



**THURBER** ENGINEERING LTD.

**PRELIMINARY FOUNDATION DESIGN  
HIGHWAY 417 – 15 BRIDGES  
CITY OF OTTAWA, ONTARIO  
GWP No. 4074–11–00  
VOLUME 1 OF 2**

**GEOCRES No. 31G5-263**

**SUBMITTED TO  
McINTOSH PERRY CONSULTING ENGINEERS LTD.**

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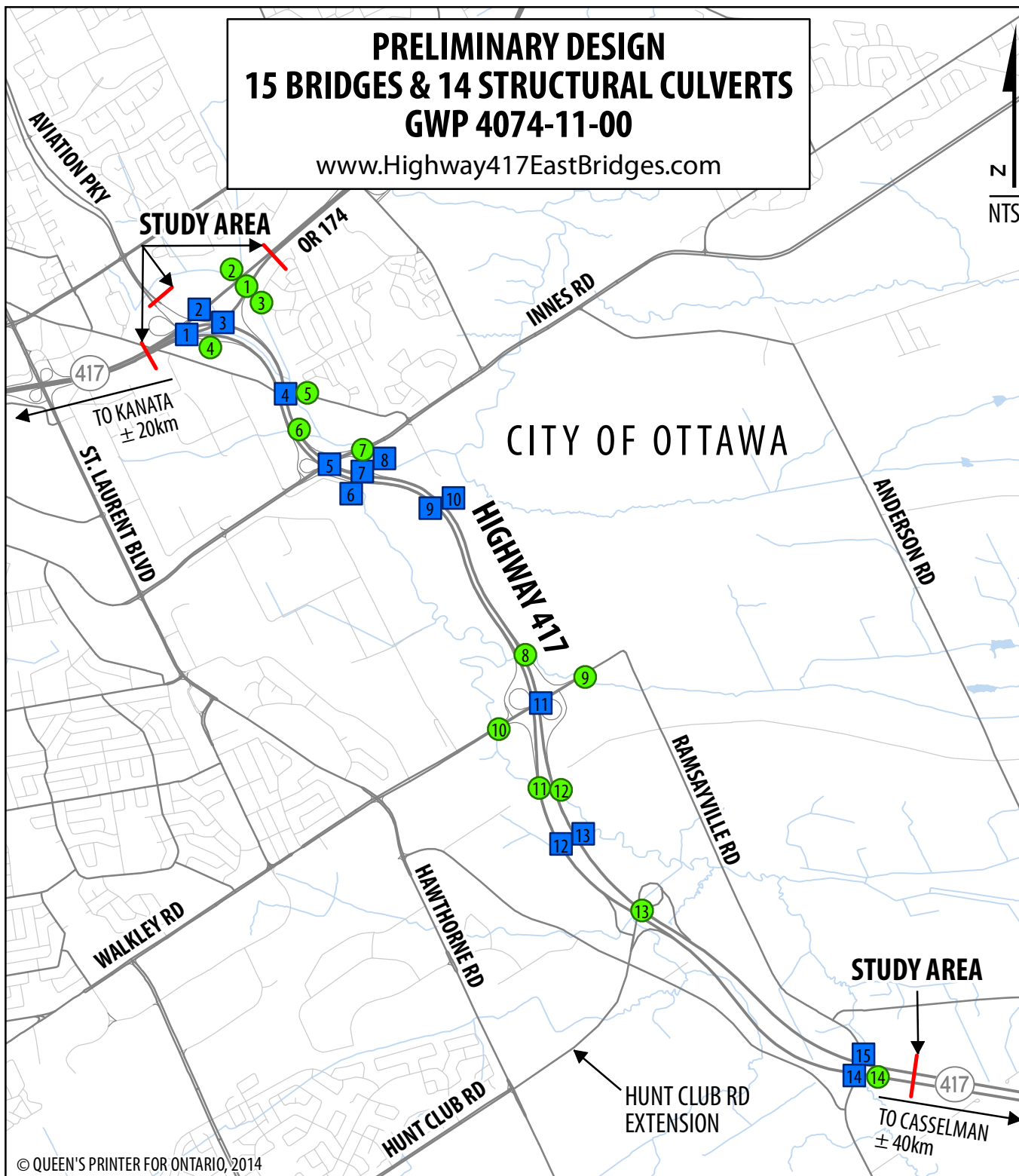
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# PRELIMINARY DESIGN 15 BRIDGES & 14 STRUCTURAL CULVERTS GWP 4074-11-00

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## LEGEND:



### BRIDGE STUDY SITES

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UNNAMED
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UNNAMED
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CYRVILLE COLLECTOR DRAIN
6. SITE 3-443/C  
SOUTH CYRVILLE DRAIN  
\* includes detail design
7. SITE 3-762/C  
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GREEN'S CREEK
13. SITE 3-315/C  
MCEWEN CREEK  
\* includes detail design
14. SITE 3-444/C  
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Report Summary Table GWP 4074-11-00 GEOCRES No. 31G5-263

Site No.	Site	Location	Structure Description	Available Site Information	Subsurface	Soil Properties	OHSa Soil Type	Seismic Profile & Liquefaction Susceptible	Existing Bridge and Embankments									
									Bridge			Embankment						
									Abutments			Piers			Height m	Geometry	Stability Issues	Settlement Issues
Foundation	Vertical Geotechnical Resistance at ULS	Vertical Geotechnical Reaction at SLS	Foundation	Vertical Geotechnical Resistance at ULS	Vertical Geotechnical Reaction at SLS													
1	3-303	Aviation Parkway At the Hwy 417/ Hwy 174 Interchange	Based on historic General Plan Drawings: 13.4 to 14.9 m wide, 229.9 m long, 6 span structure	GEOCRES Report No. 31G5-86	0.9 to 2.8 m compact to very dense silty sand 0.9 to 1.1 m stiff clayey silt 1.1 to 1.6 dense to very dense glacial till 2.7 to 4.4 m loose to very dense sand & gravel Billings formation shale bedrock Based on GEOCRES Report No. 31G5-86 Boreholes 9 to 16	Sandy Silt & Silty Sand $\gamma = 19 \text{ kN/m}^3$ $\phi = 30^\circ$ Clayey Silt $\gamma = 19 \text{ kN/m}^3$ $\phi = 29^\circ$ Sand & Gravel $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$ Glacial Till $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$	Clayey Silt Type 3 Sandy Silt, Silty Sand & Sand & Gravel, Glacial Till Type 2 above GW Type 4 below GW	Soil Profile Type I Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN / 12BP74 pile allowable As per GEOCRES Report	NA	Spread footings on Shale Bedrock	≈960 kPa Pier 1 & 2 - 4.3 m <sup>2</sup> , Pier 3 - 3.4 x 4.9 m Pier 4 & 5 - 4.6 m <sup>2</sup>	NA	9.6	2H:1V	NA	NA
2	3-304/2	Hwy 417 At the Hwy 417/ Hwy 174 Interchange	Based on historic General Plan Drawings: 10.1 m wide, 89.9 m long, 3 span structure	GEOCRES Report No. 31G5-86	1.4 m fill 1.4 m stiff clayey silt 2.7 to 4.1 m loose to dense silty sand 0.1 to 1.6 m compact to very dense till Billings formation shale bedrock Based on GEOCRES Report No. 31G5-86 Boreholes 6 to 8	Clayey Silt $\gamma = 19 \text{ kN/m}^3$ $\phi = 29^\circ$ Silty Sand $\gamma = 19 \text{ kN/m}^3$ $\phi = 30^\circ$ Till $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$	Clayey Silt Type 3 Sandy Silt, Silty Sand & Till Type 2 above GW Type 4 below GW	Soil Profile Type I Site is not susceptible to liquefaction	Shale Bedrock	HP12x53 driven to bedrock ≈845 kN / 12BP53 pile allowable As per GEOCRES Report	NA	Spread footings on Till	≈480 kPa Pier 1 & 2 - 4.9 m <sup>2</sup>	NA	8	2H:1V	NA	NA
3	3-304/1	Hwy 417 At the Hwy 417/ Hwy 174 Interchange	Based on historic General Plan Drawings: Y-Shaped North structure 81.4 m long 3 span structure South structure 99.4 m long 3 span structure	GEOCRES Report No. 31G5-86	2.7 to 4.8 m compact to dense sand & gravel 1.3 m very dense till Billings formation shale bedrock Based on GEOCRES Report No. 31G5-86 Boreholes 1 to 5	Sand & Gravel $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$ Till $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$	Sand & Gravel & Till Type 2 above GW Type 4 below GW	Soil Profile Type I Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 (abutments) HP12x53 (retaining walls) driven to bedrock ≈845 kN / 12BP74 pile allowable As per GEOCRES Report	NA	Spread footings on Shale Bedrock	≈960 kPa Pier 1 & 2 - 4.3 m2, Pier 3 & 4 - 4.6 m2	NA	9.6	2H:1V	NA	Erosion of slope paving and embankments was observed
4	3-314	Cyrville Road 600 m north of Hwy 417 / Innes Road Interchange	Based on historic General Layout Drawings: 12.8 m wide, 102.4 m long, 2 span structure	GEOCRES Report No. 31G5-114	1.8 to 2.4 m compact to dense silty sand / sandy silt 0.8 to 1.1 m compact to very dense till Billings formation shale bedrock Based on GEOCRES Report No. 31G5-114 Boreholes 1 to 6	Sandy Silt & Silty Sand $\gamma = 19 \text{ kN/m}^3$ $\phi = 30^\circ$ Till $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$	Sandy Silt, Silty Sand & Till Type 2 above GW Type 4 below GW	Soil Profile Type I. Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN / 12BP74 pile allowable As per GEOCRES Report	NA	Spread footings on Shale Bedrock	≈960 kPa Pier 1 - 5.3 m <sup>2</sup>	NA	6	2H:1V	NA	NA
5	3-305	Innes Road 350 m east o f the intersection of Star Top Rd and Innes Rd	Based on historic General Plan Drawings: 22 m wide, 85 m long, single span structure	GEOCRES Report No. 31G5-81	3.4 to 5.1 m loose to very dense silty sand Billings formation shale bedrock Based on GEOCRES Report No. 31G5-81 Boreholes 1 to 5	Sandy Silt & Silty Sand $\gamma = 19 \text{ kN/m}^3$ $\phi = 30^\circ$	Sandy Silt, Silty Sand Type 2 above GW Type 4 below GW	Soil Profile Type I. Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈890 kN pile allowable As per GEOCRES Report	NA	Shale Bedrock	≈1400 kPa Pier 1 - 3.4 m <sup>2</sup>	NA	7.3	2H:1V	NA	NA
6	3-311/1	Hwy 417 EB over Green Creek 270 m south-east of Hwy 417 / Innes Rd Interchange	Based on historic General Plan Drawings: 30.4 to 31.9 m wide, 24.4 m long, single span structure	GEOCRES Report No. 31G5-85	3.1 m loose sandy silt 7.0 m loose to compact silty sand Billings formation shale bedrock Based on GEOCRES Report No. 31G5-85 Boreholes 8 and 9	Sandy Silt & Silty Sand $\gamma = 19 \text{ kN/m}^3$ $\phi = 30^\circ$	Sandy Silt, Silty Sand Type 2 above GW Type 4 below GW	Soil Profile Type I. Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN pile allowable As per GEOCRES Report	NA	NA	NA	NA	6.8	2H:1V	NA	NA
7	3-311/2	Hwy 417 WB over Green Creek 230 m south-east of Hwy 417 / Innes Rd Interchange	Based on historic General Plan Drawings: 17.4 m wide, 24.4 m long, single span structure	GEOCRES Report No. 31G5-85	2.9 m soft to firm silty clay 1.2 to 3.5 m loose to dense silty sand Billings formation shale bedrock Based on GEOCRES Report No. 31G5-85 Boreholes 7 and 10	Silty Clay $\gamma = 17 \text{ kN/m}^3$ $\phi = 27^\circ$ Silty Sand $\gamma = 19 \text{ kN/m}^3$ $\phi = 30^\circ$	Silty Clay Type 4 Silty Sand Type 2 above GW Type 4 below GW	Soil Profile Type I. Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN pile allowable As per GEOCRES Report	NA	NA	NA	NA	6.4	2H:1V	NA	NA
8	3-310	Hwy 417 S-EW Ramp over Green Creek 130 m south-east of Hwy 417 / Innes Rd Interchange	Based on historic General Plan Drawings: 11.6 to 13.5 m wide, 27 m long, single span structure	GEOCRES Report No. 31G5-85	3.7 to 5.3 m loose to very dense silty sand Billings formation shale bedrock Based on GEOCRES Report No. 31G5-85 Boreholes 2, 11 and 13	Silty Sand $\gamma = 19 \text{ kN/m}^3$ $\phi = 30^\circ$	Silty Sand Type 2 above GW Type 4 below GW	Soil Profile Type I. Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN pile allowable As per GEOCRES Report	NA	NA	NA	NA	8	2H:1V	NA	NA



