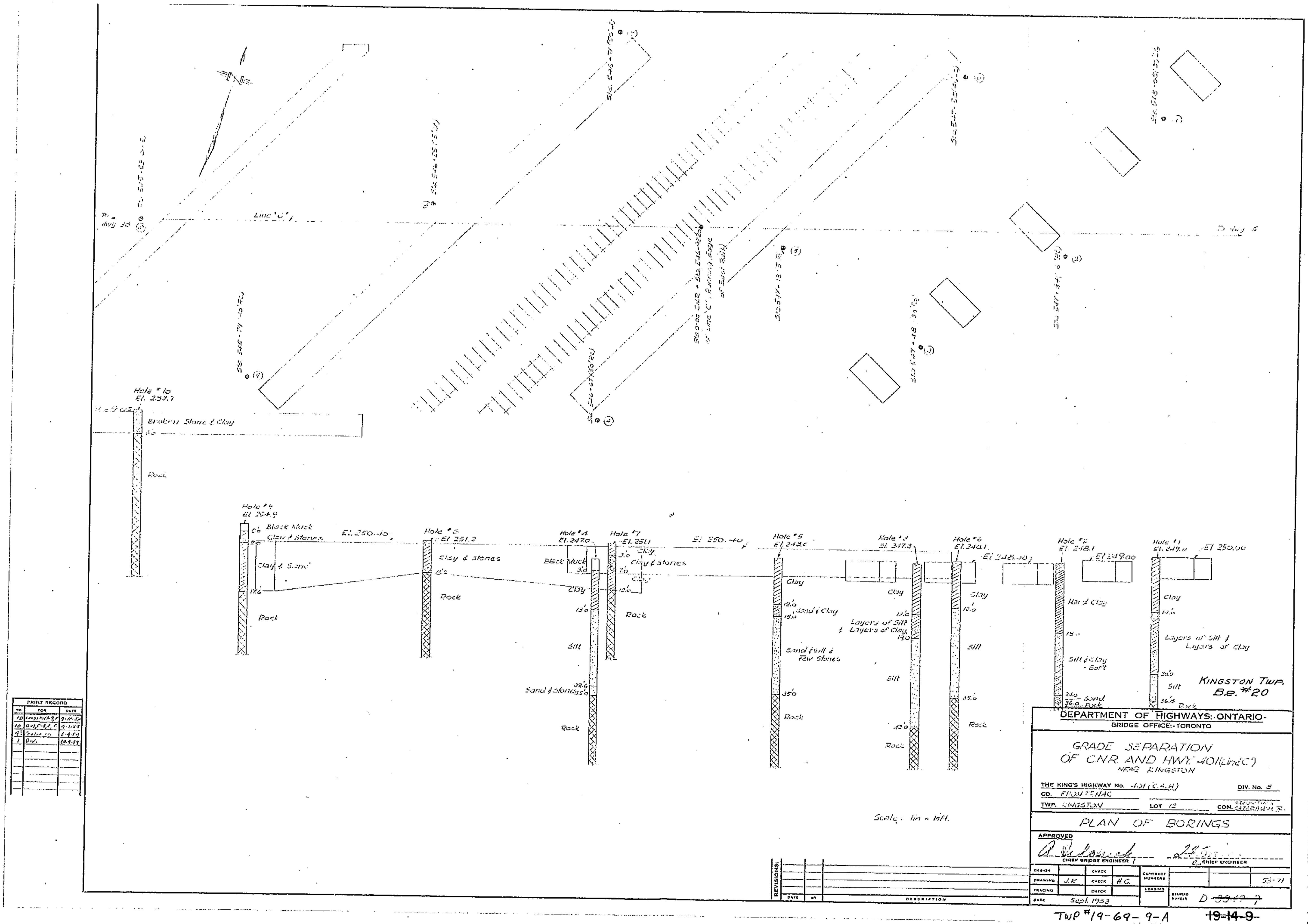
APPROVED
CHIEF BRIDGE ENGINEER
CHIEF ENGINEER

DESIGN	H.H.H.	CHECK	H.G.	CONTRACT NUMBERS	
DRAWING	H.H.H.	CHECK	H.G.		53-71
TRACING	H.G.	CHECK	H.H.H.	LOADING H20 S16-44	DRAWING NUMBER ~~D-3349-1~~
DATE	April 1953				

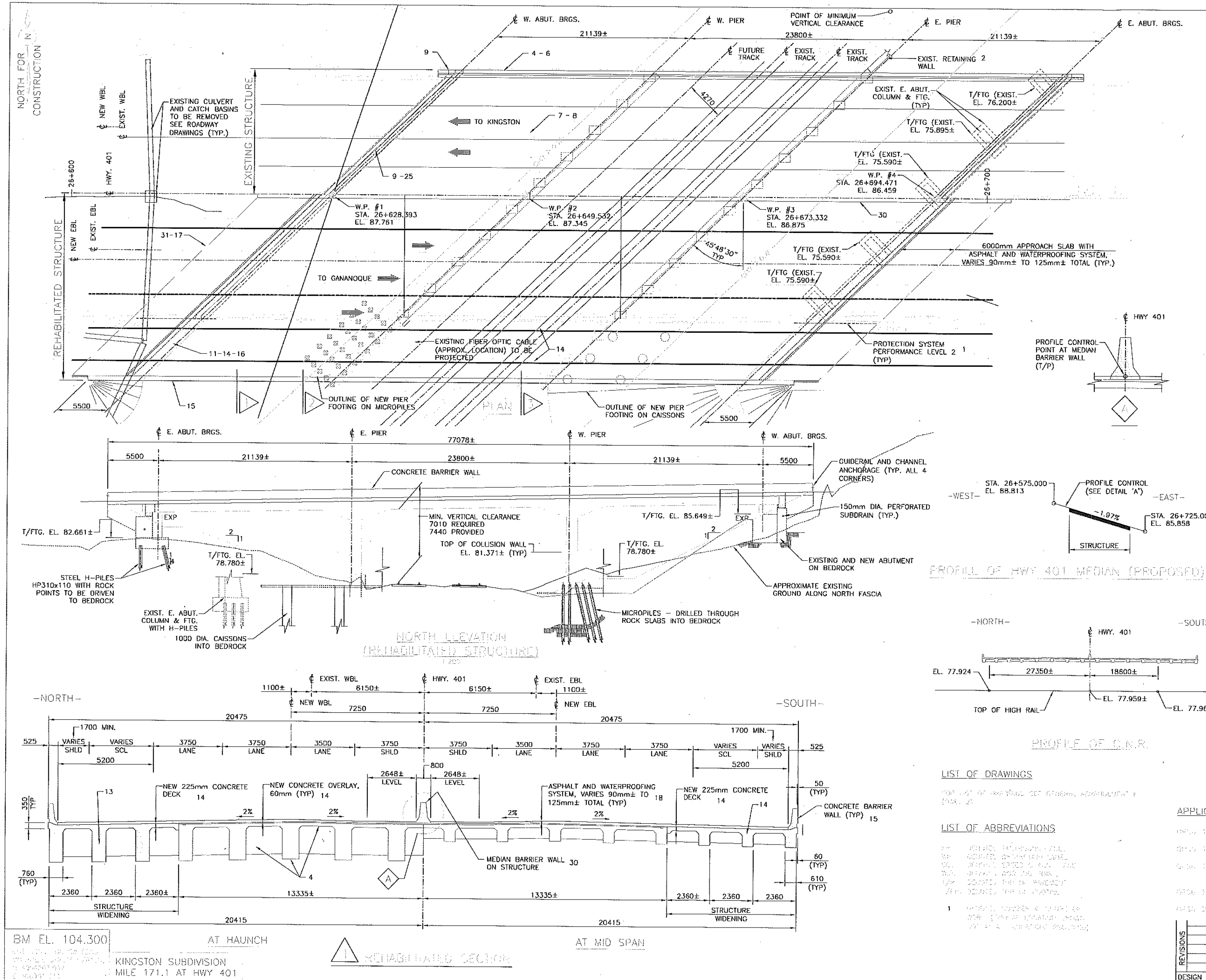
REVISIONS: Sept. 28 H.G. East pier & East abutment footings

TWP # 19-69-1-A ~~D-19-14-1~~

TWP # 19-69-1-A



KINGSTON TWP.
B.e. #20
DEPARTMENT OF HIGHWAYS:-ONTARIO-
BRIDGE OFFICE:-TORONTO
GRADE SEPARATION
OF C.N.R. AND HWY. 401 (Line C)
NEAR KINGSTON
THE KING'S HIGHWAY No. 401 (C.A.H.)
DIV. No. 3
CO. FRONTENAC
TWP. KINGSTON
LOT 12
PLAN OF BORINGS
APPROVED
CHIEF BRIDGE ENGINEER
CHIEF ENGINEER
DESIGN
DRAWING J.K.
CHECK H.G.
TRACING
DATE Sept. 1953
CONTRACT NUMBERS 53-71
D 33-17-7
TWP #19-69-9-A
19-14-9
Scale: 1in = 10ft.
PRINT RECORD
REVISIONS:
DATE
BY
DESCRIPTION
Line 'C'
Hole #10 El. 253.7
Broken Stone & Clay
Rock
Hole #9 El. 254.9
Black Muck
Clay & Stones
Clay & Sand
Rock
El. 250.10
Hole #8 El. 251.2
Clay & Stones
Rock
Hole #4 El. 247.0
Black Muck
Clay
Silt
Sand & Stones
Rock
Hole #7 El. 251.1
Clay & Stones
Rock
El. 250.40
Hole #5 El. 248.0
Clay
Sand & Clay
Sand & Silt & Few Stones
Rock
Hole #3 El. 247.3
Clay
Layers of Silt & Layers of Clay
Silt
Rock
Hole #6 El. 248.1
El. 248.00
Clay
Silt
Rock
Hole #2 El. 248.1
El. 249.00
Hard Clay
Silt & Clay - Soft
Sand
Rock
Hole #1 El. 249.8
El. 250.00
Clay
Layers of Silt & Layers of Clay
Silt
Rock



DISTRICT

CONT. No.

WP No. 4015-06-01

C.N.R. OVERHEAD AT HWY 401 WIDENING AND REHABILITATION

GENERAL ARRANGEMENT I (NORTH ELEVATION)

SHEET

McCORMICK RANKIN A member of MMMGROUP

METRIC

NOTE

GENERAL NOTES

1. CLASS OF CONCRETE

2. CLEAR COVER TO REINFORCING STEEL

3. REINFORCING STEEL

4. CONSTRUCTION NOTES

LIST OF DRAWINGS

LIST OF ABBREVIATIONS

APPLICABLE STANDARD DRAWINGS

BM EL. 104.300

KINGSTON SUBDIVISION MILE 171.1 AT HWY 401

REVISIONS	DESCRIPTION			
DESIGN JM	CHK MV	CODE CHBDC 2006	LOAD CL-625-ONT	DATE OCT 12
DRAWN RD/GC	CHK JM	SITE 7-69/R1	STRUCT SCHEME	DWG 1