Report Summary Table GWP 4074-11-00 GEOCRES No. 31G5-263

Site No.	Site	Recommendations											Construction Issues	Additional Work
		Abutment			Preferred Foundation			Unfactored Coefficient of Friction Concrete Cast-in-Place / Precast	Special Concerns	Embankment	Stability Issues	Settlement Issues		
		Foundation	Vertical Geotechnical Resistance at ULS	Vertical Geotechnical Reaction at SLS	Founding Level / Foundation Type	Vertical Geotechnical Resistance at ULS	Vertical Geotechnical Reaction at SLS							
1	3-303	HP310x110 piles driven to bedrock	2000 kN/pile	NA	Spread footings on sound bedrock	1500	NA	0.50 / 0.45	Billings formation shale is susceptible to heaving if exposed during construction	2H:1V	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified. Investigation should include monitoring wells for developing dewatering plans and bedrock sampling and chemical testing to determine pyritic heave potential. The need for vibration monitoring should also be assessed.
2	3-304/2	HP310x79 piles driven to bedrock	1450 kN / pile	NA	Spread footings min 3 m wide founded on till	600	400	0.45 / 0.40	Billings formation shale is susceptible to heaving if exposed during construction	2H:1V	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified. Investigation should include monitoring wells for developing dewatering plans and bedrock sampling and chemical testing to determine pyritic heave potential. The need for vibration monitoring should also be assessed.
		HP310x110 piles driven to bedrock	2000 kN / pile	NA	Spread footings founded on sound bedrock	1500	NA	0.50 / 0.45						
3	3-304/1	HP310x79 piles driven to bedrock	1450 kN / pile	NA	Spread footings founded on sound bedrock	1500	NA	0.50 / 0.45	Billings formation shale is susceptible to heaving if exposed during construction	2H:1V with 3 m mid height bench	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified or if excavations are planned within the fills. Investigation should include monitoring wells for developing dewatering plans, bedrock coring, sampling and compressive strength testing to determine the condition and strength properties of the bedrock, chemical testing to determine pyritic heave potential of the bedrock. The need for vibration monitoring should also be assessed.
		HP310x110 piles driven to bedrock	2000 kN / pile	NA										
4	3-314	HP310x110 piles driven to bedrock	2000 kN / pile	NA	Spread footings founded on sound bedrock	1500	NA	0.50 / 0.45	Billings formation shale is susceptible to heaving if exposed during construction	2H:1V	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified or if excavations are planned within the fills. Investigation should include monitoring wells for developing dewatering plans, bedrock sampling and chemical testing to determine pyritic heave potential. The need for vibration monitoring should also be assessed.
5	3-305	HP310x110 piles driven to bedrock	2000 kN / pile	NA	Spread footings founded on sound bedrock	1500	NA	0.50 / 0.45	Billings formation shale is susceptible to heaving if exposed during construction	2H:1V	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified or if excavations are planned within the fills. Investigation should include monitoring wells for developing dewatering plans, bedrock sampling and chemical testing to determine pyritic heave potential. The need for vibration monitoring should also be assessed.
6	3-311/1	HP310x110 piles driven to bedrock	2000 kN / pile	NA	Spread footings founded on sound bedrock	1500	NA	0.50 / 0.45	Billings formation shale is susceptible to heaving if exposed during construction	2H:1V	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified or if excavations are planned within the fills. Investigation should include monitoring wells for developing dewatering plans, bedrock sampling and chemical testing to determine pyritic heave potential, and the need for vibration monitoring should be assessed. The potential for and the implications of liquefaction must be confirmed.
7	3-311/2	HP310x110 piles driven to bedrock	2000 kN / pile	NA	Spread footings founded on sound bedrock	1500	NA	0.50 / 0.45	Billings formation shale is susceptible to heaving if exposed during construction	2H:1V	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified or if excavations are planned within the fills. Investigation should include monitoring wells for developing dewatering plans, bedrock sampling and chemical testing to determine pyritic heave potential, and the need for vibration monitoring should be assessed. The potential for and the implications of liquefaction must be confirmed.
8	3-310	HP310x110 piles driven to bedrock	2000 kN / pile	NA	Spread footings founded on sound bedrock	1500	NA	0.50 / 0.45	Billings formation shale is susceptible to heaving if exposed during construction	2H:1V	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if geometry of the approach fills are modified. Investigation should include monitoring wells for developing dewatering plans and bedrock sampling and chemical testing to determine pyritic heave potential. The need for vibration monitoring should also be accessed.

Report Summary Table GWP 4074-11-00 GEOCRES No. 31G5-263

Site No.	Site	Location	Structure Description	Available Site Information	Subsurface	Soil Properties	OHSA Soil Type	Seismic Profile & Liquefaction Susceptible	Existing Bridge and Embankments									
									Bridge						Embankment			
									Abutments			Piers			Height m	Geometry	Stability Issues	Settlement Issues
									Foundation	Vertical Geotechnical Resistance at ULS	Vertical Geotechnical Reaction at SLS	Foundation	Vertical Geotechnical Resistance at ULS	Vertical Geotechnical Reaction at SLS				
9	3-302/1	Hwy 417 EB overpass of CP Rail line 800 m south of Innes Rd	Based on historic General Plan Drawings: 20.5 m wide, 79.1 m long, 4 span structure	GEOCRES Report No. 31G5-80	10.1 to 11.9 m loose to very dense silty sand Billings formation shale bedrock Based on GEOCRES Report No. 31G5-80	Silty Sand $\gamma = 19 \text{ kN/m}^3$ $\phi = 30^\circ$	Silty Sand Type 2 above GW Type 4 below GW	Soil Profile Type I. Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN pile allowable As per GEOCRES Report	NA	42" Ø caisson with 36" Ø socket 1' into sound shale bedrock	3150 kN / caisson	NA	11.3	2H:1V	NA	NA
10	3-302/2	Hwy 417 WB overpass of CP Rail line 800 m south of Innes Rd	Based on historic General Plan Drawings: 17.4 m wide, 64 m long, 3 span structure	GEOCRES Report No. 31G5-80	9.7 to 10.1 m very loose to very dense silty sand Billings formation shale bedrock Based on GEOCRES Report No. 31G5-80	Silty Sand $\gamma = 19 \text{ kN/m}^3$ $\phi = 30^\circ$	Silty Sand Type 2 above GW Type 4 below GW	Soil Profile Type I. Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN pile allowable As per GEOCRES Report	NA	42" Ø caisson with 36" Ø socket 1' into sound shale bedrock	3150 kN / caisson	NA	11.3	2H:1V to 4H:1V	NA	NA
11	3-306	Hwy 417 / Walkley Rd Interchange 600 m east of Sheffield Rd and Walkley Rd intersection	Based on historic General Layout Drawing: 31 m wide, 83 m long, 2 span structure	GEOCRES Report No. 31G5-113	3.5 to 6.4 m very stiff to firm silty clay 2.8 to 5.2 m loose to very dense till Carlsbad formation shale bedrock Based on GEOCRES Report No. 31G5-113 Boreholes 1, 2, 4, 5, & 7	Silty Clay $\gamma = 17 \text{ kN/m}^3$ $\phi = 27^\circ$ Till $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$	Silty Clay Type 3	Soil Profile Type I. Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN / 12BP74 pile allowable reduced to 700 kN / pile for negative skin friction As per GEOCRES Report	NA	Shale Bedrock	HP12x89 driven to bedrock No capacities provided	NA	7	2H:1V	NA	Predicted settlement of 125 mm with 50% occurring within 12 months
12	3-301/1	Hwy 417 EB overpass of CN Rail line 850 m south of Walkley Rd	Based on historic General Layout Drawings: 14 m wide, 73 m long, 5 span structure	GEOCRES Report No. 31G5-79	5.2 to 5.8 m very stiff to firm silty clay 1.7 to 4.8 m very loose to very dense till Carlsbad formation shale bedrock Based on GEOCRES Report No. 31G5-79	Silty Clay $\gamma = 17 \text{ kN/m}^3$ $\phi = 27^\circ$ Till $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$	Silty Clay Type 3	Soil Profile Type III Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN / 12BP74 pile allowable reduced to 700 kN / pile for negative skin friction As per GEOCRES Report	NA	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN / 12BP74 pile allowable As per GEOCRES Report	NA	10	2H:1V with 3 to 3.4 m wide mid-height berm	erosion at both abutments was observed	Predicted settlement of 100 mm with 50% occurring within 12 months
13	3-301/2	Hwy 417 WB overpass of CN Rail line 850 m south of Walkley Rd	Based on historic General Layout Drawings: 13 m wide, 71 m long, 5 span structure	GEOCRES Report No. 31G5-79	4.0 to 5.2 m very stiff to firm silty clay 1.5 to 3.5 m very loose to very dense till Carlsbad formation shale bedrock Based on GEOCRES Report No. 31G5-79	Silty Clay $\gamma = 17 \text{ kN/m}^3$ $\phi = 27^\circ$ Till $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$	Silty Clay Type 3	Soil Profile Type III Site is not susceptible to liquefaction	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN / 12BP74 pile allowable As per GEOCRES Report	NA	Shale Bedrock	HP12x74 driven to bedrock ≈845 kN / 12BP74 pile allowable As per GEOCRES Report	NA	10	2H:1V with 3 to 3.4 m wide mid-height berm	erosion at both abutments was observed	Predicted settlement of 100 mm with 50% occurring within 12 months
14	3-265/1	Hwy 417 EB overpass of Ramsayville Rd 2 km west of Anderson Rd	Based on historic General Plan Drawings: 11 m wide, 94 m long, 5 span structure	GEOCRES Report No. 31G5-71	0.9 to 1.5 m clayey silt 31.4 to 38.1 m very stiff to firm clay 11.8 to 13.4 m compact to dense till Carlsbad formation shale bedrock Based on GEOCRES Report No. 31G5-71	Clay $\gamma = 17 \text{ kN/m}^3$ $\phi = 27^\circ$ Till $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$	Clay Type 3	Base on SCPTu testing, site is a Soil Profile Type III. Site is not susceptible to liquefaction	Shale Bedrock	12BP74 driven to bedrock ≈800 kN / 12BP74 pile allowable As per Contract Drawings	NA	Shale Bedrock	12BP74 driven to bedrock ≈800 kN / 12BP74 pile allowable As per Contract Drawings	NA	3.7	2H:1V	Possible tilt of Pier 3 with the construction of Ramsayville Road and Ramsay Creek culvert	Predicted settlement of <150 mm
15	3-265/2	Hwy 417 WB overpass of Ramsayville Rd 2 km west of Anderson Rd	Based on historic General Layout Drawings: 13 m wide, 94 m long, 5 span structure	GEOCRES Report No. 31G5-190	1.5 to 2.7 m clayey silt 30.8 to 38.6 m very stiff to firm clay 11.1 to 13.6 m loose to very dense till Carlsbad formation shale bedrock Based on GEOCRES Report No. 31G5-190	Clay $\gamma = 17 \text{ kN/m}^3$ $\phi = 27^\circ$ Till $\gamma = 20 \text{ kN/m}^3$ $\phi = 32^\circ$	Clay Type 3	Base on SCPTu testing, site is a Soil Profile Type III. Site is not susceptible to liquefaction	Shale Bedrock	12BP74 driven to bedrock ≈845 kN / 12BP74 pile allowable As per GEOCRES Report	NA	Shale Bedrock	12BP74 driven to bedrock ≈845 kN / 12BP74 pile allowable As per GEOCRES Report	NA	3.7	2H:1V	NA	Predicted settlement of <125 mm with majority occurring between 12 to 18 months

Report Summary Table GWP 4074-11-00 GEOCRES No. 31G5-263

Site No.	Site	Recommendations												Additional Work
		Abutment			Preferred Foundation			Unfactored Coefficient of Friction Concrete Cast-in-Place / Precast	Special Concerns	Embankment	Stability Issues	Settlement Issues	Construction Issues	
		Foundation	Vertical Geotechnical Resistance at ULS	Vertical Geotechnical Reaction at SLS	Founding Level / Foundation Type	Vertical Geotechnical Resistance at ULS	Vertical Geotechnical Reaction at SLS			Geometry				
9	3-302/1	HP310x110 piles driven to bedrock	2000 kN / pile	NA	914 mm caisson socket 300 mm into sound bedrock	3150 kN / caisson	NA	NA	-	2H:1V	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified or if excavations are planned within the fills. Investigation should include monitoring wells for developing dewatering plans. The need for vibration monitoring should also be assessed. Coordination with railway, railway design code requirements and track protection will need to be considered.
10	3-302/2	HP310x110 piles driven to bedrock	2000 kN / pile	NA	914 mm caisson socket 300 mm into sound bedrock	3150 kN / caisson	NA	NA	-	2H:1V	NA	NA	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified or if excavations are planned within the fills. Investigation should include monitoring wells for developing dewatering plans. The need for vibration monitoring should also be assessed. Coordination with railway, railway design code requirements and track protection will need to be considered.
11	3-306	HP310x110 piles driven to bedrock	2000 kN / pile	NA	HP310x132 piles driven to bedrock	2400 kN / pile	NA	NA	Abutment: Downdrag = 120 kN/pile Pier: Downdrag is NA unless grades are modified	2H:1V	NA	Widening of Hwy 417 will induce further settlement of existing embankment and roadway	NA	Additional investigations should be carried out in the structure is replaced or widened or if the geometry of the approach fills is modified or if excavations are planned within the fills. Investigation should include piezometer installations for developing dewatering plans, and the need for vibration monitoring should be assessed.
12	3-301/1	HP310x110 piles driven to bedrock	2000 kN / pile	NA	HP310x110 piles driven to bedrock	2000 kN / pile	NA	NA	Abutment: Downdrag = 300 kN/pile Piers 1 & 4: Downdrag = 50 kN/pile Piers 2 & 3: Downdrag is NA unless grades are modified	2H:1V with 3 m mid height bench	NA	Widening of Hwy 417 will induce further settlement of existing embankment and roadway	NA	Additional investigations should be carried out if the structure is replaced or widened or if geometry of the approach fills are modified. Investigation should include piezometer installations for developing dewatering plans. The need for vibration monitoring should also be accessed. Should the structure be widened additional boreholes at the pier locations should be considered for shoring design. Coordination with railway, railway design code requirements and track protection will need to be considered.
13	3-301/2	HP310x110 piles driven to bedrock	2000 kN / pile	NA	HP310x110 piles driven to bedrock	2000 kN / pile	NA	NA	Abutment: Downdrag = 125 kN/pile Piers 1 & 4: Downdrag = 125 kN/pile Piers 2 & 3: Downdrag is NA unless grades are modified	2H:1V with 3 m mid height bench	NA	Widening of Hwy 417 will induce further settlement of existing embankment and roadway	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills are modified. Investigation should include piezometer installations for developing dewatering plans. The need for vibration monitoring should also be assessed. Should the structure be widened additional boreholes at the pier locations should be considered for shoring design. Coordination with railway, railway design code requirements and track protection will need to be considered.
14	3-265/1	HP310x110 or HP310x132 piles driven to practical refusal in the till	1800 kN / pile	1600 kN / pile	HP310x110 or HP310x132 piles driven to practical refusal in the till	1800 kN / pile	1600 kN / pile	NA	Abutment: Downdrag = 300 kN/pile Piers 2 & 3: Downdrag = 250 kN/pile Piers 1 & 4: Downdrag is NA unless grades are modified	2H:1V	Possible with removal of Ramsay Creek culvert (3-444/C)	Widening of Hwy 417 will induce further settlement of existing embankment and roadway	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills are modified or if excavations are planned within the fills. Investigation should include piezometer installations for developing dewatering plans, and the need for vibration monitoring should also be assessed. Should the Ramsay Creek Culvert be removed or replaces a soil investigation and slope stability analysis should be undertaken to ensure the foreslope of the east abutment remains stable. In addition the eccentric loading on Pier 3 should be assessed.
15	3-265/2	HP310x110 or HP310x132 piles driven to practical refusal in the till	1800 kN / pile	1600 kN / pile	HP310x110 or HP310x132 piles driven to practical refusal in the till	1800 kN / pile	1600 kN / pile	NA	Abutment: Downdrag = 500 kN/pile Pier: 1 & 2 Downdrag = 250 kN/pile Piers 3 & 4: Downdrag is NA unless grades are modified	2H:1V	NA	Widening of Hwy 417 will induce further settlement of existing embankment and roadway	NA	Additional investigations should be carried out if the structure is replaced or widened or if the geometry of the approach fills is modified or if excavations are planned within the fills. Investigation should include piezometer installations for developing dewatering plans, and the need for vibration monitoring should be assessed



**APPENDIX 1**  
**SITE 3-303**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Paul Carnaffan, M.Eng., P.Eng.  
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN EASTERN PARKWAY SOUTHBOUND BRIDGE (SITE 3-303) GWP 4074-11-00 GEOCRETS 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the bridge structure carrying traffic on the Aviation Parkway over the Highway 417 / Highway 174 Interchange in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4311-11-01
- Site No. 3-303
- GEOCRETS No. 31G5-86
- Construction Contract 73-192
- Historic W.P. 13-68-08

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa, in the Township of Gloucester. The bridge carries the southbound traffic on the Aviation Parkway (two lanes plus paved shoulders) over the Highway 417 / Highway 174 Interchange. Based on the historic General Plan Drawing (copy attached), the bridge is a 13.4 m to 14.9 m wide, 229.9 m long, six-span structure with a concrete post-tensioned voided deck. The terrain in the vicinity of the bridge is generally flat to undulating with elevations ranging from 70 m to 75 m. Site photos showing the general site conditions are attached.



### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-86 dated December 1972. The investigation consisted of eight sampled boreholes designated 9-16, seven of which were accompanied by dynamic cone penetration tests. Drawing No. 72-11083 C (copy attached) illustrates the locations of the bridge, the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by an occasional surficial layer of silty sand overlying a clayey silt layer, overlaying sand and gravel, underlain by shale bedrock. In some boreholes, a thin layer of glacial till was encountered underlying the sand and gravel layer and immediately above the shale bedrock.

#### **3.1.1 Silty Sand**

A silty sand stratum was encountered in Boreholes 14, 15, and 16 which was described as having some gravel and some clay with occasional clayey silt seams in Borehole 14. The surface of this deposit ranged from 67.7 m to 69.9 m in elevation, and the layer had a thickness of 0.9 m to 2.8 m. The standard penetration test (SPT) 'N' values ranged from 21 to 55 blows per 0.3 m of penetration; indicating a compact to very dense condition. Gradation testing results on two samples of this material indicate gravel content between 1% and 16%, sand content between 40% and 58%, silt content between 20% and 39%, and clay content between 6% and 20%.

#### **3.1.2 Clayey Silt**

A clayey silt deposit was encountered beneath the silty sand deposit in Boreholes 9, 10, 11, and 12. The surface of this deposit ranges from 68.0 m to 68.2 m in elevation, and the layer had a thickness of 0.9 m to 1.1 m. The SPT 'N' values range from 13 to 22 blows per 0.3 m of penetration; indicating a stiff to very stiff consistency.

#### **3.1.3 Sand and Gravel**

A layer of sand and gravel was encountered in all eight boreholes. The layer was described as having trace silt and occasional boulders. The surface of this deposit ranged from 66.8 m to 67.4 m in elevation, and the layer had a thickness of 2.7 m to 4.4 m. The SPT 'N' values varied greatly for this deposit ranging from 8 to over 100 blows per 0.3 m of penetration; indicating a loose to very dense condition. Typically though this deposit was in a compact state. Gradation test results indicate a gravel content between 21% and 64%, sand content between 34% and 68%, and fines content (combined silt and clay) between 2% to 15%.

#### **3.1.4 Glacial Till**

A layer of glacial till was encountered in Boreholes 11 and 13. The till was described as a heterogeneous mixture of sand, gravel, and silt with trace clay. The surface of this deposit ranged from 64.1 m to 64.3 m in elevation, and the layer had a thickness of 1.1 to 1.6 m. The SPT 'N' values range from 57 to over 100 blows per 0.3 m of penetration, indicating a dense to very dense



condition. Gradation test results indicate a gravel content between 35% and 49%, sand content between 31% and 47%, silt content between 14% and 16%, and a clay content of 4%.

### **3.1.5 Bedrock**

Grey shale bedrock was encountered in all eight boreholes with surface elevations ranging from 62.7 m to 64.1 m. Numerous fissures were noted in the bedrock encountered in Boreholes 11 and 12. The bedrock was described as being in a sound condition in Boreholes 10, 13, 14, and 15. Geological mapping suggests the bedrock at this site is shale of the Billings Formation.

### **3.1.6 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations of 65.9 m and 67.9 m.

## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in June 2010 for Structure 3-303 with the report issued January 2011. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering Ltd. staff during the week of July 14, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

- Concrete slope paving was installed for erosion control of the abutment slopes
- Erosion below the slope paving was noted at both abutments
- Side slopes of both abutment embankments were highly vegetated and showed no signs of erosion
- Cracks in the asphalt surface were observed at the ends of the approach slabs
- A slight bump in the asphalt surface was noted just beyond the south approach slab
- The roadway displayed a slight dip approaching the bridge's north abutment
- Piers showed no signs of settlement though standing water was observed around the base of Piers 3 and 5

## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).



Where insufficient earth cover is provided, polystyrene insulation may be used to enhance frost protection for the existing pile caps.

## **5.2 Seismic Considerations**

This site is classified as a Soil Profile Type I in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC). The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the density of the silty sand/sandy silt and the till at this site, these materials are classified as “not susceptible” to liquefaction during the design earthquake event.

## **5.3 Existing Foundations**

As per the Foundation Layout Drawing the existing the bridge abutments were designed to be supported on steel HP12x74 piles driven to bedrock; while the five center piers were designed to be supported on spread footings founded on sound bedrock.

The available construction drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the abutment foundations is 95 tons / 12BP74 pile or approximately 845 kN/pile.

Steel 12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN / HP310x110 pile driven to bedrock. OPSS 903 requires Grade 350W steel for the H-piles.

The Foundation Layout Drawing also indicates that Piers 1 & 2 (northwest) are supported by 4.3 m x 4.3 m footings founded on bedrock, Pier 3 is supported by a 6.7 m x 4.9 m footing founded on bedrock, and Piers 4 & 5 (southeast) are supported by 4.5 m x 4.5 m footings founded on bedrock. Construction notes on the Foundation Layout Drawing indicate that the pier footings were to be founded on sound bedrock. The design loads were not indicated, however the Foundation Design Report recommended the allowable bearing value of 10 tsf or approximately 960 kPa be used for design of spread footings founded on bedrock.



## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Shallow Foundations**

Although spread footings within the overburden are possible, effective dewatering of the sand and gravel strata would be required. Given the anticipated relatively shallow depth to the shale bedrock, and the higher geotechnical resistance offered by the bedrock, spread footings within the overburden are not recommended. In addition, footings for potential widenings should match the elevation of the existing foundation element, i.e. be founded on bedrock.

The factored vertical geotechnical resistance of 1500 kPa at ULS may be used for the preliminary design of shallow foundations, founded on or in sound shale bedrock. The SLS condition will not govern for footings in or on the bedrock. The design of any new foundations on bedrock would need to consider interactions with the existing foundations and potential for undermining of the existing structural elements.

Resistance to lateral forces and sliding resistance between concrete and underlying materials should be evaluated using an unfactored coefficient of friction of 0.5 for cast-in-place concrete and 0.45 for pre-cast concrete on shale bedrock.

Based on current geological mapping, the shale present at this site may be from the Billings Formation which is susceptible to heaving if allowed to weather in the presence of oxygen. The general mechanism is that oxidation of pyrite within the shale produces sulfuric acid, which in turn reacts with calcite in the shale to form gypsum crystals, which occupy a larger volume than the original materials. A by-product of this chain of reactions also tends to increase sulphate levels which can attack buried concrete structures.

The potentially detrimental effects of shale heaving can be avoided by preventing exposure of the shale to oxygen both during construction and long term. This is typically achieved by limiting exposure of the shale to no more than one day prior to covering with a protective layer such as shotcrete or a concrete mud slab. Sulphate resistant cement should be used in such applications.

### **6.2 Deep Foundations – Piles**

Although the depth to bedrock is too shallow at the existing pier locations for piles to be practical, driven piles are considered suitable for the support of abutment foundations perched within the



approach embankments. It should be noted that the bedrock surface elevation ranges from 62.7 m to 64.1 m at this site.

### **6.2.1 Axial Resistance**

Steel piles (Grade 350W steel) end-bearing on sound bedrock at this site may be designed on the basis of the following factored vertical geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

### **6.2.2 Pile Tips**

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### **6.2.3 Integral Abutment Considerations**

As per the Foundation Layout Drawing the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### **6.2.4 Pile Spacing**

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### **6.2.5 Downdrag**

The overburden at this site is relatively thin and incompressible. Downdrag on existing and new piles is not considered a design issue at this site.

### **6.2.6 Lateral Resistance of Piles**

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formula for cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where:

- $k_h$  = coefficient of horizontal subgrade reaction
- $n_h$  = coefficient related to soil density given in Table A
- $z$  = depth of pile embedment (m)
- $B$  = pile width perpendicular to load direction (m)

**Table A:**  $n_h$  Values for Cohesionless Soils

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 67.0	Embankment Fill	3,000
Between 67.0 and 64	Sandy Silt to Silty Sand	2,000
Below 64	Glacial Till	3,500

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B below. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.2.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

- $C$  = pile circumference (m)
- $q_s$  = soil unit shaft friction
- $\Delta z$  = subdivided segment of the embedded length (m)
- $W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)

**Table C:  $\beta$  Values for Driven Piles**

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 67.0	Embankment Fill	20	0.4
Between 67.0 and 64	Sandy Silt to Silty Sand	19	0.4
Below 64	Glacial Till	20	0.5

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D: Static Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Glacial Till	OPSS Granular B Type I / Sandy Silt & Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$32^\circ$	$30^\circ$
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.47	0.5
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.31	0.33

#### 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Glacial Till	OPSS Granular B Type I / Sandy Silt & Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$32^\circ$	$30^\circ$
<b>Yielding Wall</b>			
$K_{AE}$	0.33	0.37	0.40

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Glacial Till	OPSS Granular B Type I / Sandy Silt & Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	20 / 19
Angle of Internal Friction, $\phi$	35°	32°	30°
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36
<b>Non-Yielding Wall</b>			
$K_{AE}$	0.55	0.62	0.66
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil ( $\text{kN/m}^3$ )

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = Total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in the table above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## 6.5 Approach Embankments

Based on the original Foundation Design Report, the maximum height of approach fills were to be on the order of 9.6 m with an embankment slope of 2H:1V (Horizontal:Vertical). The embankment foundation is expected to be stable and no further consolidation settlement is expected unless the fills are reconfigured.



## **6.6 Erosion Control**

Active erosion below the slope paving was noted at both abutments locations at this site.

The slope paving should be repaired where required and maintained and drainage measures should be enhanced beneath the abutments to prevent further erosion below the slope paving.

## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native overburden soils reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

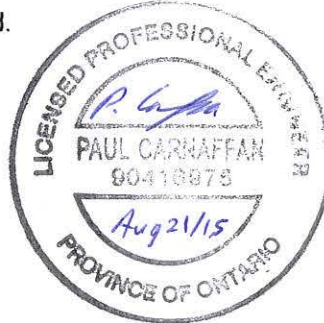
During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one.

Should the proposed works result in excavations to bedrock, samples of the bedrock should be acquired during the investigation and submitted for chemical testing to determine the pyritic heave potential.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.

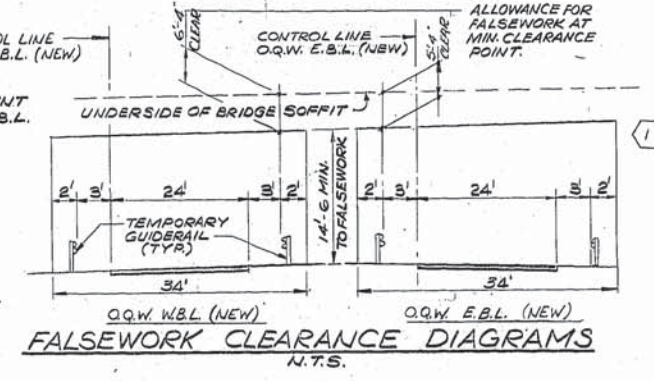
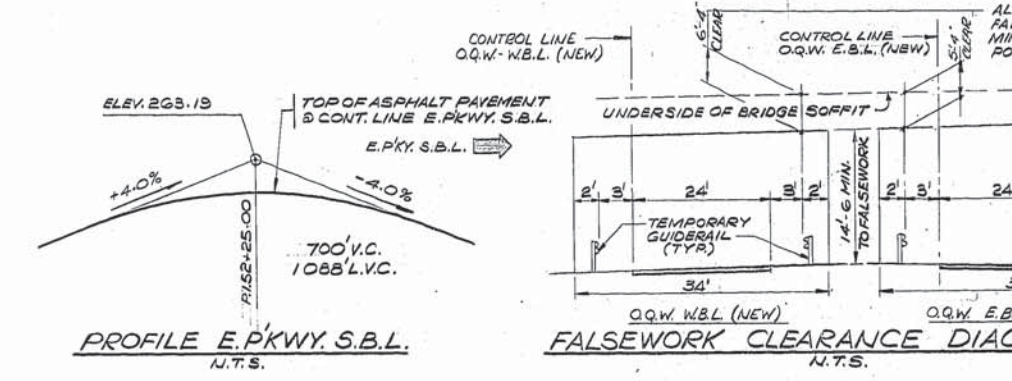
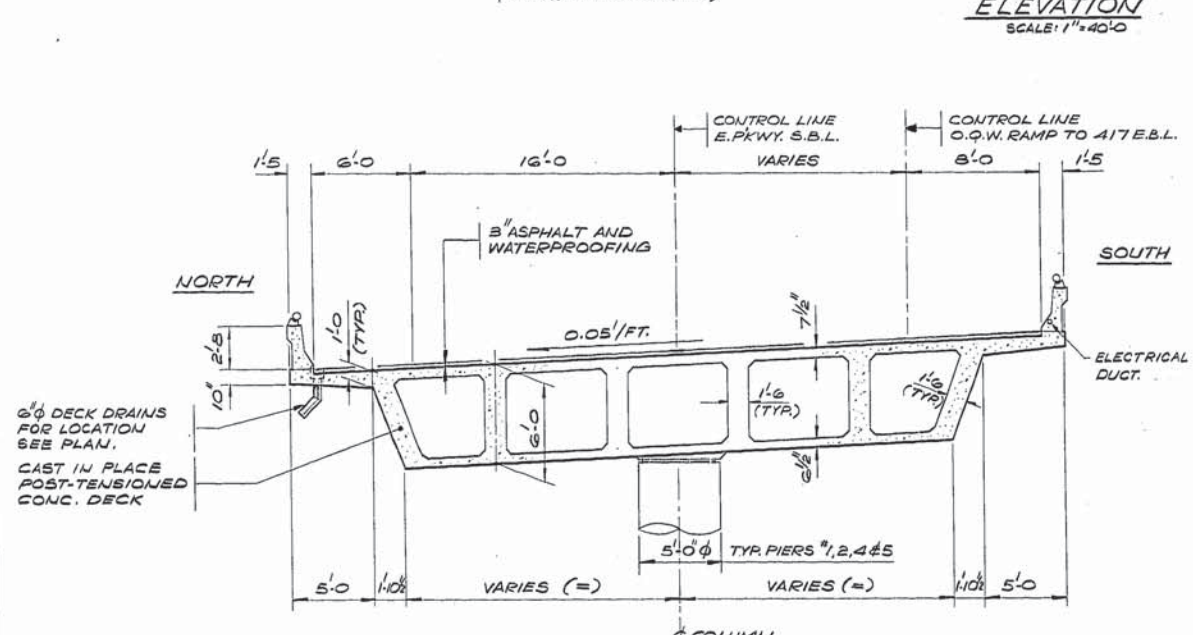
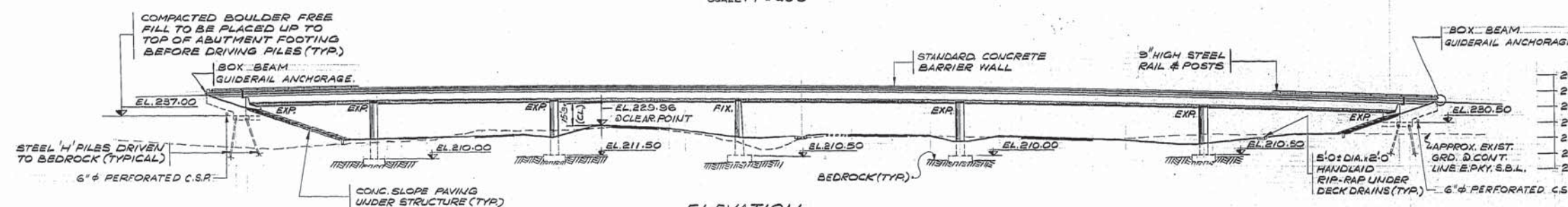
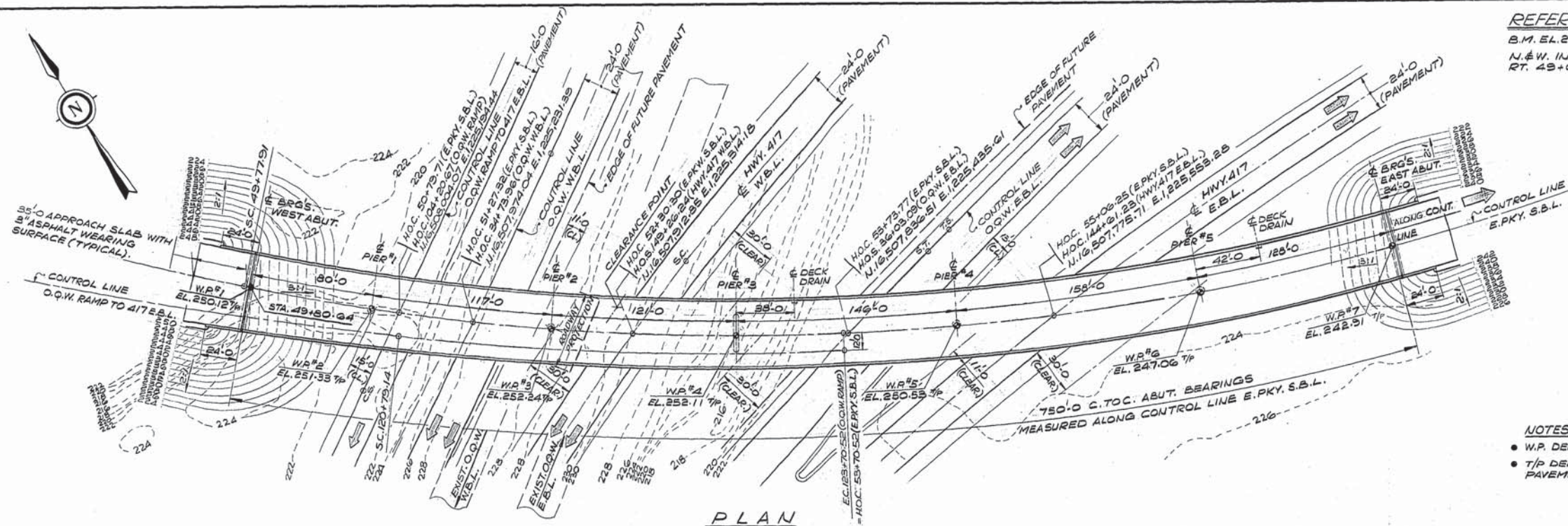


Paul Carnaffan, P.Eng.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

Attachments



**REFERENCE BENCH MARK**  
B.M. EL. 224.09 GEODETIC DATUM  
N.E.W. IN N.E. ROOT OF 2.0' ELM. 85.0'  
RT. 43+04 (E.P.K.W. S.B.L.)

### CURVE DATA:

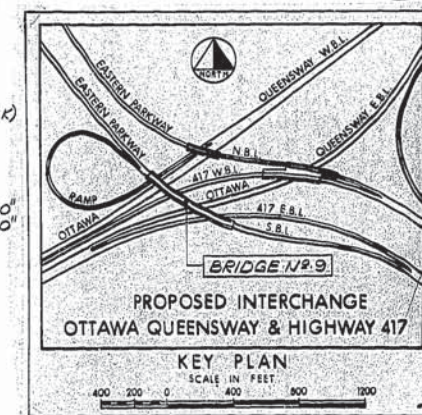
**E.P.K.W. S.B.L.**  
CURVE COM. SPIRAL (BACK)  
Δ = 57°00'00" CS = 4°30'00"  
Δc = 32°37'20" LS = 300.00'  
D = 3°30'00" LT = 118.57'  
R = 1637.02' ST = 81.53'  
LC = 932.14' P = 0.73'  
E = 68.66' D(FLAT) = 1°00'00"  
D(SHARP) = 3°00'00"

### O.Q.W. RAMP TO 417 E.B.L.

CURVE  
Δ = 11°44'29.38"  
Δc = 2°44'29.38"  
D = 3°00'00"  
R = 1909.86'  
LC = 291.38'  
E = 5.57'  
SPIRAL  
Δs = 3°00'00"  
LS = 200.00'  
Ts = 292.17'

### NOTES:

- W.P. DENOTES WORKING POINT
- T/P DENOTES TOP OF ASPHALT PAVEMENT



### LIST OF DRAWINGS

- GENERAL PLAN
- BOREHOLE LOCATIONS & SOIL STRATA
- FOUNDATION LAYOUT
- FOOTING REINFORCEMENT
- PIERS
- WEST ABUTMENT
- EAST ABUTMENT
- DECK LAYOUT
- DECK REINFORCEMENT I
- DECK REINFORCEMENT II
- DECK REINFORCEMENT III
- DECK REINFORCEMENT IV
- CABLE DETAILS I
- CABLE DETAILS II
- CABLE DETAILS III
- BEARINGS & EXPANSION JOINT DETAILS
- CONCRETE BARRIER WALL (2'-8" HIGH)
- DETAILS OF 9" HIGH STEEL RAILING
- APPROACH SLABS
- DETAILS OF CONC. SLOPE PAVING
- STANDARD DETAILS I
- STANDARD DETAILS II
- PLAN-EMBEDDED DETAILS
- ELECTRICAL STANDARD DETAILS

### GENERAL NOTES

#### CLASS OF CONCRETE:

DECK, BARRIER WALLS, PIER COLUMNS & SHAFT, APPROACH SLABS  
REMAINDER

#### CLEAR COVER ON REINFORCING STEEL:

FOOTINGS, ABUTMENTS & PIERS - 3"  
DECK: TOP SLAB - TOP 1 1/2", BOT. 1"  
BOT. SLAB - TOP & BOT. 1"  
WEBS - 1 1/2"  
APPROACH SLABS - 2", BARRIER WALLS - 1 1/2"

#### CONSTRUCTION NOTES:

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8".  
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED, STRESSED AND CROUTED.

REVISIONS	DATE	BY	DESCRIPTION
1	APR 1973	H.S.	(1) FALSEWORK CLEARANCE DIAGRAMS REVISED

### DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

ONTARIO

**DeLew, Cather**

ENGINEERS & PLANNERS - OTTAWA

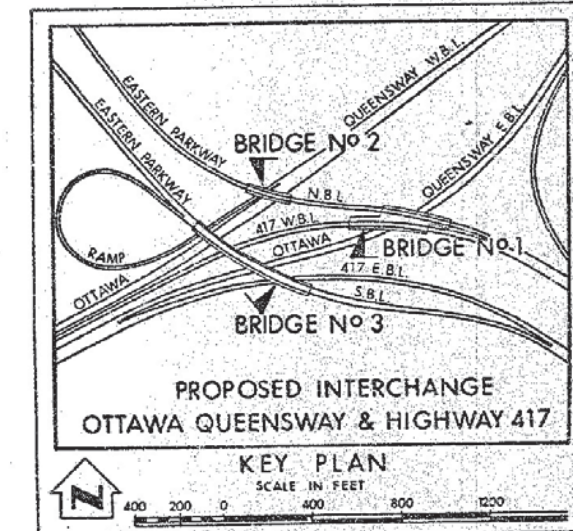
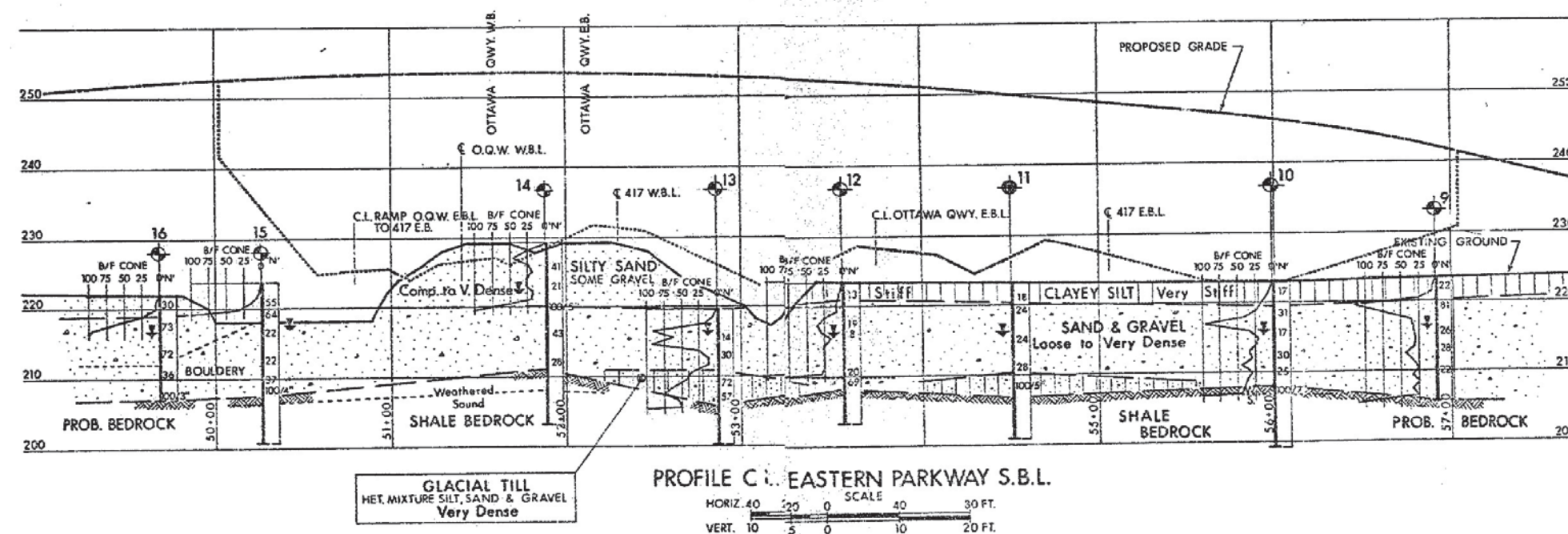
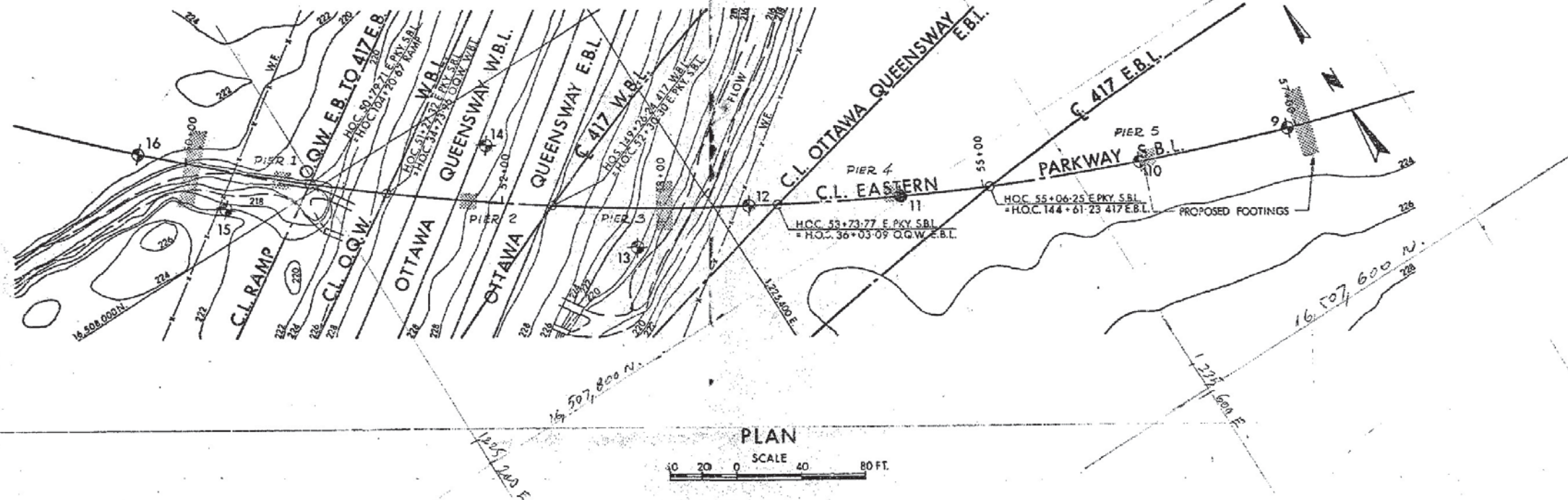
### EASTBOUND OVERPASS OF O.Q.W. BRIDGE No. 9.