CAD FILE LOCATION AND NAME: T:\Projects\2008\08-1111-0044 (MRC, Kingston)\XDC\20121029 Micropiles\7437-302-N-ELEV_FOR-MICROPILES.dwg
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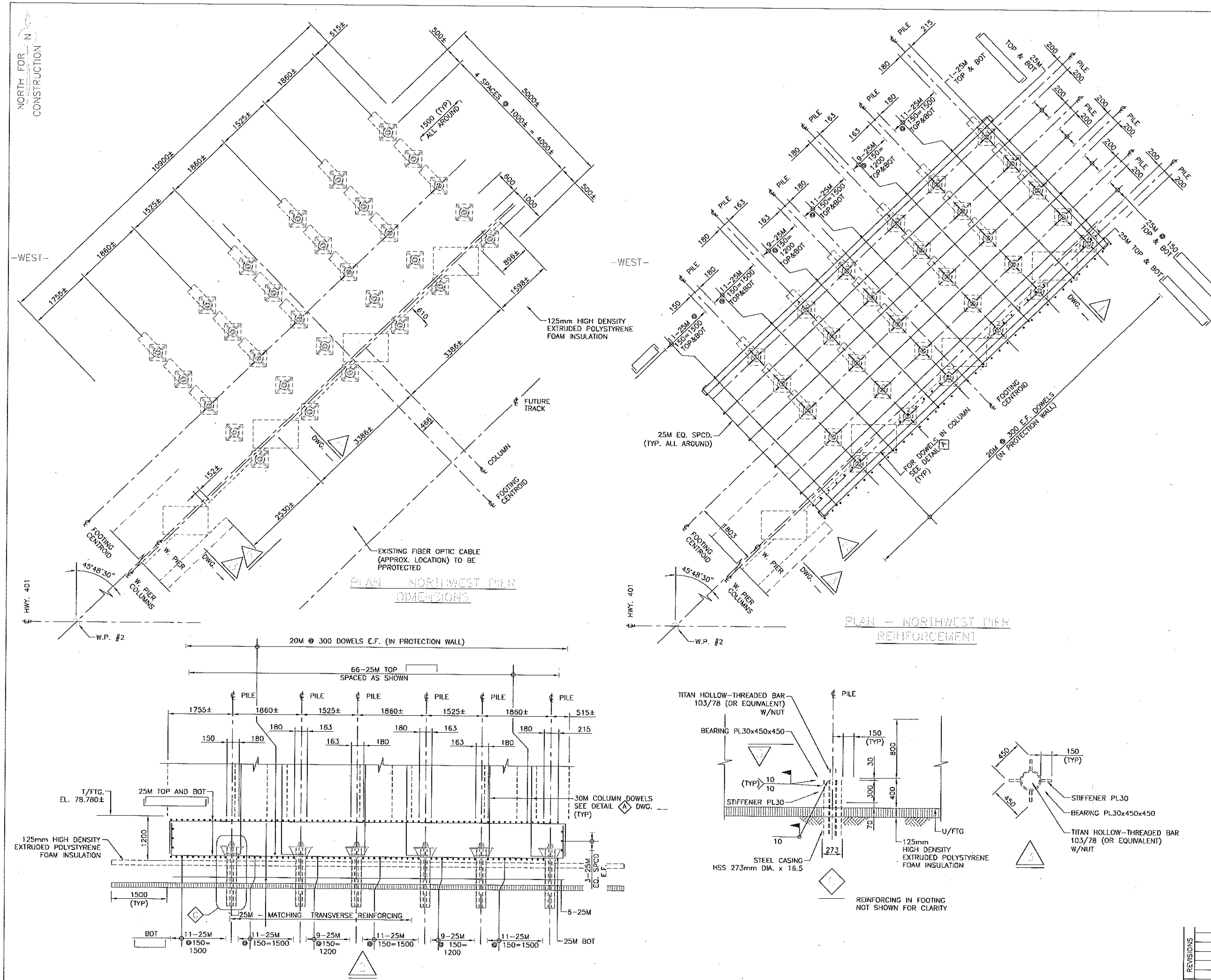
DATE PLOTTED: 10/29/2012 2:15:32 PM BY: WHEATONM



DISTRICT

CONT. No.

WP No. 4015-06-01

C.N.R. OVERHEAD AT HWY 401 WIDENING AND REHABILITATION

FOUNDATION III NW PIER

SHEET

McCORMICK RANKIN A member of MMM GROUP

METRIC

NOTE

LIST OF ABBREVIATIONS

REVISIONS	DESCRIPTION

DESIGN JM	CHK MV	CODE CHBDC 2006	LOAD CL-625-ONT	DATE OCT 12
DRAWN BG	CHK JM	SITE 7-69/R1	STRUCT	SCHEME	DWG 16

CAD FILE LOCATION AND NAME: T:\Projects\2008\08-1111-0044 (MRC, Kingston)\XDC\20121029 Micropiles\7437-302-018-CNR_FOUNDATION_III_NW_PIER.dwg
MODIFIED: 10/26/2012 12:15:30 PM BY: WHEATONM
DATE PLOTTED: 10/29/2012 2:17:23 PM BY: WHEATONM

LIST OF ABBREVIATIONS

W.P. DENOTES WORKING POINT
T/P DENOTES TOP OF PAVEMENT
T/FTG DENOTES TOP OF FOOTING
U/FTG DENOTES UNDERSIDE OF FOOTING
E.F. DENOTES EACH FACE
BOT DENOTES BOTTOM
SIM. DENOTES SIMILAR

DISTRICT
CONT. No.
WP No. 4015-06-01

C.N.R. OVERHEAD AT HWY 401 WIDENING AND REHABILITATION

FOUNDATION IV
SW PIER

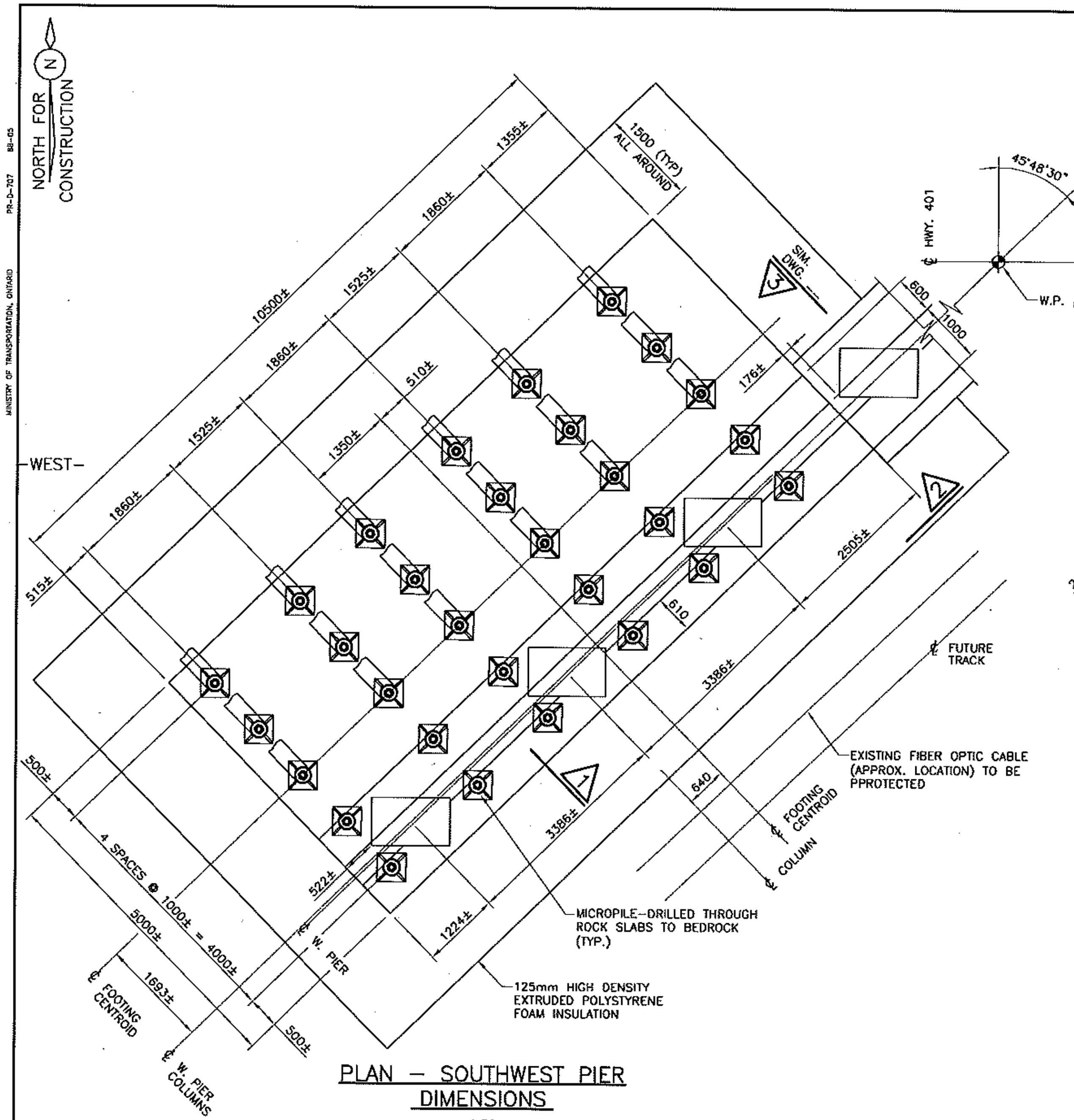
SHEET

McCORMICK RANKIN
A member of MMM GROUP

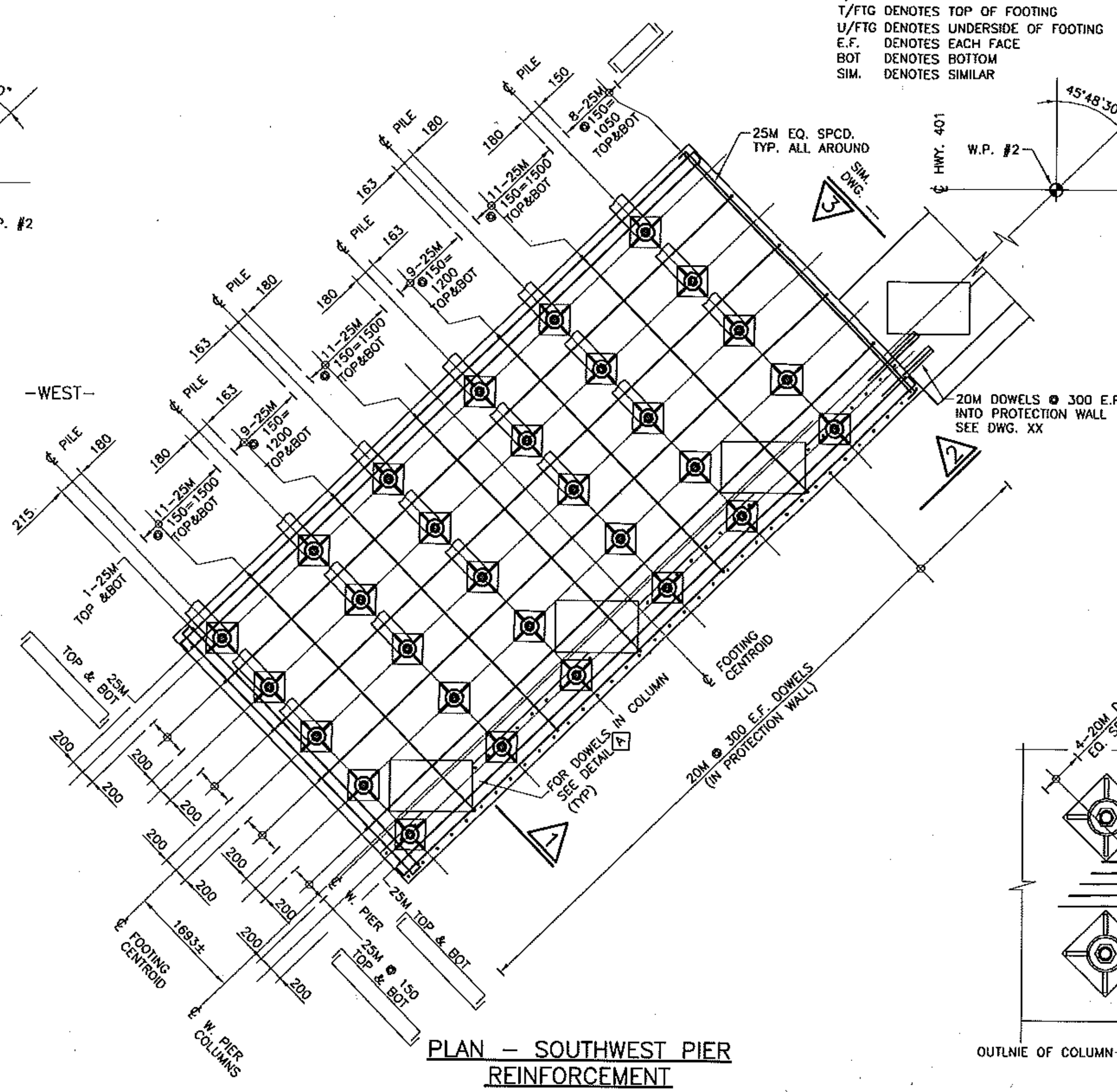
METRIC

NOTE

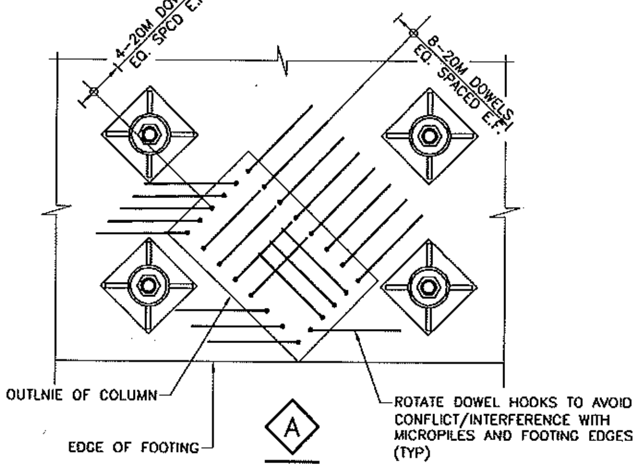
1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH DWG. 2, XX, XX AND XX
2. DETAILS, DIMENSIONS AND ELEVATIONS OF EXISTING STRUCTURE ARE BASED ON ORIGINAL AND REHABILITATION DRAWINGS AND CONSEQUENTLY, THEY ARE APPROXIMATE. THE CONTRACTOR SHALL VERIFY ALL DETAILS, DIMENSIONS AND ELEVATIONS AGAINST EXISTING FIELD CONDITIONS AND ALSO BETWEEN NORTH AND SOUTH SIDES AND SHALL INFORM THE CONTRACT ADMINISTRATOR OF ANY DISCREPANCIES BEFORE COMMENCEMENT OF THE MICROPILE WORK (SEE PECIFICATIONS).
3. THE CONTRACTOR MAY CONSIDER SUBMITTING A PROPOSAL FOR ADJUSTING AND/OR SPLICING THE MAIN REINFORCING STEEL IN THE FOOTINGS TO ACCOMMODATE COLUMN DOWELS TO AVOID CONFLICTS/INTERFERENCES.



PLAN – SOUTHWEST PIER
DIMENSIONS
1:50



PLAN – SOUTHWEST PIER
REINFORCEMENT
1:50



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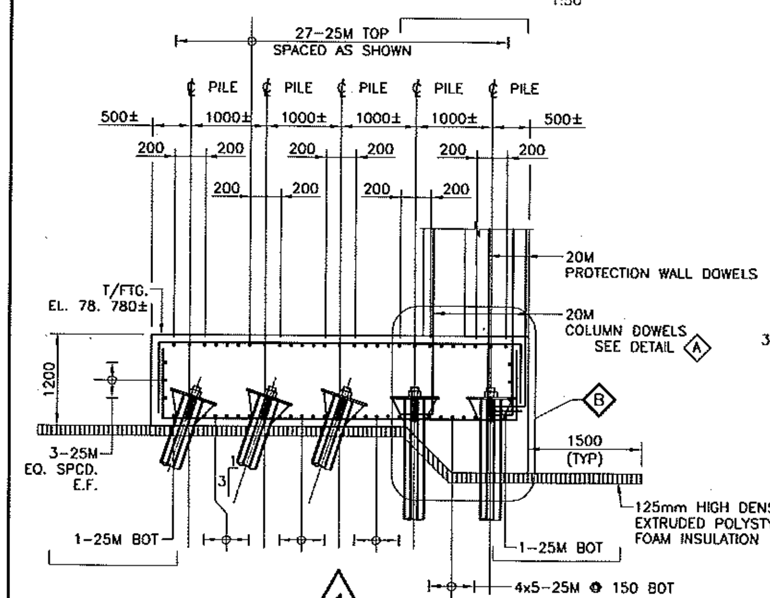

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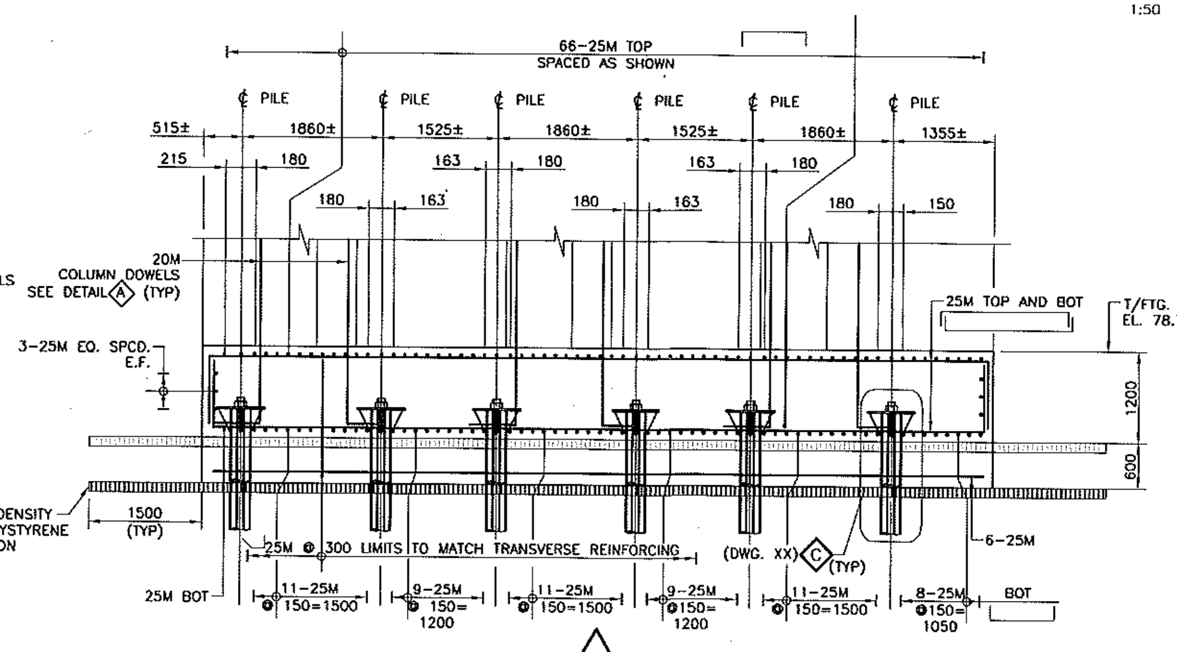

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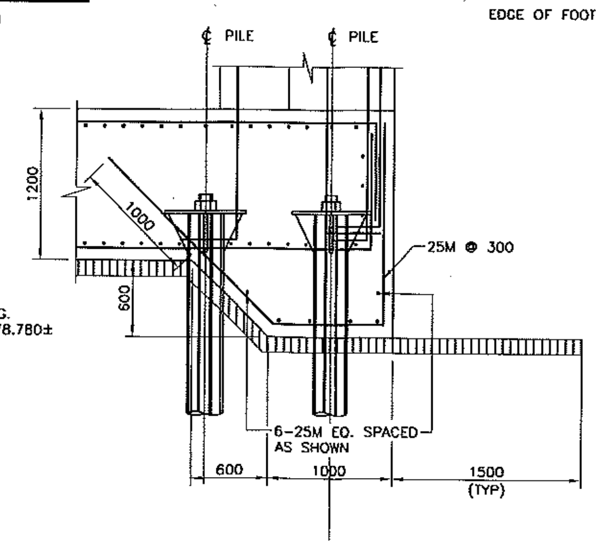

B
1:30

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS	DESCRIPTION			
DESIGN JDM	CHK JM	CODE CHBDC 2006	LOAD CL-625-ONT	DATE OCT 12
DRAWN BG	CHK JDM	SITE 7-69/R1	STRUCT	SCHEME	DWG 18

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DATE PLOTTED: 11/19/2012 4:37:03 PM BY: WHEATONM

DISTRICT

CONT. No.

WP No. 4015-06-01

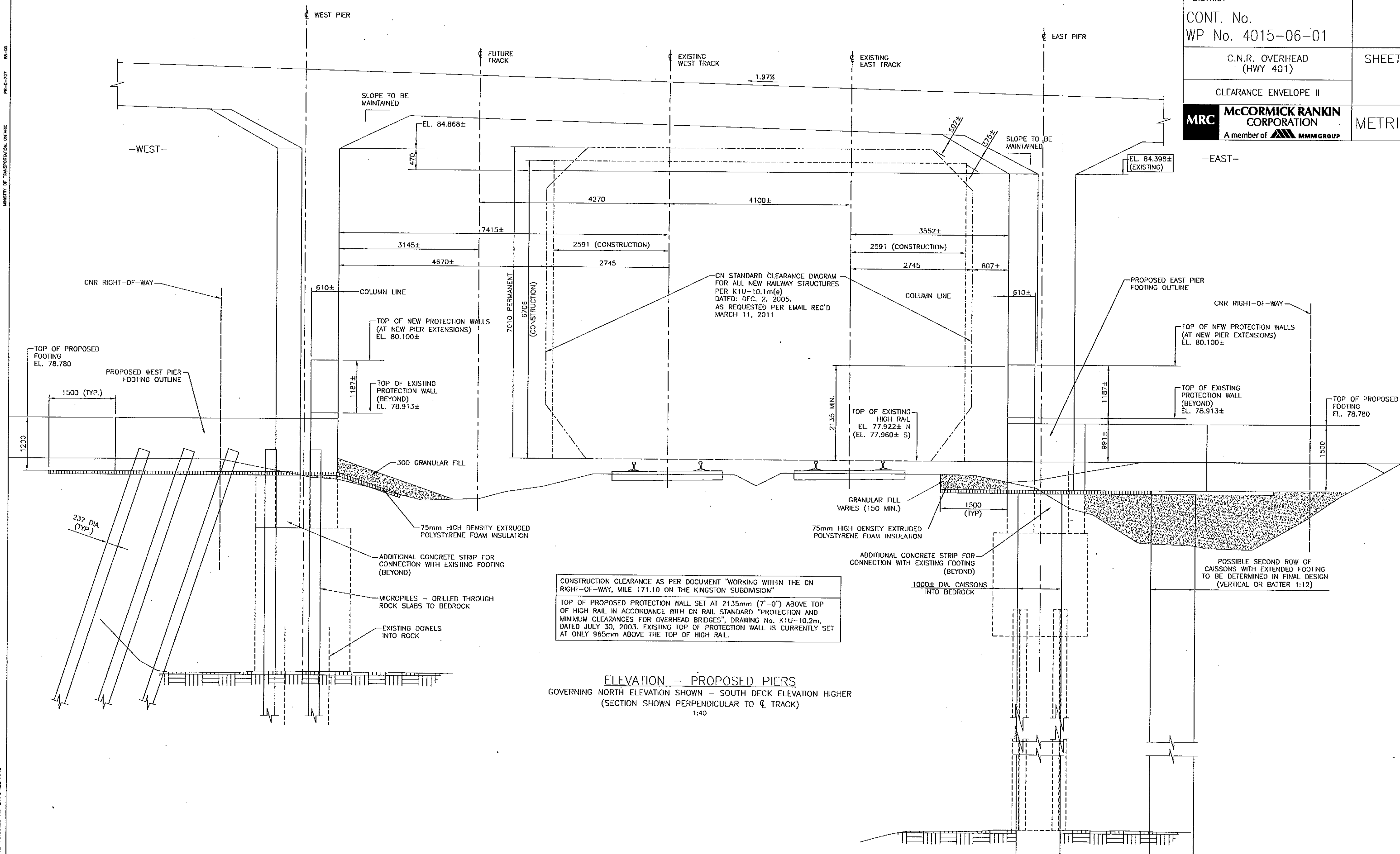
C.N.R. OVERHEAD (HWY 401)