KING'S HIGHWAY No. 417 DIST. No. 2  
CO. REG. MUNICIPALITY OF OTTAWA - CARLETON  
TWP. GLOUCESTER LOT 25 CON. II

### GENERAL PLAN

APPROVED	DATE	BY	DESCRIPTION
DESIGN	G.S.S.	CHECK	L.O.H.
DRAWING	R.A.P.	CHECK	G.S.S.
DATE	APR 73	LOADING	HS20 44

57037 Twp. 56-303-1-A



LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation Aug. & Oct. 1972.		
NO.	ELEVATION	CO-ORDINATES NORTH	EAST
9	223.6	16,507,706	1225,729
10	223.2	507,739	225,639
11	223.3	507,799	225,502
12	223.5	507,844	225,420
13	219.8	507,862	225,347
14	229.3	507,967	225,300
15	223.7	508,018	225,141
16	222.1	508,078	225,112

**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO  
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

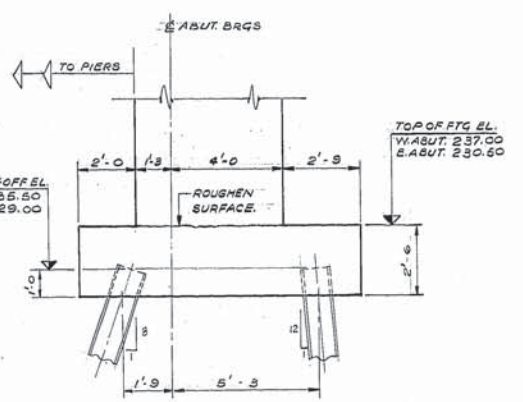
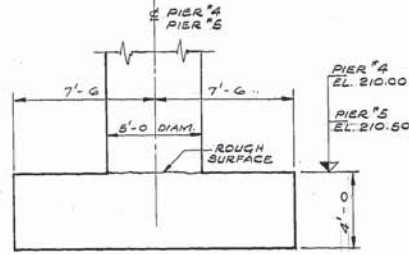
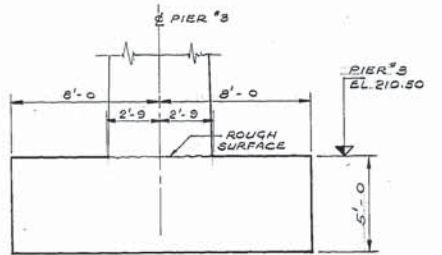
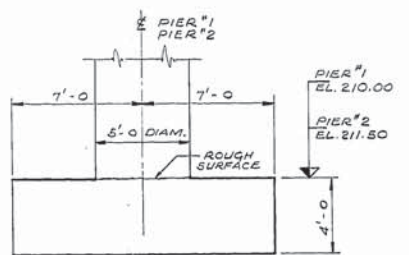
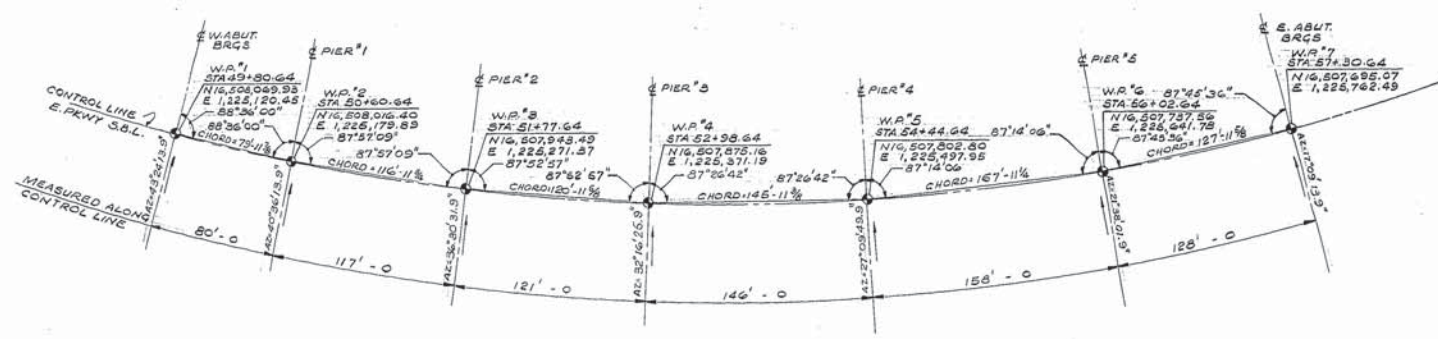
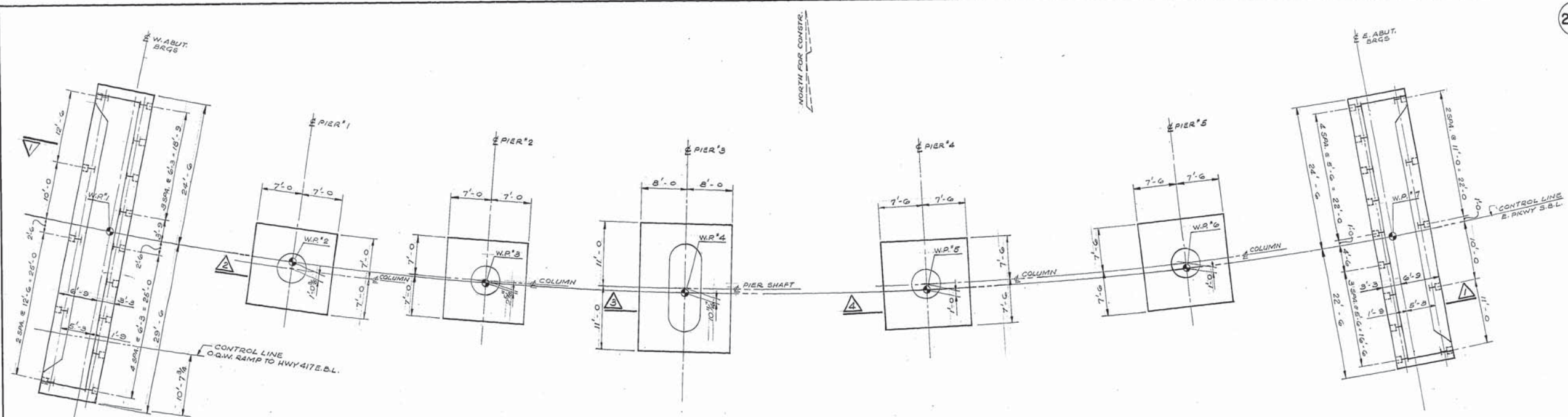
**BRIDGE No 3**  
EASTERN PARKWAY S.B. OVER OTTAWA QUEENSWAY & HWY. 417

HIGHWAY NO. 417 DIST. NO. 9  
CO. REGIONAL MUNICIPALITY OF OTTAWA—CARLETON  
TYP. GLOUCESTER LOT 25 CON. II

**BORE HOLE LOCATIONS & SOIL STRATA**

SUBMIT S.A. CHECKED	W.P. NO. 13-68-08	DRAWING NO.
DRAWN J.G. CHECKED	W.O. NO. 72-11083	72-11083C
DATE NOV. 22, 1972	SITE NO.	BRIDGE DRAWING NO.
APPROVED	CONT. NO.	

REF. N° B-56-26 & E-5239-1



STEEL 'H' PILE DATA			
LOCATION	NE	LENGTH	TYPE
W.ABUT.	14	34'-0	HPI2 x 74
E.ABUT.	14	28'-0	(TYP)

- NOTES:
- PILES TO BE DRIVEN TO BEDROCK.
  - SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTINGS.
  - PIER FOOTINGS TO BE FOUNDED ON SOUND BEDROCK.



REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO  
**De Leuw, Cather**  
ENGINEERS & PLANNERS - OTTAWA

**EASTBOUND OVERPASS OF O.Q.W. BRIDGE No. 9.**

KING'S HIGHWAY No. 417 DIST. No. 9  
CO. REG. MUNICIPALITY OF OTTAWA - CARLETON  
TWP. GLOUCESTER LOT 25 CON. II

**FOUNDATION LAYOUT**

APPROVED: [Signature] SITE No. 3-303 W.F. No. 73-68-08  
CONTRACT No. 72-192

DESIGN L.D.H. CHECK G.S.S.  
DRAWING K.A.B. CHECK R.A.P.  
DATE APR. 73 LOADING H520.44 DRAWING No. 3-303-3





**APPENDIX 2**  
**SITE 3-304/2**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Paul Carnaffan, M.Eng., P.Eng.  
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 - EASTERN PARKWAY NORTHBOUND BRIDGE (SITE 3-304/2) GWP 4074-11-00 GEOCRETS 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the bridge structure carrying traffic from Highway 417 westbound to the Aviation Parkway northbound over Highway 174 westbound in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4310-11-01
- Site No. 3-304/2
- GEOCRETS No. 31G5-86
- Construction Contract 73-192
- Historic W.P. 13-68-03

### **2 SITE DESCRIPTION**

The bridge site is located in eastern Ottawa in the Township of Gloucester. Based on the historic General Plan Drawing (copy attached), the bridge is a 10.1 m wide, 89.9 m long, three-span structure with a concrete post-tensioned voided deck. The bridge abutments are supported by steel HP12x53 piles driven to bedrock, while the center piers were founded on the existing native soils. The terrain in the vicinity of the bridge is generally flat to undulating with elevations ranging from 70 m to 75 m. Site photos showing the general site conditions are attached.



### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRETS Report No. 31G5-86 dated December 1972. The investigation was conducted in July 1971 and consisted of three sampled boreholes designated Borehole 6, 7 and 8; all of which were accompanied by dynamic cone penetration tests (DCPT). Drawing No. 72-11083B (copy attached) illustrates the locations of the bridge and the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by a surficial layer of fill or clayey silt overlying silty sand over a thin layer of glacial till underlain by shale bedrock.

#### **3.1 Fill**

A 1.4 m thick surficial layer of fill was encountered in Borehole 7. The fill was described as silty sand with gravel and trace clay. Gradation test results indicate a gravel content of 38%, sand content of 35%, silt content of 22%, and clay content of 5%. The moisture content was reported to be 8%.

#### **3.2 Clayey Silt**

A layer of clayey silt was encountered at the ground surface of Borehole 6. The layer was 1.4 m thick and contained some sand. The standard penetration test (SPT) 'N' value in this layer is 12 blows per 0.3 m of penetration, indicating a stiff soil consistency. Gradation test results indicate a gravel content of 0%, sand content of 11%, silt content of 66%, and clay content of 23%. The moisture content was reported to be 21%.

#### **3.3 Silty Sand**

A silty sand to sand layer was encountered beneath the surficial layers in Borehole 6 and 7 and at the ground surface of Borehole 8. The layer was described as silty sand having trace to some gravel in Boreholes 7 and 8 to a silty sand with gravel in Borehole 6. The surface of this deposit ranged from 67.2 m to 68.1 m in elevation, and the layer had a thickness of 2.7 m to 4.1 m. The SPT 'N' values ranged from 9 to 70 blows per 0.3 m of penetration; indicating a loose to dense condition with occasional very dense zones. Gradation test results indicate a gravel content between 8% and 27%, sand content between 60% and 89%, and fines content (combined silt and clay) between 3% and 18%. The moisture content was reported to range from 8% to 18%.

#### **3.4 Glacial Till**

A layer of glacial till was encountered in all three boreholes beneath the silty sand layer. The glacial till was described as a heterogeneous mixture of silt, sand, and gravel, with trace clay. The surface of this deposit ranged from 63.9 m to 65.2 m in elevation, and the layer had a thickness of 0.1 m to 1.6 m. The SPT 'N' values ranged from 29 to over 100 blows per 0.3 m of penetration, indicating a compact to very dense deposit. Gradation test results indicate a gravel content from 18% to 39%, sand content from 39% to 52%, silt content from 13% to 25%, and clay content from 5% to 6%. The moisture content was reported to range from 2% to 10%.



### **3.5 Bedrock**

Grey shale bedrock was encountered beneath the glacial till in all three boreholes with surface elevations ranging from 62.4 m to 64.0 m. The shale bedrock was identified as being in a sound condition in Borehole 7. The upper zones in Boreholes 6 and 8 were identified as being weathered with the lower elevations described as being in a strong condition. Geological mapping suggests the bedrock at this site is shale of the Billings Formation.

### **3.6 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations of 66.5 m and 66.8 m at the time of the original investigation.

## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in July 2012 for Bridge 3-304/2 with the report issued October 2013. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering staff during the week of July 14, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

- Concrete slope paving was installed for erosion control of the abutment slopes
- No evidence of erosion below the slope paving at either abutment was observed
- The side slopes of the approach embankments were noted to be heavily vegetated and showed no signs of erosion
- There were no signs of settlement observed at the north embankment
- A gap was observed between the south abutment face and upper concrete tiles;
- Pavement cracks were observed along the ends of both approach slabs
- A moderate dip was observed south of the south abutment, suggesting some settlement of the embankment

## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

Where insufficient earth cover is provided, polystyrene insulation may be used to enhance frost protection of the existing pile caps and footings.



## 5.2 Seismic Considerations

This site is classified as a Soil Profile Type I in accordance with the Canadian Highway Bridge Design Code (CHBDC) Section 4.4.6. The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the density of the silty sand/sand, and the till at this site, these materials are classified as “not susceptible” to liquefaction during the design earthquake event.

## 5.3 Existing Foundations

As per the Footings & Pier Columns Drawing (copy attached), the bridge abutments were designed to be supported on steel HP12x53 piles driven to bedrock.

The available construction drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the abutment foundations is 95 tons / 12BP74 pile or approximately 845 kN/pile.

HP12x53 piles are nominally equivalent in dimension and mass to HP310x79 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 1450 kN / HP310x79 pile driven to bedrock. OPSS 903 requires Grade 350 W steel for the H-piles.

The Footings and Pier Columns Drawing also indicates that the existing bridge Piers 1 and 2 are supported by 4.9 m x 4.9 m, footings founded at elevations 64.5 m and 63.9 m respectively. Based on soil stratigraphy reported on the borehole records the footings are founded on glacial till. The available construction drawings do not indicate the design loads for the pier foundations; however, the Foundation Design Report recommended an allowable bearing pressure of 5 tsf or approximately 480 kPa for spread footing founded on glacial till.

## 6 GEOTECHNICAL RECOMMENDATIONS

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### 6.1 Shallow Foundations

Shallow foundation founded on undisturbed dense to very dense glacial till at the same elevation as the existing piers and having a minimum width of 3 m may be designed using a factored geotechnical resistance at ULS of 600 kPa. The vertical geotechnical reaction at SLS is 400 kPa based on a total footing settlement of 25 mm.



For preliminary design purposes, the factored vertical geotechnical resistance for footings on or in sound shale bedrock is 1,500 kPa at ULS. The SLS condition will not govern for footings in or on the bedrock. The design of any new foundations on bedrock would need to consider interactions with the existing foundations and potential for undermining of the existing structural elements.

Resistance to lateral forces and sliding resistance between concrete and underlying materials should be evaluated using an unfactored coefficients of friction provided in Table A.

**Table A:** Unfactored Coefficients of Friction between Concrete and Founding Material

Concrete Type	Founding Material	
	Glacial Till	Shale Bedrock
Cast-in-place concrete	0.45	0.50
Precast concrete	0.40	0.45

Based on current geological mapping, the shale present at this site may be from the Billings Formation which is susceptible to heaving if allowed to weather in the presence of oxygen. The general mechanism is that oxidation of pyrite within the shale produces sulfuric acid, which in turn reacts with calcite in the shale to form gypsum crystals, which occupy a larger volume than the original materials. A by-product of this chain of reactions also tends to increase sulphate levels which can attack buried concrete structures.

The potentially detrimental effects of shale heaving can be avoided by preventing exposure of the shale to oxygen both during construction and long term. This is typically achieved by limiting exposure of the shale to no more than one day prior to covering with a protective layer such as shotcrete or a concrete mud slab. Sulphate resistant cement should be used in such applications.

## **6.2 Deep Foundations – Piles**

Although the depth of bedrock is too shallow at the existing pier locations for driven piles to be practical, driven piles are considered to be suitable for the support of abutment foundations perched within the approach embankments. It should be noted that the bedrock surface elevation ranges from 62.4 m to 64.0 m at this site.

### **6.2.1 Axial Resistance**

Steel piles (Grade 350W steel) end-bearing on sound bedrock at this site may be designed on the basis of the following factored vertical geotechnical resistances at ULS:

- 1,450 kN per HP310x79 pile; and
- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.



### 6.2.2 Pile Tips

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### 6.2.3 Integral Abutment Considerations

As per the Footings & Pier Columns Drawing the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### 6.2.4 Pile Spacing

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### 6.2.5 Downdrag

The overburden at this site is relatively thin and incompressible. Downdrag on existing and new piles is not considered a design issue at this site.

### 6.2.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formula for cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where:

$k_h$  = coefficient of horizontal subgrade reaction

$n_h$  = coefficient related to soil density given in Table B

$z$  = depth of pile embedment (m)

$B$  = pile width perpendicular to load direction (m)

**Table B:**  $n_h$  Values For Cohesionless Soils

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 68.0	Embankment Fill	3,000
Between 68 and 67	Clayey Silt	2,500

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Between 67 and 63	Silty Sand	2,000
Below 63	Glacial Till	3,500

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table C. Intermediate values may be obtained by linear interpolation.

**Table C:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.2.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table D

$\sigma'_v$  = effective vertical stress (kPa)

**Table D:  $\beta$  Values for Driven Piles**

<b>Elevation (m)</b>	<b>Soil Description</b>	<b>Unit Weight, <math>\gamma</math>, (kN/m<sup>3</sup>)</b>	<b><math>\beta</math></b>
Above 68.0	Embankment Fill	20	0.4
Between 68 and 67	Clayey Silt	19	0.4
Between 67 and 63	Silty Sand	19	0.4
Below 63	Glacial Till	20	0.5

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient (see below)

$\gamma$  = unit weight of retained soil (see below)

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table E are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table E: Static Lateral Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Glacial Till	OPSS Granular B Type I / Silty Sand	Native Clayey Silt
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	20 / 19	19
Angle of Internal Friction, $\phi$	35°	32°	30°	29°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.47	0.50	0.52
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.31	0.33	0.35

#### 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table F.

**Table F: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Glacial Till	OPSS Granular B Type I / Silty Sand	Native Clayey Silt
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	20 / 19	19
Angle of Internal Friction, $\phi$	35°	32°	30°	29°
<b>Yielding Wall</b>				
$K_{AE}$	0.33	0.37	0.40	0.42
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36	0.36
<b>Non-Yielding Wall</b>				
$K_{AE}$	0.55	0.62	0.66	0.69
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that considers of material properties and the soil profile:



$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth, d (kPa)

d = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

H = Total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in the tables above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## 6.5 Approach Embankments

Based on the original Foundation Design Report, the existing embankments are up to 8 m high and have 2H:1V (Horizontal:Vertical) side slopes. The embankment foundation is expected to be stable and no long term settlement problems are expected unless the fills are reconfigured.

## 6.6 Erosion Control

The slope paving installed at the abutments should be repaired where required and maintained and drainage measures should be enhanced beneath the abutments to prevent erosion below the slope paving.

## 6.7 Excavations and Backfilling

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native silty sand and glacial till reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA. The native clayey silt reported at this site should be classified as Type 3.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.



Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

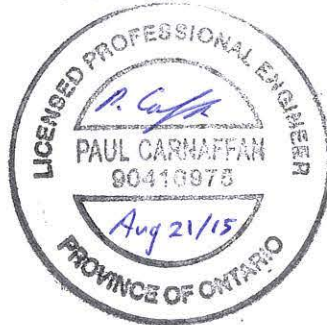
During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one.

Should the proposed works result in excavations to bedrock, samples of the bedrock should be acquired during the investigation and submitted for chemical testing to determine the pyritic heave potential.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.

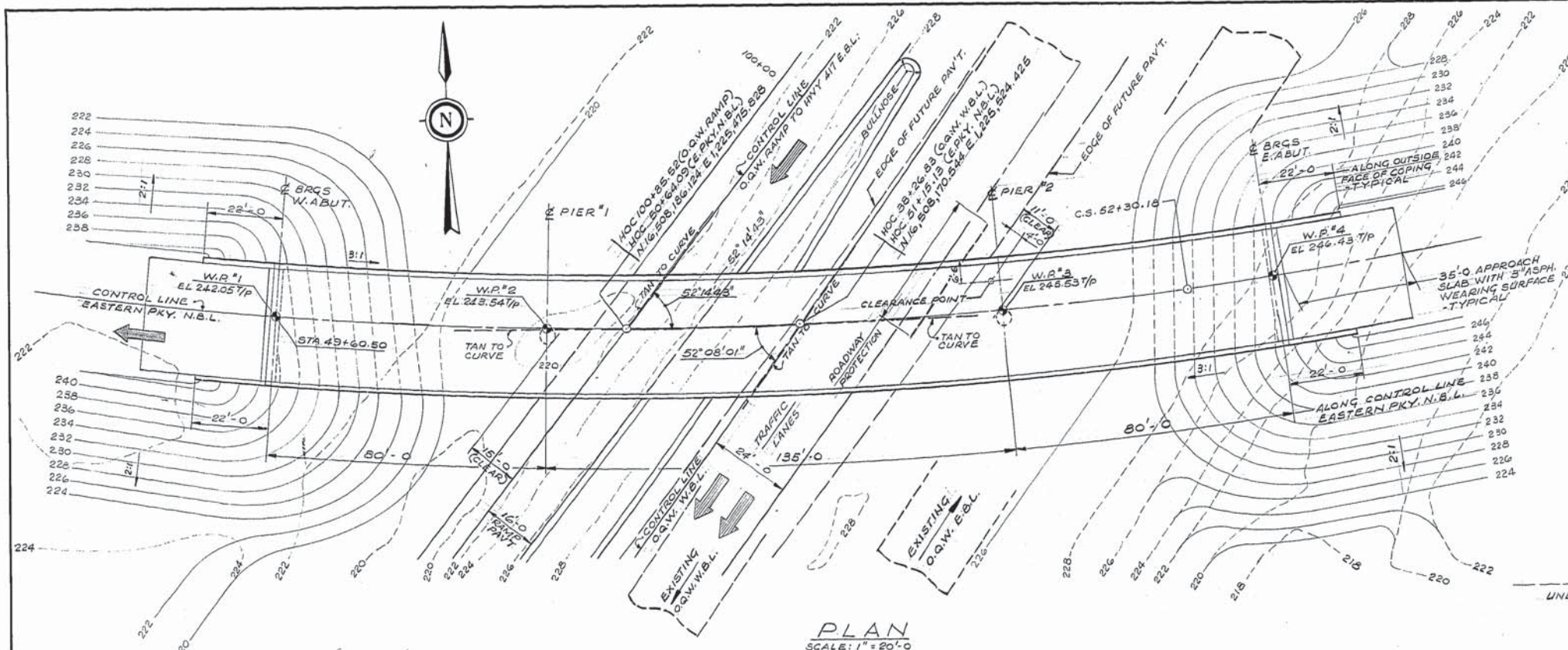


Paul Carnaffan, M. Eng., P. Eng.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P. Eng.  
Review Principal, Designated MTO Contact

Attachments



REFERENCE BENCH MARK  
B.M. EL 224.09 GEODETIC DATUM  
N & W IN S. ROOT OF 2'-0" ELM  
276.0' LT. 147+28 (417 W.B.L.)

CURVE DATA:

O.Q.W. RAMP TO HWY 417 E.B.L.  
Δ = 1° 52' 13.31"  
D = 0° 15' 00"  
R = 22918.31'  
L = 614.81'  
T = 307.48'  
O.Q.W. W.B.L.  
Δ = 3° 56' 0.57"  
D = 1° 00' 00"  
R = 5729.58'  
L = 393.35'  
T = 196.75'

EASTERN PKY. N.B.L.

Δ = 4° 55' 00"  
Δc = 41° 03' 00"  
D = 4° 30' 00"  
R = 1273.24'  
Lc = 912.22'  
E = 86.31'

SPIRAL

Qs = 4° 30' 00"  
Ls = 200'  
Ts = 536.40' S<sub>2</sub> = 633.26'

NOTES:

- W.P. DENOTES WORKING POINT.
- T/P DENOTES TOP OF ASPHALT PAVEMENT.

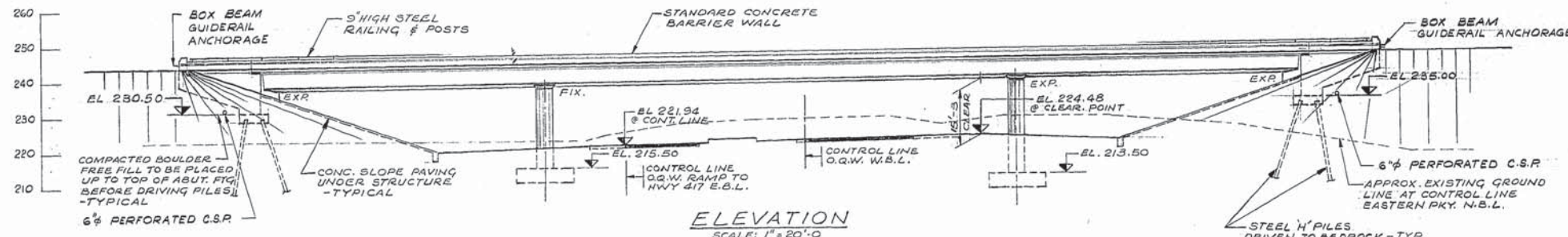


LIST OF DRAWINGS

- GENERAL PLAN.
- BOREHOLE LOCATIONS & SOIL STRATA.
- FOOTINGS & PIER COLUMNS.
- WEST ABUTMENT.
- EAST ABUTMENT.
- DECK LAYOUT & BEARINGS.
- DECK REINFORCEMENT I.
- DECK REINFORCEMENT II.
- LONGITUDINAL CABLES.
- TRANSVERSE CABLES.
- CONCRETE BARRIER WALL (2'-8" HIGH)
- DETAILS OF 9" HIGH STEEL RAILING
- APPROACH SLABS.
- DETAILS OF CONCRETE SLOPE PAVING.
- STANDARD DETAILS I.
- STANDARD DETAILS II.
- PLAN - EMBEDDED DETAILS.
- ELECTRICAL STANDARD DETAILS

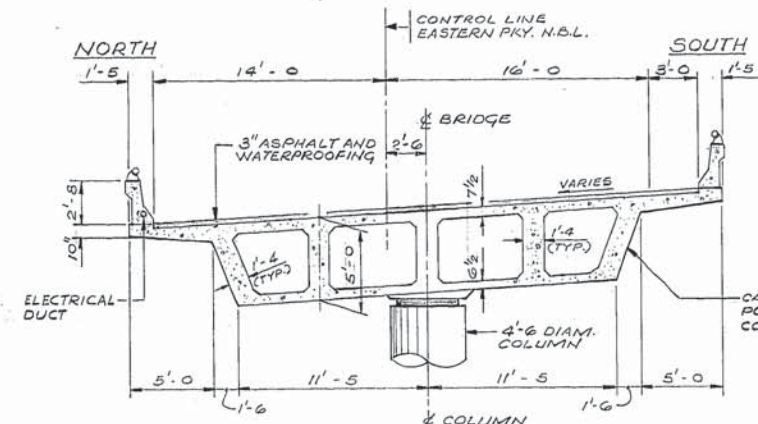
GENERAL NOTES

CLASS OF CONCRETE:  
DECK, BARRIER WALLS  
PIER COLUMNS, APPROACH SLABS - 5000 RS.I.  
REMAINDER - 3000 RS.I.  
CLEAR COVER ON REINFORCING STEEL:  
FOOTINGS, ABUTMENTS & PIERS - 3"  
DECK: TOP SLAB - TOP 1 1/2" BOT. 1"  
BOT. SLAB - TOP 4 BOT. 1"  
WEBS 1 1/2"  
BARRIER WALLS 1 1/2" APPROACH SLABS 2"  
CONSTRUCTION NOTES:  
THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8".  
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED, STRESSED & GROUTED.



FALSEWORK CLEAR. DIAGRAM

N.T.S.



PROFILE - O.Q.W. RAMP TO HWY 417 E.B.L.

N.T.S.

TOP OF ASPHALT PAV'T AT CONTROL LINE RAMP TO HWY 417 E.B.L.

CONTROL LINE O.Q.W. W.B.L.

EL 221.94

±0.36%

TOP OF ASPHALT PAV'T AT CONTROL LINE O.Q.W. W.B.L.