SHEET

CLEARANCE ENVELOPE II

MRC McCORMICK RANKIN CORPORATION A member of MMM GROUP

METRIC



ELEVATION – PROPOSED PIERS

GOVERNING NORTH ELEVATION SHOWN – SOUTH DECK ELEVATION HIGHER (SECTION SHOWN PERPENDICULAR TO ℄ TRACK)

1:40

NOTES:

1. THE MINIMUM STANDARD CLEARANCES SHOWN ARE BASED ON DIAGRAM No. 1 OF CN RAIL SPC 2103 "TRACK CENTRES AND CLEARANCES" DATED APR. 2005, AND MINIMUM STANDARD CLEARANCES FOR OVERHEAD BRIDGES SHOWN ON CN RAIL STANDARD DRAWING K1U-10.2m DATED JULY 30, 2003. TEMPORARY CONSTRUCTION CLEARANCES ARE NOT SHOWN AND SHALL BE COORDINATED WITH THE SYSTEM ENGINEER.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS		
		DESCRIPTION

DESIGN JM	CHK	CODE CHBDC 2006	LOAD CL-625-ONT	DATE JAN 11	
DRAWN TJN	CHK JM	SITE 7-69/R1	STRUCT	SCHEME	DWG

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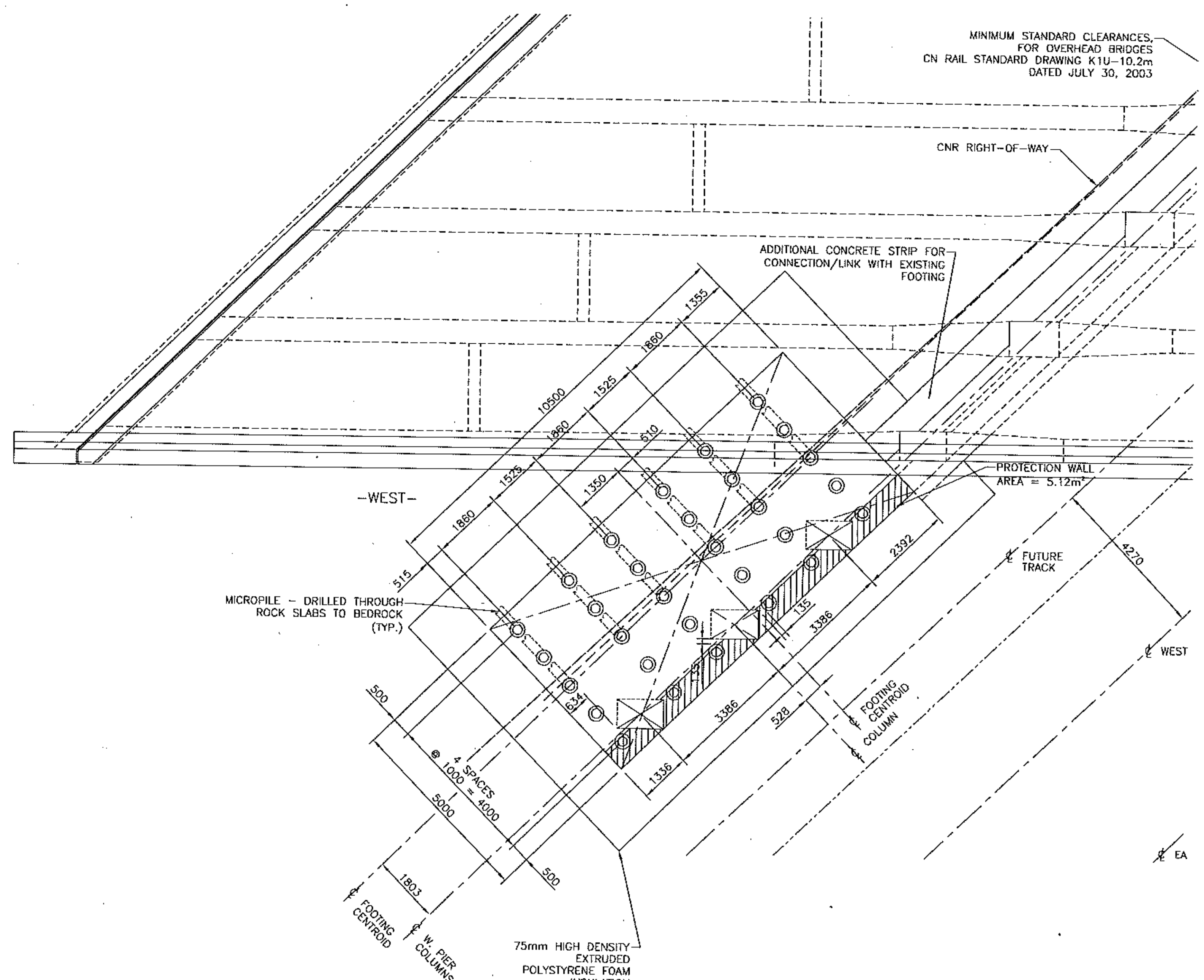
MODIFIED: 9/23/2011 8:49:31 AM BY: GREENANG
DATE PLOTTED: 1/11/2012 10:50:59 AM BY: GREENANG

DISTRICT	
CONT. No. WP No. 4015-06-01	
C.N.R. OVERHEAD (HWY 401)	SHEET
PROPOSE PIER FOOTINGS II	
MRC McCORMICK RANKIN CORPORATION A member of MMM GROUP	METRIC

NOTES:

1. THE MINIMUM STANDARD CLEARANCES SHOWN ARE BASED ON DIAGRAM No. 1 OF CN RAIL SPC 2103 "TRACK CENTRES AND CLEARANCES" DATED APR. 2005, AND MINIMUM STANDARD CLEARANCES FOR OVERHEAD BRIDGES SHOWN ON CN RAIL STANDARD DRAWING K1U-10.2m DATED JULY 30, 2003. TEMPORARY CONSTRUCTION CLEARANCES ARE NOT SHOWN AND SHALL BE COORDINATED WITH THE SYSTEM ENGINEER.



PART PLAN – W. PIER

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REVISIONS			DESCRIPTION		
DESIGN JM	CHK	CODE CHBDC 2006	LOAD CL-625-ONT	DATE JAN 11	
DRAWN TJN	CHK JM	SITE 7-69/R1	STRUCT	SCHEME	DWG

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-707 88-05

CAD FILE LOCATION AND NAME: T:\Projects\2008\08-1111-0044 (MRC, Kingston)\XC\MRC\20110926\7437-302-TMP-PROPOSED-FOOTING-PLAN-WPIER-Aug_09_11.dwg
MODIFIED: 9/23/2011 8:47:18 AM BY: GREENANG
DATE PLOTTED: 1/11/2012 10:51:08 AM BY: GREENANG

APPENDIX C

Selected Contract Drawings (From McCormick Rankin)

APPENDIX D

NSSP – Technical Specification for Grouted Micropiles

Golder Associates

SUPPLY EQUIPMENT FOR INSTALLING MICROPILES - Item No.
PRE-PRODUCTION MICROPILE - Item No.
PRODUCTION MICROPILE - Item No.
PRODUCTION MICROPILE TESTING - Item No.
GROUT FOR MICROPILES – Item No.

Non-Standard Special Provision October 2012

1.0 SCOPE

This Special Provision covers the requirements for the installation and testing of grouted micropiles at west pier for the widening of the existing Highway 401 structure over CNR line (Mile 171.10 of the Kingston Subdivision), located approximately 310 m east of the Montreal Street interchange in Kingston, Ontario, as shown on the Contract Drawings.

1.01 Qualifications of the Contractor

The minimum pre-qualification requirements of the micropile Contractor are specified as follows:

i. The Contractor shall be fully experienced in all aspects of micropile construction and with the execution of pile load tests. The Contractor shall demonstrate that he has successfully completed at least three (3) projects in the previous five (5) years of similar scope, complexity and size.

ii. The micropile superintendent, micropile project manager and the drill and grout operators responsible for installation of the micropile system must have micropile installation experience on at least 3 successfully completed projects over the past 5 years. The Contractor shall provide resumes of key personnel who will be present full time on site (and will be substantially involved) and who will each have at least five (5) years of relevant experience. These personnel include as a minimum the micropile superintendent, the micropile project manager, the foreman driller and grouter and the Quality Verification Engineer.

iii. The micropile work shall be carried out in whole by a specialist Contractor having the qualifications stated above.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General and Construction:

OPSS 180	Management of Excess Material
OPSS 903	Piling
OPSS 904	Concrete Structures
OPSS 905	Steel Reinforcement for Concrete
OPSS 906	Structural Steel

Ontario Provincial Standard Specifications, Material:

OPSS 1002	Aggregates – Concrete
OPSS 1301	Cementing Materials
OPSS 1302	Water
OPSS 1303	Admixtures for Concrete

OPSS 1350	Concrete - Materials and Production
OPSS 1440	Steel Reinforcement for Concrete
OPSS 1442	Epoxy Coated Reinforcing Steel Bars for Concrete
OPSS 1840	Non-Pressure Polyethylene (PE) Plastic Pipe Products

Canadian Standards Association Standards, CSA:

A23.1-04/A23.2-09	Concrete Materials and Methods of Concrete Construction/Methods of Test and Standard Practice for Concrete
A283-06	Qualification Code for Concrete Testing Laboratories
G30.18-09	Billet-Steel Bars for Concrete Reinforcement
G40.20-98/G40.21-04	General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steels
W59-03(R2008)	Welded Steel Construction (Metal Arc Welding)

American Society for Testing and Materials Standards, ASTM:

A722/A722M-07	Uncoated High-Strength Bar for Prestressing Concrete
A 252-10	Standard Specification for Welded and Seamless Steel Pipe Piles
C144-11	Standard Specification for Aggregate for Masonry Mortar
D1143M-07	Standard Test for Piles Under Static Axial Compressive Load
D3966-07	Standard Test Methods for Deep Foundations Under Lateral Load
D1784-11	Standard Specification for Rigid Poly (Vinyl Chloride) (PVC) Compounds and Chlorinated Poly (Vinyl Chloride) (CPVC) Compounds
D3689-07	Standard Test Methods for Deep Foundations Under Static Axial Tensile Load
D 4380-84 (2006)	Standard Test Method for Density of Bentonite Slurries

American Welding Society (AWS):

D1.1/D1.1M:2004	Structural Welding Code - Steel
D 1.4:2005	Structural Welding Code – Reinforcing Steel

American Society of Civil Engineers (ASCE):

ASCE 20-96 Standard Guidelines for the Design and Installation of Pile Foundations

International Organization for Standardization/International Electrotechnical Committee, ISO/IEC

DIS 17025:2005 General Requirements for the Competence of the Testing and Calibration Laboratories

Others:

Post Tensioning Institute Publications - Recommendations for Prestressed Rock and Soil Anchors – 2004.
Canadian Foundation Engineering Manual (CFEM), 4th Edition.
Federal Highway Administration Publication No. FHWA NHI–05-039: Micropile Design and Construction Reference Manual, December 2005 (FHWA 2005).

3.0 DEFINITIONS

For the purposes of this Non-Standard Special Provision, the following definitions apply:

Admixture means a substance added to the grout to either control bleed and/or shrinkage, improve flowability, reduce water content, retard setting time, or resist washout.

Alignment Load (AL) means a nominal load applied to a micropile during testing to keep the testing equipment correctly positioned.

Apparent Free Micropile Length means the length of micropile that is not bonded to the surrounding ground, as calculated from the elastic movement data during testing.

Bond Length means the length of the micropile that is bonded to the ground and capable of transferring the applied axial loads to the surrounding soil or rock.

Bond-Breaker means a sleeve placed over the reinforcement steel to prevent load transfer.

Casing means a steel pipe introduced during the drilling process to temporarily stabilize the drill hole and/or permanently reinforce the pile.

Centralizer means a device used to centrally locate the reinforcing element(s) within the casing and/or borehole to ensure that minimum grout cover is provided.

Central Bar or Central Steel means steel reinforcing bars (solid or hollow core) or pipes used to strengthen or stiffen the pile, excluding any left-in drill rod or casing.

Coupler means a device used to transmit load from one partial length of reinforcement to another.

Creep Movement means the movement that occurs during the creep test of a micropile under a constant load.

Design Engineer means the Engineer retained by the Contractor who produces the Working Drawings and designs the pile load test system(s).

Design Checking Engineer means the Engineer retained by the Contractor who checks the Working Drawings and the design of the pile load test system(s).

Design Load (DL) means the anticipated final maximum service load in the micropile. The design load includes appropriate factors to ensure that the overall structure has adequate capacity for its intended use.

Duplex Drilling means a drilling system involving the simultaneous rotation and advancement of (inner) drill rod and (outer) drill casing in which the cuttings from the inner drill rod exit the borehole via the annulus between the rod and the casing.

Elastic Movement means the recoverable movement measured during a micropile load test.

Encapsulation means a corrugated or deformed tube protecting the reinforcing steel against corrosion.

Engineer means a professional engineer, licensed by Professional Engineers, Ontario to practice in the Province of Ontario.

Flush Grout means a suitably thin Portland cement based grout that is injected through the micropile hollow core central reinforcing bar during drilling.

Free (Unbonded) Length means the designed length of the micropile that is not bonded to the surrounding ground or grout during testing.

Low Mobility Grout means a low slump grout with a mix design that typically contains sand or a suitable admixture (thixotropic agent) in order to control its travel from its point of injection.

Micropile means a bored, cast-in-place pile containing steel reinforcement, designed to accept load (axial, bending or lateral) directly, and transfer it to an appropriate bearing stratum.

Maximum Test Load (TL) means the maximum load to which the micropile is subjected during testing.

Overburden means a non-lithified material, natural or placed, which normally requires cased drilling methods to provide an open borehole to underlying strata.

Post-Grouting means the injection of additional grout into the load transfer length of a micropile after the primary grout has set.

Pre-Production Micropile means a sacrificial micropile that is not part of the final foundation system and is subjected to load testing to verify the design and installation procedures.

Primary/Structural Grout means a Portland cement based grout that is injected into the micropile hole prior to, during or after the installation of the reinforcement to provide the load transfer to the surrounding ground along the micropile and affords a degree of corrosion protection when the micropile is in compression.

Production Micropile means a micropile that forms part of the final foundation support system to a structure.

Proof Load Test means the incremental loading of a production micropile, recording the total movement at each increment.

Quality Verification Engineer (QVE) means an engineer who has a minimum of five (5) years experience in the field of design and/or installation of micropiling or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the Contract Documents and to issue Certificate(s) of Conformance.

Reinforcement Steel means the steel component(s) of the micropile which accepts and/or resists applied loadings. This includes the central steel bar and the permanent steel casing on this project.

Residual Movement means the non-elastic (non-recoverable) movement of a micropile measured during load testing.

Rotary Percussive Duplex (Concentric) means a drilling system involving the simultaneous rotation, percussion and advancement of an (inner) drill rod and an (outer) drill casing in which the cuttings from the inner drill rod exit the borehole via the annulus between rod and casing.

Rotary Percussive Duplex (Eccentric or Lost Crown) means a drilling system involving the simultaneous rotation, percussion and advancement of an (inner) drill rod combined with an eccentric underreaming bit and an (outer) drill casing in which the cuttings from the inner drill rod exit the borehole via the annulus between rod and casing. Previously called the Overburden Drilling Eccentric (ODEX) System.

Sheathing means a smooth or corrugated piping or tubing that protects the reinforcing steel against corrosion.

Spacer means a device used to separate elements of a multiple-element steel bar reinforcement.

Tremie Grouting means the placing of grout in a borehole via a grout pipe introduced to the bottom of the hole.

Ultimate Grout-To-Ground Bond Value means the estimated ultimate geotechnical unit grout-to-ground bond strength selected for use in design.

Verification Load Test means a pile load test performed to verify the design of the pile system and the construction methods proposed, prior to installation of production piles.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design Requirements

The Contractor shall be responsible for the design of the pile load testing set-up including the reaction system(s), the reaction piles/ground anchors and all loading frame connections.

The reaction piles/ground anchors and the reaction system(s) shall be designed to safely withstand the applied loads specified in the Contract Documents.

The design assumptions of the reaction piles/ground anchors, the reaction system(s) and all loading frame connections shall accurately represent the subsurface conditions prevalent at the site.

Except as specified herein, the reaction piles/ground anchors shall be designed in accordance with the design recommendations of the Post Tensioning Institute Recommendations for Prestressed Rock and Soil Anchors (2004).

4.02 Submission Requirements

4.02.01 Site Survey

One week prior to commencing any work associated with the micropile operation, the Contractor shall submit to the Contract Administrator, a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, the locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as bridges at or adjacent to the site.

4.02.02 Working Drawings

At least three (3) weeks prior to the commencement of the micropile operations, the Contractor shall submit three copies of the Working Drawings to the Contract Administrator for information purposes only. These Working Drawings shall bear the seal and signature of the Design and Checking Engineers who have a minimum of five years of experience on projects of a similar nature and scope to the required work.

Information to be shown on the Working Drawings shall describe and illustrate the complete details of the micropile installations, as well as the micropile testing equipment, test set-up, and reaction system(s) for the pre-production and production test micropile(s). The information on the Working Drawings shall include the following:

a) Plans, Elevations and Sections (at each of the south and north side pier widenings)

   i. micropile spacing
   ii. orientation
   iii. minimum total micropile length

iv. casing plunge length
v. uncased bond length
vi. design load
vii. a unique identification number for each micropile
viii. micropile components and details

b) Materials

i. physical properties of reinforcement steel (central bar and casing)
ii. physical properties of pile top attachment
iii. bond length grout materials and mix proportions
iv. corrosion protection material physical/mechanical properties

c) Micropile Installation

i. construction methods
ii. work restrictions
iii. schedule of major equipment resources
iv. sequence of pile installation and coordination of work
v. procedures for monitoring micropile installation
vi. type, number and location of pre-production load tests
vii. method of evaluation of load test results

d) Micropile Construction Details

i. Detailed description of the proposed construction procedures.
ii. Method of drilling the micropile holes and maintaining the stability of the holes during the micropile installation.
iii. Method to be employed to penetrate the limestone slab layer and limestone bedrock while minimizing the surface ground movement at the adjacent structure.
iv. Detailed description of the drilling equipment and materials including drill bit/auger diameter and lengths, casing diameter and lengths, flush type, slurry materials or other materials to facilitate the construction of the micropile hole.
v. Method of verifying the lengths of micropile holes.
vi. Detailed description of the grout mixing procedure and the method of grout installation and placement. The description shall include the grout pressures and details of the grout mix design(s) and procedure(s) for limiting the grout takes within the voids in the limestone slab layer.

e) All design assumptions, loads, parameters and bond stresses used for the micropile load tests.

f) Testing records and evaluation when testing has been completed to assess bond stress and micropile movement.

4.02.03 Mill Certificates

The Contractor shall submit to the Contract Administrator at the time of delivery to the job site, one copy of the certified mill test reports, indicating that the steel meets the requirements for the appropriate standards for casing and central bar reinforcement, plates and shapes. The ultimate strength, yield strength, elongation, and material properties composition shall be included. For steel pipe used as permanent casing, or core steel, the Contractor shall submit a minimum of two representative coupon tests or mill certifications on each load delivered to the project.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificate verified by testing by a Canadian laboratory. The laboratory shall be accredited by a Canadian National Accreditation Body to comply with the requirements of ISO/IEC 17025 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date and the signature of an authorized officer of the Canadian testing laboratory.

One copy of the stress-strain curves representative of the lots to be used shall be submitted to the Contract Administrator together with the mill test certificates detailed in OPSS 1440.

4.02.04 Grout

The Contractor shall submit to the Contract Administrator a suitable, site specific grout mix design(s), including details of all materials to be incorporated, and the procedure for mixing and placing the grout. This submittal shall include certified test results verifying the acceptability of the proposed mix design(s). The acceptability of the mixes will be further verified on site prior to production.

4.02.05 Installation Records

The Contractor shall submit micropile installation records, signed by the Quality Verification Engineer, to the Contract Administrator, within 3 business days after each pile installation (including all test piles and production piles) is completed. The installation records shall include the following information:

a) Pile identification number and location;
b) Pile drilling duration, including date of installation and start and finish time;
c) Pile drilling observations, including nature of and variation in cuttings return, penetration rates for each 0.5 m of penetration, presence of boulders or obstructions or voids, connections between holes, top of limestone slab layer, top of bedrock;
d) Information on depth of drilling and soil and rock types encountered, including description of strata, depth to water, etc.;
e) Sequence of installation;
f) Inclination and direction;
g) Final tip elevation;
h) Casing tip elevation;
i) Cut-off elevation;
j) Length and diameters of all components;
k) Bar length, spacers/coupler details;
l) Casing length, joint location details;
m) Description of unusual installation behaviour, conditions, voids, high grout takes;
n) Any deviations from the intended parameters, exceptions and "unusual" events;
o) Grout pressures attained, where applicable;
p) Grout mix proportions;
q) Grout quantities pumped, including depths were larger than normal grout takes occur;
r) Pile materials and dimensions;
s) Micropile test records, analysis and details;
t) As-built drawings showing the location of the piles, their depth and inclination, and details of their composition shall be submitted within thirty (30) calendar days of each pier completion.

4.02.06 Micropile Load Testing

The Contractor shall submit to the Contract Administrator details of the micropile load testing, three (3) weeks prior to construction. The details shall include the following:

a) Detailed description of the proposed load testing procedures.

b) Shop drawings and structural calculations for the design of the pile load testing, including reaction system(s). The structural calculations shall confirm that the materials will meet the specified load and movement criteria.

c) Detailed plans for the set-up method proposed for testing the pre-production and production micropiles including all necessary drawings and details to clearly describe the test method, means for providing reaction, equipment proposed including independent reference beams for measuring micropile head movement. Special attention shall be paid to ensuring safety and providing adequate structural stability of the reaction piles/ground anchors and loading frame connections.

d) Calibration reports for each test jack, pressure gauge, and master pressure gauge to be used. The calibration tests shall have been performed by an independent testing laboratory and tests shall have been performed within one year of the date submitted. Testing shall not commence until the Contract Administrator has approved the jack, pressure gauge and master pressure gauge calculations.

4.02.07 Quality Control

4.02.07.01 Interim Inspections during Installation of Micropiles

The Quality Verification Engineer shall carry out Interim Inspections of the:

a) drilling and casing installation (including cleanliness of casing and depth of penetration of casing into the limestone slab layer); and
b) drilling and grouting with central hollow-bar reinforcement steel (including depth, diameter and length of penetration into bedrock, and grout takes in limestone slab layer).

The above shall be carried out for each individual micropile to verify that the works are constructed in general conformance with the Contract Documents and Working Drawings.

4.02.07.02 Certificate of Conformance

4.02.07.02.01 Pre-Production Micropile Tests

The Contractor shall submit, to the Contract Administrator, a Certificate of Conformance sealed, signed and dated by the Quality Verification Engineer (QVE) upon completion of all of the pre-production micropile testing. The certificate shall state that the pre-production micropiles have been installed and tested in general conformance with the Contract Documents and Working Drawings.

4.02.07.02.02 Production Micropile Tests

The Contractor shall submit, to the Contract Administrator, a Certificate of Conformance sealed, signed and dated by the Quality Verification Engineer (QVE) upon completion of each production micropile test. The certificate shall state that the production micropile has been installed and tested in general conformance with the Contract Documents and Working Drawings.