CONTROL LINE O.Q.W. W.B.L.

EL 223.14

±0.36%

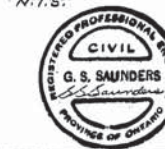
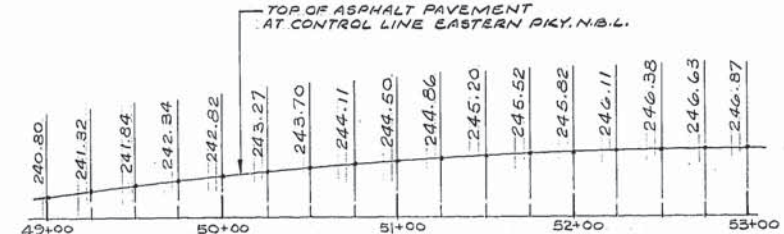
PROFILE - O.Q.W. W.B.L.

N.T.S.

PROFILE - EASTERN PKY. N.B.L.

(SPLINED)

N.T.S.



FOR REDUCED PLAN

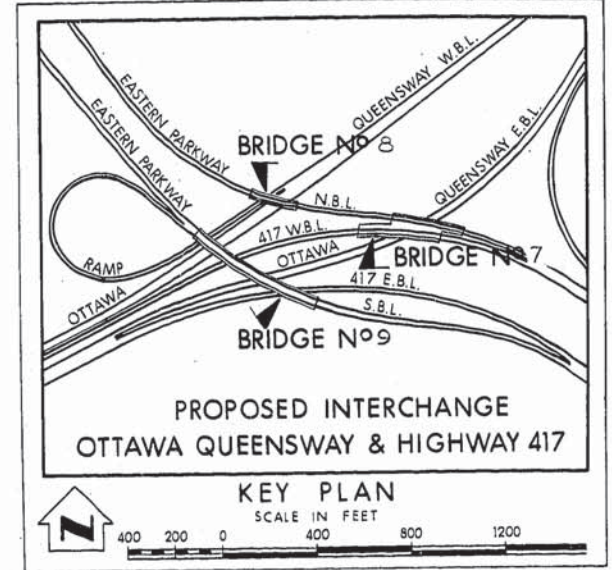
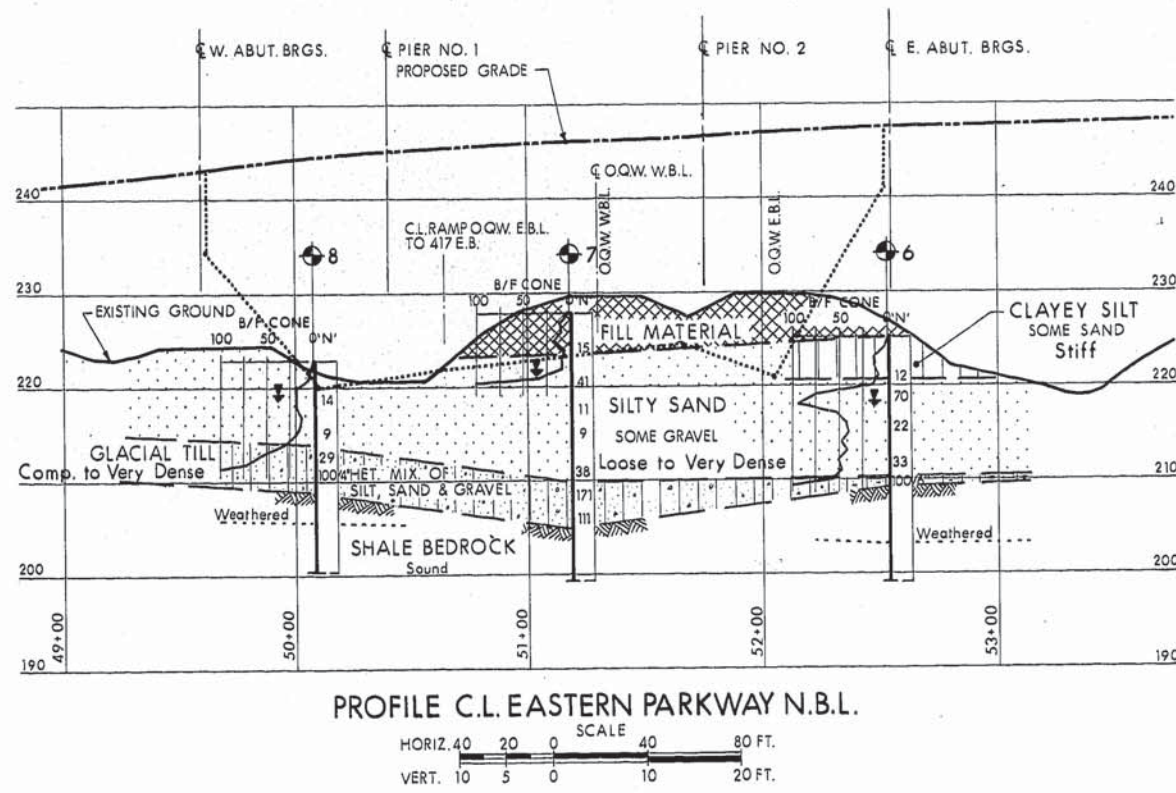
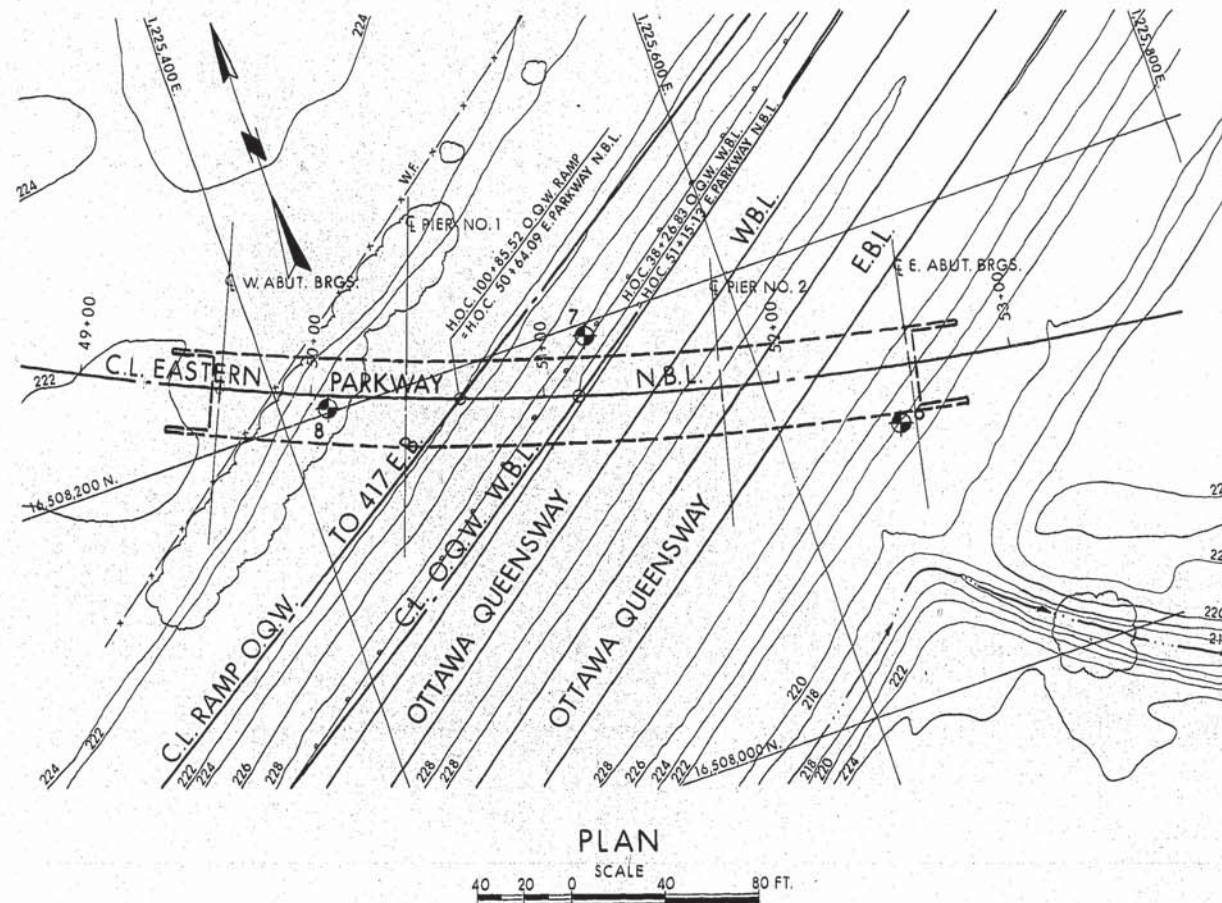
USE SCALE BELOW

10 11 12 13

3 INCHES ON ORIGINAL PLAN

REVISIONS	DATE	BY	DESCRIPTION
1	APR 73	H.J.D.	(1) CLEARANCE DIAGRAM REVISED

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO			
De Leuw, Cather ENGINEERS & PLANNERS - OTTAWA			
WESTBOUND OVERPASS OF O.Q.W. TO EASTERN PARKWAY BRIDGE No. 8			
KING'S HIGHWAY No. 417		DIST. No. 9	
CO. REG. MUNICIPALITY OF OTTAWA - CARLETON			
TWP. GLOUCESTER		LOT 26 CON. II	
GENERAL PLAN			
APPROVED	DATE	SITE No.	W.P. No.
G.S.S.	APR 73	3-304B	73-68-03
CHECK	LOADING	CONTRACT	NO.
G.S.S.	11/20/74		73-192
DRAWING	DATE	DRAWING	No.
K.A.B.	APR 73	3-304B-1	



LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation July 1972		

NO.	ELEVATION	CO-ORDINATES NORTH	EAST
6	225-1	16,508,115	1,225,651
7	227-8	508,195	225,535
8	222-8	508,201	225,420

— NOTE —  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO  
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

**BRIDGE No 8**  
EASTERN PARKWAY N.B. OVER OTTAWA QUEENSWAY W.B.  
HIGHWAY NO. 417 DIST. NO. 9  
CO. REGIONAL MUNICIPALITY OF OTTAWA—CARLETON  
TWP. GLOUCESTER LOT 25 CON. II

**BORE HOLE LOCATIONS & SOIL STRATA**

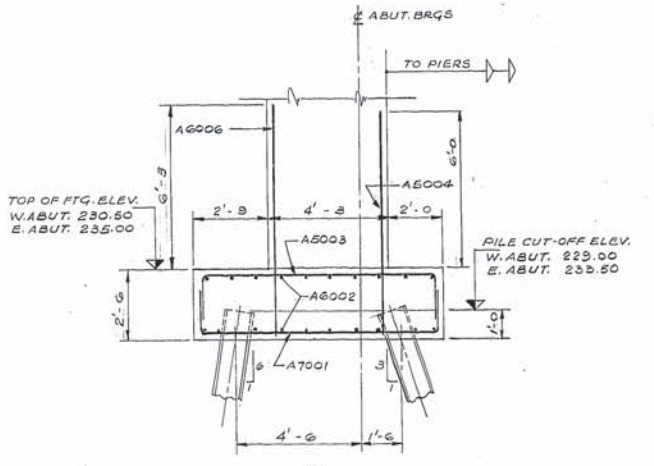
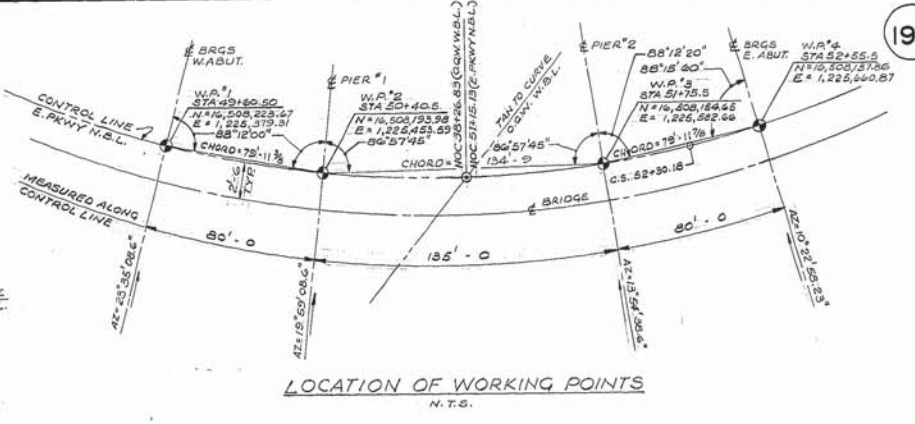
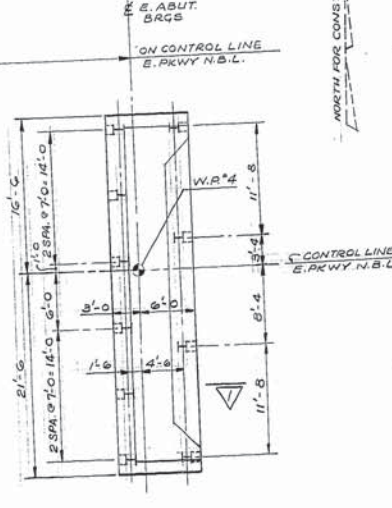
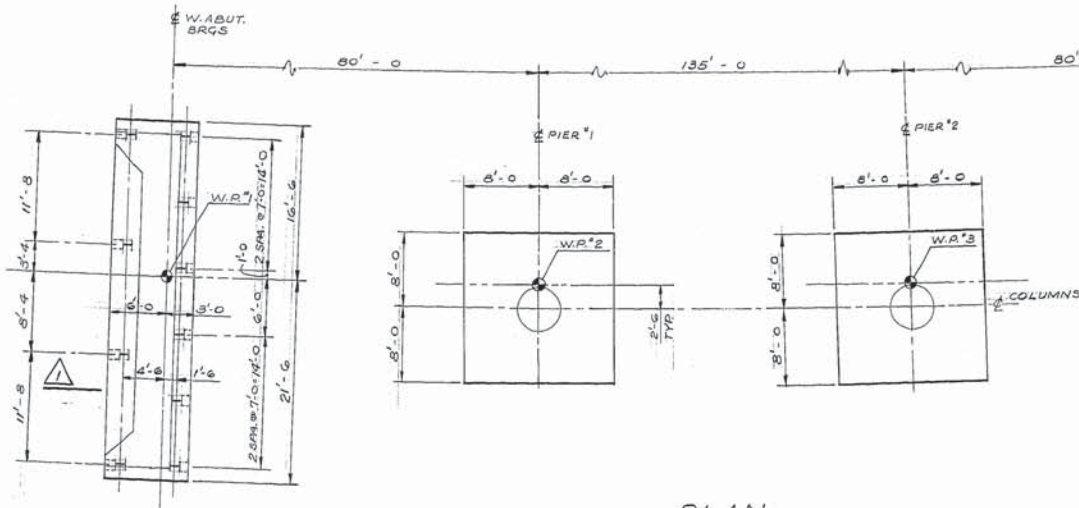
SUBMD. S.A. CHECKED <input checked="" type="checkbox"/>	W.P. NO. 13-68-03	DRAWING NO. 72-11083B
DRAWN J.I.G. CHECKED <input checked="" type="checkbox"/>	W.O. NO. 72-11083	BRIDGE DRAWING NO. 3-304B-2
DATE NOV. 21, 1972	SITE NO. 3-304B	CONT. NO. 73-192
APPROVED <i>[Signature]</i>	PRINCIPAL FOUNDATION ENGINEER	

NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the Ottawa District Office.