4.02.07.02.03 Production Micropiles

The Contractor shall submit, to the Contract Administrator, a Certificate of Conformance upon completion of all of the micropile installations. The certificate shall be sealed, signed and dated by the QVE. The certificate shall state that all of the production micropiles have been supplied and installed in general conformance with the Contract Documents and Working Drawings.

4.02.08 As-Built Drawings

As-built drawings shall be submitted to the Contract Administrator in a reproducible format prior to final acceptance of work.

The as-built drawings shall be dated and bear the seal and signature of the Quality Verification Engineer.

5.0 MATERIALS

5.01 Water

Water for mixing grout shall be according to OPSS 1302.

5.02 Admixtures

Admixtures shall be according to OPSS 1303. Admixtures which control bleed, improve or control flowability, reduce water content, and retard set may be used in the grout only if the admixture manufacturer certifies that their use will not affect the required properties of the grout. Expansive admixtures shall only be added to the grout used for filling sealed encapsulations (if used). Accelerators and admixtures with chlorides shall not be permitted. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations.

5.03 Cement

All cement shall be Type GU General Use hydraulic cement conforming to OPSS 1301.

5.04 Fillers

Inert fillers, such as sand, may be used in the grout in special situations (e.g., presence of large voids in the ground) to limit grout take and travel and only if the QVE certifies that their use will not affect the required properties of the grout.

5.05 Grout

The grout mix materials and procedures for placement and testing shall conform to OPSS 1301, OPSS 1302, OPSS 1303, OPSS 1350 and CSA A23.2-1B.

The Contractor shall provide a stable, homogenous neat cement grout or a sand-cement grout. The grout shall be free of any lumps and not contain any evidence of poor or incomplete mixing. The grout shall be mixed to the supplier's specification. The water /cement grout ratio of the thin grout (by weight) shall not exceed 0.90. The water/cement ratio of the grout (by weight) shall not exceed 0.45. The structural grout shall have the following physical properties:

a) A minimum compressive strength of 25 MPa at 7 Days.
b) A minimum compressive strength of 35 MPa at 28 Days.
b) No segregation and a bleed of less than 2 percent when allowed to stand for 1 hour.

5.06 Reinforcement Steel

5.06.01 Central Bar

The central bar reinforcement steel shall be continuously threaded Titan Hollow Bar or approved equivalent, according to OPSS 1440 with minimum yield strength of 570 MPa.

5.06.02 Couplers

Couplers for the central bar reinforcement steel shall be as specified by the supplier of the central bar and shall develop at least 100% of the guaranteed minimum ultimate strength of the central bar.

5.06.03 Casing

The steel casing shall meet the requirements of ASTM A252-10, Grade 3 with minimum yield strength of 552 MPa.

New "Structural Grade" (a.k.a. "Mill Secondary") steel pipe meeting the above but without Mill Certification is acceptable for use as permanent casing provided it is free from defects (dents, cracks, tears) and is accompanied by two coupon tests per truckload confirming it meets the above requirements.

All casing joints shall be threaded or comprised of full penetration field welds. The casing joints shall develop at least the required compressive, tensile and/or bending strength used in the design of the micropile.

5.07 Plates and Shapes

Structural steel plates and shapes for pile top attachments shall be according to OPSS 906 Grade 300W.

5.08 Centralizers and Spacers

Centralizers shall be fabricated from schedule PVC pipe or tube, steel, or material that is non-detrimental to the reinforcement steel. Wood shall not be used.

6.0 EQUIPMENT

6.01 General

All equipment for the installation, testing and monitoring of the pre-production (verification) and production micropiles shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment used shall be capable of installing and grouting the micropiles to the prescribed depths or elevations without damage to the pile materials or to the adjacent structures.

6.02 Grouting Equipment

All grout mixers, pumps and hoses shall be of an adequate capacity and shall be sized to enable the grout to be pumped in one continuous operation, while keeping the grout in constant agitation prior to pumping, and to allow continuous grouting of an individual micropile with the final structural grout within one hour.

A high speed, high shear, colloidal grout mixer with a gauge to measure the quantity of water discharged into the mixer shall be used. A paddle mixer is not acceptable.

The grout pump(s) shall be equipped with a pressure gauge to monitor grout pressures of at least 1 MPa or twice the actual grouting pressures used, whichever is greater.

6.03 Micropile Testing Equipment

The equipment shall be capable of loading the test piles to the maximum specified test load (TL) within the rated capacity.

The equipment shall be capable of loading the pile in increments so that the load on the pile can be increased or decreased in accordance with the test procedures outlined in the Contract Documents.

Dial gauges shall have at least a 75 mm travel and longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel where required. Gauges shall have precision of at least 0.02 mm.

Dial gauges shall allow the measurement of total micropile movement at every load increment to be read to the nearest 0.02 mm increment. The gauge shall have sufficient travel to record the total pile movement at Test Load without the need to reset at an interim point.

Loading equipment shall be calibrated within an accuracy of +/-2% immediately prior to use.

Current calibration curves, dated and bearing the seal and signature of an Engineer shall be provided for all gauges and jacks.

7.0 CONSTRUCTION

7.01 General

The Contractor shall be responsible for the material, fabrication, installation, testing and monitoring of the test micropiles and the production micropiles. In addition, for non-Owner designed reaction piles/ground anchors, the Contractor shall be responsible for design parameters and the design of the reaction piles/ground anchors.

The Contractor's attention is specifically drawn to the following details:

a. The drilling, grouting and micropile installation shall be carried out immediately adjacent to the west pier foundation of the existing Highway 401 bridge (over the CNR line Mile 171.10 of the Kingston Subdivision) which is comprised of spread footings founded on limestone slabs and considered to be sensitive to settlement. The Contractor shall select construction techniques (including the use of hollow core central steel reinforcement bars, with sacrificial drill bits, to allow the simultaneous drilling and injection of grout during installation of the micropiles) that will prevent settlement or heave of the existing structure. The Contractor shall select drilling and grouting methods and be prepared with suitable equipment and procedures to penetrate through the overburden soils, limestone slabs and into the bedrock while minimizing basal heave, soil cave in and surface ground movement at the adjacent structure so as to avoid causing an unacceptable level of distrubance as defined elsewhere in the Contract Documents. The use of rotary percussion drilling with compressed air flush to advance the micropile through the limestone slab layer and into the bedrock will not be allowed.

b. The equipment as well as the drilling, grouting and installation method shall allow for modifying the grout type from the thin grout utilized during the drilling and flushing of the hollow core central bars to a thicker Low Mobility Grout (LMG) to minimize grout takes where voids are encountered, if necessary. Gouting pressures shall be controlled to prevent movement of the adjacent structure during micropile installation.

c. The lateral performance of the micropiles relies, in part, on the short embedment of the outer casing within the limestone slab layer, the elevation of the top of the limestone layer which is anticipated to be variable at the site. In this regard, the Contractor shall ensure that the permanent outer steel casings are embedded 0.3 m beyond the top of the limestone slab layer. However, advancement of the casing significantly beyond the elevation of the top of the limestone slab layer is to be avoided due to the risk of disturbance to the limestone slab layer that supports the adjacent existing bridge foundation.

d. Where mechanical connectors are used for the casing, threaded joints shall not be located above Elevation 74.58 m (e.g., no threaded joints allowed within the portion of the micropile extending down from the top of the pile to within 3 m below the underside of the pile cap). In this regard, the Contractor shall verify that sufficient vertical clearance exists below the underside of the existing bridge girders to satisfy this casing installation requirement prior to the commencement of micropile drilling.

The Contractor shall not proceed with the installation of production micropiles until the satisfactory completion of the pre-production load tests and until approval has been given by the Contract Administrator.

The Contractor shall control all drilling fluids, water and drill cuttings during micropile installation and upon completion of the micropile installations shall clean up, and off-site dispose of all excess fluids and cuttings in accordance with the requirements of OPSS 180.

The Contractor shall comply with all environmental provisions as specified elsewhere in the Contract Documents.

7.02 Subsurface Conditions

A Foundation Investigation Report that describes the subsurface conditions for the project is available, as specified elsewhere in the Contract Documents. The Ministry warrants that the information provided in the Foundation Investigation Report can be relied upon with the following limitations and exceptions:

a) Any interpretation of data or opinions expressed in the reports is not warranted.

b) Although the raw measured data presented is warranted, the Contractor must satisfy itself as to the sufficiency of the information presented for the intended construction purpose and obtain any updating or additional information as required to facilitate the deep foundation works.

The Contractor is alerted that the micropiles will be installed (in part) through the overburden soils, limestone slab layer containing voids or zones of loose soil and into the underlying limestone bedrock. Voids or loose soil zones, interpreted to be up to 0.6 m in height, were encountered in the limestone slab layer during the foundation investigation. The factual information contained in the Foundation Investigation Report (that is available as specified elsewhere in the Contract Documents), provides information on the subsurface conditions at the site.

7.03 Transportation, Handling, Storage

Casings and central bar reinforcement shall be transported, stored and handled in such a manner that damage and distortion is prevented and that the strength and integrity are maintained.

All materials, including cement, additives for grout and pile reinforcement steel (central bar and casing) shall be stored off-ground, under cover and protected against moisture and directly from the elements.

Lifting of any casings and bar reinforcement shall not cause excessive bending.

7.04 Installation of Micropiles

7.04.01 General

The Contractor shall install the micropiles in accordance with the diameter, orientation and length specified in the Contract Documents and as detailed on the Working Drawings.

The micropile installation technique shall be such that it is consistent with the geotechnical, logistical, environmental, and load carrying conditions of the project.

The micropiles will be installed in close proximity to each other and to the existing spread footing foundation at the west pier of the existing adjacent bridge. The Contractor shall carry out the drilling and grouting works in such a manner to prevent any damage to previously installed micropiles, to prevent any loss of ground, and to prevent ground movement at the adjacent bridge structure. In this regard, the sequence of installation of the micropiles, at both the south side and north side widening, shall start furthest from the existing bridge and progress along the row(s) perpendicular to the adjacent rail line.

Movement of the existing bridge structure shall be monitored as specified elsewhere in the Contract Documents. The Contractor's method of installing the micropiles shall be adjusted according to the observation(s) of the response of the adjacent bridge during the course of the work to minimize undue impact to the overall structure.

If the bridge structure monitoring indicates movements of the existing structure are within acceptable limits (as specified elsewhere in the Contract Documents) following installation of the first row(s) of piles at the widenings, the next row of piles closer to the bridge can be installed. Subsequent to this, the next row(s) progressively closer to the existing structure can be installed so long as the bridge structure monitoring indicates total movements remain within acceptable limits.

If, at any point in the micropile installation, the bridge structure monitoring indicates that the Review Levels (Threshold Limits) are being reached, the Contractor will have to re-evaluate and modify the micropile installation and grouting method prior to continuing construction.

The available working space is limited. The Contractor shall inspect the work area to ensure that adequate access and headroom are available for the proposed equipment and procedures for the micropile installation work.

7.04.02 Drilling

The Contractor shall employ drilling equipment and methods suitable for drilling through the anticipated subsurface conditions to be encountered and cause no damage or disruption to these conditions or any overlying or adjacent structure or service. The Contractor shall use steel casing during drilling and installation. Bentonite slurries to stabilize the holes are not permitted.

The upper cased sections of the micropiles shall be drilled using duplex drilling techniques with the cuttings returning up the inside of the casing. The lower uncased sections of the micropiles shall be installed using the hollow core central steel reinforcing bars with sacrificial drill bits that allow the simultaneous drilling and injection of grout.

Drilling shall be conducted in a manner that does not result in significant loss of ground beyond the hole diameter. Disposed cuttings from the upper cased sections of the micropiles shall not exceed 110% of the theoretical borehole volume based on the outside diameter of the casing. The Contractor shall take appropriate measures to prevent interconnection between drill holes during the work.

The Contractor shall determine and schedule all installation techniques such that there will be no interconnection or damage to previously installed micropiles.

7.04.03 Reinforcement Steel

7.04.03.01 General

Pile reinforcement steel (central bar and permanent casing) shall be installed as specified in the Contract Documents and detailed on the stamped Working Drawings.

The Contractor's attention is drawn to the Contract Drawings which show the embedment requirements for the permanent casing into the limestone slab layer as well as for the central bar into the good quality bedrock for each pile.

7.04.03.02 Placement

The Contractor shall be responsible for determining the number of centering devices required. As a minimum, centralizers shall be provided at 3 m centre maximum spacing on the central bar reinforcement. The uppermost centralizer shall be located a maximum of 1.5 m from the top of the micropile. Centralizers shall permit the free flow of grout without misalignment of the reinforcement.

All pile top elevations shall be checked and adjusted to ensure all installed micropiles are installed to the planned elevations.

7.04.03.03 Connections

The pile reinforcement steel connections (splices and joints) shall be constructed to develop the required design strength of the pile section. The central bar reinforcement steel connections (splices) shall be constructed using mechanical connectors only. The casing connections shall be constructed using either mechanical connectors (e.g., threaded joints) or full penetration field welds.

If mechanical connectors are used for the casing, threaded joints shall not be located above Elevation 74.58 m (e.g., no threaded joints allowed within the portion of the micropile extending down from the top of the pile to within 3 m below the underside of the pile cap).

The proposed pile splice/connection details shall be submitted to the Contract Administrator, for information purposes only, prior to use.

Reinforcement steel central bar connections shall not be in the same plane as casing connections/splices.

Secure lengths of casing and reinforcement steel central bar shall be joined in proper alignments and in such a manner that causes no eccentricity between the axes of the two joined lengths or the angle between them.

7.04.04 Grouting

7.04.04.01 General

The grout shall be installed as specified in the Contract Documents and as detailed on the stamped Working Drawings.

The Contractor shall provide systems and equipment to measure the grout quality, quantity and pumping pressure during the grouting operations.

During advancement of the hollow core central reinforcement bar (with sacrificial drill bit), the Contractor shall continuously flush the hole using the thin flush grout. Flushing with water shall not be allowed.

The Contractor's attention is drawn to the expected presence of voids and/or zones of loose soil in the limestone slab layer which may necessitate the use of a thicker Low Mobility Grout (LMG) to minimize grout takes at some locations.

Upon completion of drilling of the uncased section of the micropile to the design tip elevation, the Contractor shall inject the final structural grout from the lowest point of the drill hole until clean, pure structural grout flows from the top of the micropile (to be verified by specific gravity testing with a Baroid mud balance).

Subsequent to completion of grouting, all installation operations associated with completion of the micropile must ensure complete continuity of the grout column. The use of compressed air to directly pressurize the fluid grout is not permissible. The grout pressures and grout take volumes shall be controlled for each stage of each pile to prevent excessive heave or fracturing in the foundation soils, rock formations or adjacent structure. The entire micropile shall be grouted to the design cut-off level.

The grout within the micropiles shall be permitted to attain the minimum design strength prior to being loaded.

Any micropiles not installed according to the specifications shall be replaced, or otherwise remediated appropriately. The cost of replacement and any required foundation modifications are to be carried out at no additional cost to the Owner.

If necessary, the Contractor shall undertake cold weather protection requirements, preparation and protection in accordance with CSA CAN3-A23.1. The temperature of the grout during mixing and pumping shall be maintained between 10 °C and 30 °C.

7.04.04.02 Quality of Grout Mixture

7.04.04.02.01 General

Any grout mixture showing evidence of dampness, lumps, harden pieces, or contamination shall not be incorporated into the work.

The Contractor shall be responsible for testing of bleed, preparation and initial storage of grout cubes for determination of compressive strength, and delivery of the grout cubes to a testing laboratory designated by the Owner.

The Contractor shall employ staff from a testing company certified according to CSA A283 - Certification for Additional Tests 1B, by an organization accredited by the Standards Council of Canada, to carry out testing for bleed, making and curing of grout cubes and early strength determination.

Making of grout cubes for compressive strength test and testing of bleeding, shall be done on a level, vibration free surface.

The Contractor shall perform and record specific gravity testing using a Baroid mud balance following ASTM D4380-84 on the grout utilized for each and every micropile.

7.04.04.02.02 Bleed Requirements

The testing for bleed of the grout shall be according to CSA A23.2-1B.

Prior to the grouting operation, in the presence of the Quality Verification Engineer and the Contract Administrator, a trial batch shall be mixed and the grout tested for bleed, to ensure that the grout meets the requirements specified in the Contract Documents. The trial batch of grout shall not be used in the actual grouting operation unless it meets the requirements for bleed as specified herein.

During the grouting operation, bleeding measurements shall be performed on the grout sampled at the mixer. The measurements shall be performed and recorded at least once a day and as requested by the Contract Administrator.

The bleed test results shall be submitted to the Contract Administrator in writing at the end of each working day. The test results that indicate the grout is not meeting the requirements of the Contract Documents shall be reported immediately to the Contract Administrator and the grouting operation halted until the cause of the problem is identified and corrected.

7.04.04.02.03 Strength Requirements

Grout cubes shall be prepared as follows on site from the grout pumped into the micropile:

a) Three sets of grout cubes, consisting of three cubes each, shall be made each day the grouting operations are carried out.

b) The grout cubes shall be prepared and stored according to CSA A23.2-1B, and shall not be moved prior to demoulding.

c) The grout cubes shall be demoulded and transported to the laboratory within 24 hours ± 4 hours.

d) The grout cubes shall be transported in a sealed white opaque plastic bag containing at least 250 mL of water and maintained at a temperature between 15°C and 25°C.

e) Compressive strength testing shall be carried out on the grout cubes according to CSA A23.2-1B at 3, 7 and 28 days and the test results provided to the Contract Administrator.

f) The Contractor shall prepare and test additional grout cubes to determine when the grout has attained a compressive-strength of 25 MPa.

7.05 Tolerances

The allowable tolerances are as follows:

a) Centreline of the installed micropiles shall not be more than 75 mm from that shown on the Contract Drawings.

b) Micropile-hole alignment shall be within 2% of the total-length plan alignment.

d) Top elevation of micropiles shall be within +25 mm or -50 mm maximum of the vertical design elevation.

e) Centreline of central bar reinforcement shall not be more than 19 mm from centerline of micropile.

7.06 Testing of Micropiles

7.06.01 General

Verification load tests shall be carried out on pre-production test micropile(s) and proof load tests shall be carried out on selected production micropiles. The micropile load testing shall be carried out according to the Working Drawings and as specified herein.

The Contractor shall provide to the Contract Administrator a minimum of three (3) Working Days notice of when the load tests will be carried out. The load tests shall be conducted at a time mutually acceptable to the Contractor and Contract Administrator.

The maximum load in the reaction piles/ground anchors shall not exceed 80% of the guaranteed minimum ultimate tensile strength of the central bar reinforcement or tendon.

The testing shall not be performed until after the grout in the micropiles (or reaction piles/ground anchors) has reached a minimum 7 Days unconfined compressive strength of 25 MPa.

The load tests shall be closely monitored for the duration of the test by the Quality Verification Engineer and the test results recorded and submitted to the Contract Administrator.

7.06.02 Reaction System

The reaction system(s) for the pre-production and production micropile load tests shall be designed by the Contractor and shall be installed as detailed on the Working Drawings.

The Contractor shall determine the number of reaction piles (or ground anchors) required for proper execution of the axial compression load tests. The reaction piles or ground anchors shall located no closer than 2 m to the micropile to be tested under axial compression conditions.

The reaction system for the lateral load tests can be provided by jacking between the two (2) test micropiles.

The Contractor shall make provisions, as appropriate and necessary, to ensure safety and structural stability of the reaction piles, ground anchors and their connection to the load frame and load apparatus.

7.06.03 Reference System, Testing Equipment and Procedures

The layout of the reference systems and testing equipment required for testing shall be as detailed on the Working Drawings and as specified herein.

The Contractor shall supply a suitable means for providing independent reference beams for measuring micropile head movement, jack, electronic load cell, dial gauges or electronic displacement transducers, anchor extension, and any other hardware necessary to carry out the load tests. A minimum of 3 dial gauges or electronic displacement transducers and piano wire, is required. Dial gauges or displacement transducers shall have an accuracy of ±0.0254 mm (0.001 in). Load cells shall have an accuracy of ±2% of the maximum intelcore2duo1107!test load. The calibration curve between the jack pressure and the load shall be submitted to the Contract Administrator for information purposes.

All reference beams shall be independently supported with the support firmly embedded in the ground at a distance of not less than 2.5 m from the reaction system. Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

All gauges, scales and reference points attached to the micropile (or reaction piles/ground anchors) shall be mounted so as to prevent movement relative to the micropile (or reaction piles/ground anchors) during the test.

The jacks shall be secured (with chains or other protective housing(s)) to provide adequate protection to personnel in the event of breakage of the micropile, ground anchor or loading system.

The Contractor shall perform the micropile load tests according to ASTM D-1143, ASTM D-3689 and ASTM D-3966, superseded where applicable by the procedures specified in this Special Provision, and indicate the minimum following information:

a) Type and accuracy of apparatus for measuring load.

b) Type and accuracy of apparatus for applying load.

c) Type and accuracy of apparatus for measuring the micropile displacement.

d) Type and capacity of reaction load system, including sealed Working Drawings.

e) Hydraulic jack calibration report.

7.06.04 Pre-Production Micropile Load Tests

7.06.04.01 General

The Contractor shall perform a pre-production load test (axial compression and lateral) on sacrificial micropiles to verify the design assumptions and the appropriateness of the proposed construction procedures, prior to installation of production micropiles. In the pre-production axial compression micropile test, the micropile shall be subjected to an axial compressive load equal to 2.5 times the factored axial geotechnical resistance at Ultimate Limit States (ULS); but not necessarily to failure. In the pre-production lateral micropile test, two (2) micropiles shall be subjected to a lateral load equal to 2.5 times the maximum lateral design load; but not necessarily to failure.

For the purposes of the pre-production axial compression load test, ground anchors or micropiles will be used to provide the tensile reaction. For the purposes of the pre-production lateral load test, the two micropiles will be loaded by jacking between the two test micropiles.

The pre-production lateral load tests shall be completed prior to carrying out the axial compression load test. After completion of the lateral load tests and the axial compression load test on the sacrificial micropiles, both of the ground anchors (reaction piles) shall be tested in tension to failure (if possible) so that the ultimate grout-to-ground bond can be assessed for design verification purposes.

The pre-production sacrificial micropile load tests with dead weight, or reaction piles/ground anchors shall be designed, constructed and tested by the Contractor and, based on the load test results, the design verified by the Contract Administrator prior to approval being given to the Contractor to start installation of the production micropiles. Approval to start production micropiles shall be given no later than 3 Working Days after completion of all of the pre-production load tests.

7.06.04.02 Installation of Pre-Production Sacrificial Test Micropiles

The Contractor shall install a four (4) micropile pre-production load test section (consisting of three (3) micropiles and one (1) reaction micropile/ground anchor). One of the micropiles subjected to the lateral load test may be subsequently used as a reaction pile for the axial load compression test, provided no damage is caused during its lateral load test.

The pre-production test piles and reaction anchor(s) will be installed near, but a minimum of 2 m away from, the proposed micropile group cap located on the north side of the existing bridge at a location selected by the Contract Administrator. The Contractor shall install and test two (2) micropiles under lateral loading, one (1) micropile under axial compressive loading and two (2) reaction piles under axial tensile loading at the test section location.

The Contractor shall employ the drilling and grouting methods, casing and other reinforcement details, and depth of embedment for the test micropiles identical to those to install the vertical production micropiles, except where specified otherwise by the Contract Administrator. In this regard, the ground surface in the area of the pile load tests shall be excavated to Elevation 77.58 m, approximately the same elevation as the underside of the pile cap for the production micropiles. The details of the test pile set-up and excavation area shall be submitted to the Contract Administrator for information purposes only.

The Quality Verification Engineer shall be responsible for logging the holes for the pre-production micropiles to be tested and for the associated reaction piles (ground anchors). The subsurface conditions in terms of stratigraphy at the test locations are required for proper interpretations of the load test results. The Contractor shall also make provisions, as appropriate, to facilitate the Contract Administrator in carrying out their own logging of the holes for the pre-production sacrificial micropiles and the associated reaction piles.

Upon completion of the sacrificial, pre-production micropile load tests, and prior to demobilization from the site, the test section area is to be restored to near original conditions as per the direction of the Contract Administrator.

7.06.04.03 Test Procedures and Measurement

The Contractor shall load the tested micropile(s) to a minimum of 250 % of the design load (DL) (i.e., 2.5 DL). The jack shall be positioned at the beginning of the tests such that unloading and repositioning of the jack during the test will not be required. The Contractor shall apply an Alignment Load (AL) to the piles prior to setting the movement recording devices. This Alignment Load shall be no more than 10 % of the Design Load (i.e., 0.1 DL); dial gauges shall be zeroed at the first setting of the AL.

The Contractor shall carry out the lateral load test by loading the micropiles and recording the micropile head movement, as well as the deflection at the ground line, according to the load increments presented in Table 1. The maximum lateral test load (TL) shall be 175 kN. The Contractor shall maintain each load increment for a minimum duration as indicated in Table 1.

Table 1
Lateral Micropile Load Test Increments

Load	Minimum Hold Time (Minutes)
AL	-
0.25 DL	10
0.50 DL	10
AL	5
0.25 DL	5
0.50 DL	5
0.75 DL	10
1.00 DL	10
AL	5
0.25 DL	5
0.50 DL	5
0.75 DL	5
1.00 DL	5
1.25 DL	10
1.50 DL	10
AL	5
0.50 DL	5
1.00 DL	5
1.50 DL	10
2.00 DL	10
1.50 DL	5
1.00 DL	5
0.50 DL	5
AL	5
0.50 DL	5
1.00 DL	5
1.50 DL	5
2.00 DL	5
2.25 DL	10
2.50 DL	20
2.00 DL	10
1.50 DL	10
1.00 DL	10
0.50 DL	10
AL	5

AL = Alignment Load;
DL= Design Load
Lateral DL = 70 kN
TL = maximum test load = 175 kN (2.5 DL) unless failure occurs at a lower load increment

The Contractor shall carry out the axial load test by loading the micropile and recording the micropile head movement according to the load increments presented in Table 2. The maximum axial test load (TL) shall be 1,975 kN. The Contractor shall maintain each load increment minimum duration indicated in Table 2 or until the settlement rate is less than 1 mm/log cycle of time.

Table 2
Axial Micropile Compression Load Test Increments

Load	Minimum Hold Time (Minutes)
AL	-
0.15 DL	2.5
0.30 DL	2.5
0.50 DL	10
AL	10
0.15 DL	1
0.45 DL	1
0.60 DL	2.5
0.70 DL	2.5
0.80 DL	10
0.90 DL	10
1.00 DL	30
AL	10
0.15 DL	1
1.00 DL	1
1.15 DL	2.5
1.30 DL	2.5
1.50 DL	10
AL	10
0.25 DL	2.5
0.50 DL	2.5
0.75 DL	2.5
1.00 DL	10
1.25 DL	2.5
1.50 DL	2.5
1.75 DL	2.5
2.00 DL	60
1.50 DL	5
1.00 DL	5
0.50 DL	5
AL	10
0.25 DL	2.5
0.50 DL	2.5
0.75 DL	2.5
1.00 DL	10
1.25 DL	2.5
1.50 DL	2.5
1.75 DL	2.5
2.00 DL	10
2.25 DL	10
2.50 DL	60
2.00 DL	5
1.50 DL	5
1.00 DL	5
0.50 DL	5
AL	10

AL = Alignment Load;
DL= Design Load
Axial DL = 790 kN
TL = maximum test load = 1,975 kN (2.5 DL) unless failure occurs at a lower load increment

The Contractor shall carry out an axial tension load test on each of the reaction piles (ground anchors) following the completion of the pre-production micropile lateral load tests and axial compression load test. The tension load tests shall be carried out in a manner to induce uplift failure of the reaction piles (ground anchor) to enable assessment of the ultimate value of the grout-to-ground bond.

The Contractor shall identify and record the geometry of the grouted section and the rock in which the grouted section exists. This information will provide additional confirmation of the design grout-to-ground bond value for the production micropiles. Construction and installation procedures used for the grouted section of the tested reaction piles (ground anchors) shall be identical to those used for the grout-to-ground section of the pre-production and production micropiles. The load increments and hold times for the uplift test shall be as shown in Table 3.

Table 3
Axial Reaction Pile (Ground Anchor) Tension Load Test Increments

Load	Minimum Hold Time (Minutes)
AL	-
0.15 TL	2.5
0.25 TL	2.5
AL	1
0.15 TL	2.5
0.30 TL	2.5
0.50 TL	2.5
AL	1
0.15 TL	2.5
0.45 TL	2.5
0.60 TL	2.5
0.75 TL	2.5
AL	1
0.15 TL	2.5
0.45 TL	2.5
0.75 TL	2.5
0.90 TL	5.0
1.00 TL	5.0
0.75 TL	5.0
0.50 TL	5.0
0.25 TL	5.0
AL	5.0

AL=Alignment Load
TL=Maximum Test Load estimated to cause failure = 1,975 kN

The movement of the test pile (or reaction pile/ground anchor) shall be measured at each load increment. The load hold period shall be started as soon as the test load is applied. The pile movement shall be measured and recorded, with respect to a fixed reference, at 1, 2, 3, 4, 5 and 10 minutes, and at 10 minute increments thereafter (if applicable). For durations longer than 60 minutes, readings shall be taken at 30 minute intervals (if applicable).

7.06.04.04 Design Acceptance Criteria for Pre-Production Load Tests

The acceptance criteria for micropile pre-production load tests are:

a) Sustaining the lateral design load (1.0 DL) with no more than 15 mm total lateral movement at the ground line, as measured relative to the vertical axis of the pile prior to the start of testing. If an Alighnmnet Load is used, then the allowable movement will be reduced by multiplying by a factor of (DL-AL)/DL.

b) Sustaining the axial compression design load (1.0 DL) with no more than 4 mm total vertical movement at the top of the pile, as measured relative to the top of the pile prior to the start of testing. If an Alignment Load is used, then the allowable movement will be reduced by multiplying by a factor of (DL-AL)/DL.

c) Creep rate at the end of the 2.5 DL increment on Axial Test micropiles not greater than 1 mm/log cycle time from 1 to 10 minutes or 2 mm/log cycle time from 6 to 60 minutes and having a linear or decreasing creep rate.

d) Failure does not occur at the 2.5 DL axial load where failure is defined as the slope of the applied load versus deflection (at end of load increment) curve exceeding 0.15 mm/kN.

e) Overall micropile alignment of all test micropiles within 2% of vertical. This is required given the close spacing of the micropiles to avoid interaction between or intersection of micropiles at depth.

f) The above is under the assumption that the test piles have been installed in accordance with the specifications and drawings to a proper standard of care.

7.06.05 Production Micropile Load Tests and Target Criteria

7.06.05.01 General

The Contractor shall carry out proof load tests (under axial compression conditions) on a minimum of one (1) vertical production micropile at each of the two (2) pier widenings. The Contractor shall submit to the Contract Administrator a proposal recommending the production micropile(s) to be selected for testing, however, the final selection will be up to Contract Administrator.

The selected micropiles for proof testing will be tested no sooner than 7 Days after installation to allow the grout to reach sufficient strength. It is noted that the use of adjacent vertical micropiles as reaction anchors is acceptable, provided that the minimum spacing requirements between the reaction piles and the test pile (as defined in Section 7.06.02) can be satisfied. The reaction system, including any reaction anchors, if necessary, is to be entirely designed by the Contractor and submitted to the Contract Administrator for information purposes.

The Contractor shall set-up the reaction frame and load test set-up, complete with jacks, load cells and gauges in a manner similar to the pre-production load tests. The Contractor shall repair any previously installed production micropiles that may have been damaged during the course of proof load testing to the approval of the Contract Administrator and at no cost to the Owner.

The Contractor shall carry out proof load testing in axial compression and loading in increments to 1.6 DL as shown in Table 4.

Table 4
Axial Production Micropile Load Test Increments

Load	Minimum Hold Time (Minutes)
AL	-
0.15 DL	2.5
0.30 DL	2.5
0.45 DL	2.5
0.60 DL	2.5
0.75 DL	2.5
0.85 DL	10*
0.95 DL	10*
1.00 DL	10*
1.30 DL	10*
1.60 DL	10*
1.00 DL	4
0.75 DL	4
0.50 DL	4
0.25 DL	4
AL	4

AL = Alignment Load
DL = Design Load = 790 kN
* Hold until acceptance criterion for creep movement is satisfied as specified in Section 7.06.05.02 of this specification.

7.06.05.02 Target Criteria for Production Load Tests

The target criteria for proof load tests shall be as follows:

a) Total vertical movement at the top of the micropile shall not be greater than 4 mm (at 1.0 DL), as measured relative to the top of the micropile prior to the start of testing.

b) Creep rate at the end of the 1.00 DL and 1.60 DL load increments shall not be greater than 1 mm/log cycle of time.

c). Failure does not occur where failure is defined as the slope of the applied load versus deflection (at the end of load increments 1.00 DL and 1.60 DL) curve exceeding 0.15 mm/kN.

d) Overall micropile alignment of all production micropiles is within 2 % of vertical.

7.07 Management of Excess Material

Management of excess material shall be according to OPSS 180.

7.08 As-Built Drawings

As-built drawings shall be prepared by the Contractor for Owner designed installations as follows:

a) For all work incorporated in the completed structure that required the submission of Working Drawings.

b) For all changes from the original Contract requirements.

The as-built drawings shall be submitted to the Contract Administrator in a reproducible format prior to final acceptance of the work.

The as-built drawings shall be dated and bear the seal and signature of the Quality Verification Engineer.

8.0 MEASUREMENT FOR PAYMENT

8.1 Pre-Production Micropile Testing

A count will be made of the number of pre-production micropiles that are load tested and satisfy the Contract requirements.

8.2 Production Micropile

Measurement will be made in metres of the micropiling left in place after cut-off.

8.3 Grouting

Measurement will be made in cubic metres of grout pumped into the subsurface during the installation of the micropiles.

8.4 Production Micropile Testing

A count will be made of the number of production micropiles that are load tested and satisfy the Contract requirements.

9.0 BASIS FOR PAYMENT

9.1 Supply Equipment for Installing Micropiles - Item

Payment at the Contract price for the above items shall be full compensation for all labour, equipment and material required to do the work.

It will be assumed, for payment purposes, that 50% of the work under this item has been completed when the satisfactory performance of the equipment has been demonstrated to the Contract Administrator by the installation of one (1) micropile. The remaining 50% will be paid on the satisfactory completion of the installation.

9.2 Grout for Micropiles – Item

Payment at the Contract price for the above items shall be full compensation for all Labour, Equipment and Material required to do the work.

9.3 Pre-Production Micropile - Item

Payment at the Contract price for the above items shall be full compensation for all labour, load testing, equipment and material required to do the work.

No payment will be made for micropiles that fail the load test.

For pre-production micropiles that fail the load test, payment for any additional work directed by the Contract Administrator shall be made as Extra Work.

No payment will be made for additional work for pre-production micropiles that fail to meet the Design Acceptance Criteria for Pre-Production Load Tests.

9.4 Production Micropile - Item

Payment at the Contract price for the above items shall be full compensation for all labour, equipment and material required to do the work.

9.5 Production Micropile Testing - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, equipment and materials required to do the work.

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