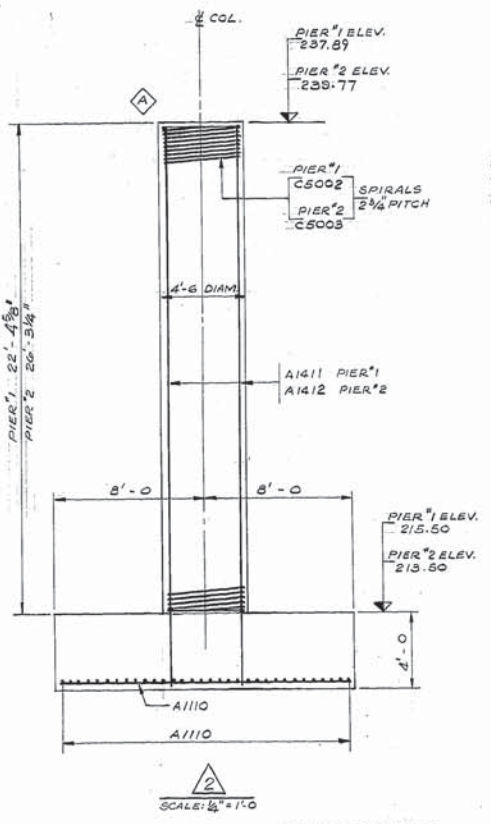
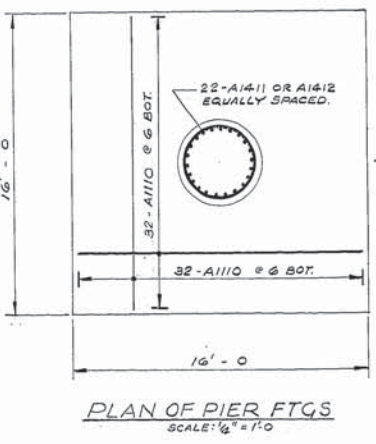
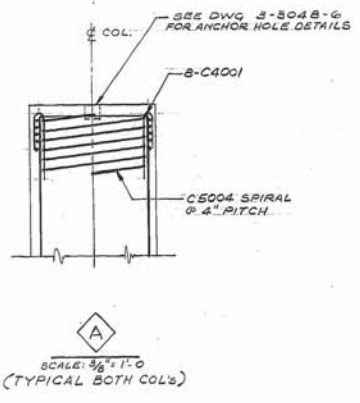
REF. N° B-56-28  
REF. N° E-5240-1

W.B.-13-e8-03

3-304B-S



LOCATION	Nº	LENGTH	TYPE
W. ABUT.	10	24'-0"	HP12x53
E. ABUT.	10	30'-0"	(TYR)



- NOTES:
- PILES TO BE DRIVEN TO BEDROCK.
  - SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTINGS.

No.	FOR	DATE

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO

**DeLew, Cather**  
ENGINEERS & PLANNERS - OTTAWA

**WESTBOUND OVERPASS OF O.Q.W.  
TO EASTERN PARKWAY  
BRIDGE Nº 8**

KING'S HIGHWAY No. 417 DIST. No. 9  
CO. REG. MUNICIPALITY OF OTTAWA-CARLETON  
TWP. GLOUCESTER LOT 25 CON. II

**FOOTINGS & PIER COLUMNS**

APPROVED: [Signature] SITE No. 3-3048 W.P. No. 73-68-03  
DESIGN: A.G. CHECK: L.D.H. CONTRACT No. 113-192  
DRAWING: K.A.B. CHECK: R.A.P. DRAWING No. 3-3048-3  
DATE: APR. 73 LOADING: HS20-44

57072 TWP-51-304 B-3A

W.B-13-68-03

3-3048-3



**APPENDIX 3**  
**SITE 3-304/1**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Paul Carnaffan, M.Eng., P.Eng.  
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN EASTERN PARKWAY NORTHBOUND "Y" BRIDGE (SITE 3-304/1) GWP 4074-11-00 GEOCRETS 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the twin bridge structure carrying the Highway 417 westbound lanes and the ramp connecting Highway 417 westbound to the northbound Aviation Parkway over the eastbound lanes of Highway 174 in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4312-11-01
- Site No. 3-304/1
- GEOCRETS No. 31G5-86
- Construction Contract 73-192
- Historic W.P. 13-68-02

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa in the Township of Gloucester. Based on the historic General Plan Drawing (copy attached), the bridge is actually two structures in a "Y-shaped" configuration. The north structure has three spans and a total length of 81.4 m. The south structure has three spans and a total length of 99.4 m. Both structures have concrete post-tensioned voided decks. Retaining walls are present at four locations around the abutments. The natural terrain in the vicinity of the bridge is generally flat to undulating with elevations ranging from 66.8 m to 67.8 m. finished grade at the abutments ranges from elevation 74.1 m at the north west abutment to 76.0 m at the south east abutment. Embankment slopes were generally designed at 2H:1V (Horizontal: Vertical) although the forward slope at the south west abutment is indicated to be 3H:1V. Site photos showing the general site conditions are attached.



### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-86 dated December 1972. The investigation was conducted in July 1971 and consisted of five sampled boreholes designated Borehole 1 through 5; all of which were accompanied by dynamic cone penetration tests. Drawing No. 72-11083A (copy attached) illustrates the locations of the bridge and the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by a surficial layer of organics, over a layer of silty sand to sand which transitions into sand and gravel towards the east, all of which are underlain by shale bedrock. In Borehole 5, a thin layer of glacial till was reported immediately above the shale bedrock.

#### **3.1 Ground Surface Cover**

A surficial layer of organics was encountered in all boreholes except Boreholes 5. The layer ranged in thickness from 0.2 m to 0.8 m. The soil was black in colour and was described as organic silt, organic matter, and topsoil. Field shear vane tests indicate a soft to firm consistency.

#### **3.2 Sand and Gravel**

A non-cohesive layer of grey sand and gravel with varying amounts of silt was encountered beneath the surficial layer in Boreholes 1 to 4, and at the ground surface in Borehole 5. The surface of this deposit ranged from 66.0 m to 67.8 m, in elevation and the layer had a thickness of 2.7 m to 4.8 m. The standard penetration test (SPT) 'N' values ranged from 6 to 60 blows per 0.3 m of penetration indicating a loose to dense condition with occasional very dense zones. Gradation test results on six samples of this material indicate a gravel content between 0% and 53%, sand content between 39% and 93%, and fines content (combined silt and clay content) between 4% and 49%. The moisture content of the samples tested ranged from 5% to 20%.

#### **3.3 Glacial Till**

A layer of glacial till was reported in Borehole 5 beneath the layer of sand and gravel. The till contained a bouldery zone in its lower portion. The surface of this deposit was at elevation 65.1 m, with a thickness of 1.3 m. The SPT 'N' values were over 100 blows per 0.3 m of penetration, indicating a very dense condition. Gradation test results indicate a gravel content of 37%, sand content of 43%, silt content of 14%, and clay content of 6%.

#### **3.4 Bedrock**

Grey shale bedrock was encountered in all five boreholes with surface elevations ranging from 62.2 m to 63.8 m. The shale bedrock was described as being in a sound condition in Boreholes 1 to 3. The upper zones in Boreholes 4 and 5 were identified as being weathered with the lower elevations described as being in a strong condition. Geological mapping suggests the bedrock at this site is shale of the Billings Formation.



### **3.5 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations of 65.3 m and 66.3 m at the time of the original investigation.

## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in August 2012 for Bridge 3-304/1 with the report issued October 2013. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering staff during the week of July 14<sup>th</sup>, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

- Concrete slope paving was installed for erosion control of the abutment slopes
- Settlement and erosion of the slope paving were noted in several sections at both abutment slopes
- Side slopes of the approach embankments are heavily vegetated and erosion of the embankments was observed
- Pavement cracks were observed along the ends of both approach slabs
- Piers show no signs of settlement; standing water was observed around the base of the north pier

## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

Where insufficient earth cover is provided, polystyrene insulation may be used to enhance frost protection at the existing pile caps and footings.

#### **5.2 Seismic Considerations**

This site is classified as a Soil Profile Type I in accordance with the Canadian Highway Bridge Design Code (CHBDC) Section 4.4.6. The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the density of the sand and gravel, sand to silty sand, and till at this site, these materials are classified as “not susceptible” to liquefaction during the design earthquake event.



### **5.3 Existing Foundations**

As per the Abutment & Retaining Wall Footings Drawing (copy attached), the bridge abutments were designed to be supported on steel HP12x74 piles driven to bedrock; while the retaining walls were to be supported on steel HP12x53 piles also driven to bedrock. The available construction drawings do not indicate the design loads or grade of steel used for the piles however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the abutment foundations is 95 tons / 12BP74 pile or approximately 845 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles; while HP12x53 piles are nominally equivalent to HP310x79 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN / HP310x110 pile and 1450 kN / HP310x79 for piles driven to bedrock. OPSS 903 requires Grade 350W steel for the H-piles.

Based on the Piers and Bridge Bearings Drawing (copy attached) the existing bridge Piers 1 and 2 were designed to be supported by 4.3 m x 4.3 m, footings; while Piers 3 and 4 were designed to be supported by 4.6 m x 4.6 m, footings all founded on sound bedrock. The design loads were not indicated, however the Foundation Design Report recommended the allowable bearing value of 10 tsf or approximately 960 kPa be used for design of spread footings founded on bedrock.

## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Shallow Foundations**

Although spread footings within the overburden are possible, effective dewatering of the sand and gravel strata would be required. Given the anticipated relatively shallow depth to the shale bedrock, and the higher geotechnical resistance offered by the bedrock, spread footings within the overburden are not recommended. In addition, footings for potential widenings should match the elevation of the existing foundation element, i.e. be founded on bedrock.

The factored vertical geotechnical resistance of 1500 kPa at ULS may be used for the preliminary design of shallow foundations, founded on or in sound shale bedrock. The SLS condition will not govern for footings in or on the bedrock. The design of any new foundations on bedrock would need to consider interactions with the existing foundations and potential for undermining of the existing structural elements.



Resistance to lateral forces and sliding resistance between concrete and underlying materials should be evaluated using an unfactored coefficient of friction of 0.50 for cast-in-place concrete and 0.45 for pre-cast concrete on sound shale bedrock.

Based on current geological mapping, the shale present at this site may be from the Billings Formation which is susceptible to heaving if allowed to weather in the presence of oxygen. The general mechanism is that oxidation of pyrite within the shale produces sulfuric acid, which in turn reacts with calcite in the shale to form gypsum crystals, which occupy a larger volume than the original materials. A by-product of this chain of reactions also tends to increase sulphate levels which can attack buried concrete structures.

The potentially detrimental effects of shale heaving can be avoided by preventing exposure of the shale to oxygen both during construction and long term. This is typically achieved by limiting exposure of the shale to no more than one day prior to covering with a protective layer such as shotcrete or a concrete mud slab. Sulphate resistant cement should be used in such applications.

## 6.2 Deep Foundations – Caissons Socketed in to Bedrock

For preliminary design purposes, the factored vertical geotechnical resistance for varying diameters of caissons is provided in the Table A. The values provided have been calculated based on the following assumptions:

- Caisson capacity is calculated based on shaft resistance of the rock socket
- Rock-quality designation (RQD) greater than 25%
- Minimum unconfined compressive strength (UCS) of the bedrock of 35 MPa. Note we have assumed a conservative value as there is no site specific test results available. Regional information suggests the UCS may be in excess of 40 MPa. The caisson capacities should be reassessed when additional borehole data and bedrock strength data is available for detailed design.
- Note that the upper portions of the shale bedrock are likely to be weathered. The caisson may have to penetrate several metres of weathered rock above the actual rock socket.
- Caisson concrete compressive strength of 35 MPa.

**Table A:** Geotechnical Axial Capacity of Rock Socketed Caisson

Caisson Diameter (m)	Socket Length (m)	Factored Axial Geotechnical Resistance at ULS (kN)
1.2	2	8,000
1.5		10,000
1.8		12,000
2.0		13,300
1.2	3	12,000
1.5		15,000



Caisson Diameter (m)	Socket Length (m)	Factored Axial Geotechnical Resistance at ULS (kN)
1.8		18,000
2.0		20,000

The factored ULS values provided in Table A includes a resistance factor of 0.4 as recommended in Table 6.1 of the CHBDC. The SLS condition will not govern for caissons socketed into bedrock. The geotechnical resistance could be increased by increasing the socket length within the bedrock.

### 6.3 Deep Foundations – Piles

Although the depth to bedrock is too shallow at the existing pier locations for piles to be practical, driven piles are considered to be suitable for the support of abutment foundations perched within the approach embankments. It should be noted that the bedrock surface elevation ranges from 62.2 m to 63.8 m.

#### 6.3.1 Axial Resistance

Steel piles (Grade 350 W steel) end-bearing on sound bedrock at this site may be designed on the basis of the following factored, vertical, geotechnical resistances at ULS:

- 1,450 kN per HP310x79 pile; and
- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

#### 6.3.2 Pile Tips

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

#### 6.3.3 Integral Abutment Considerations

As per the Abutment & Retaining Wall Footings Drawing the existing abutments and retaining walls are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### 6.3.4 Pile Spacing

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### 6.3.5 Downdrag

The overburden at this site is relatively thin and incompressible. Downdrag on existing and new piles is not considered a design issue at this site.

### 6.3.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formula for cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where:

$k_h$  = coefficient of horizontal subgrade reaction

$n_h$  = coefficient related to soil density given in Table B

$z$  = depth of pile embedment (m)

$B$  = pile width perpendicular to load direction (m)

**Table B:**  $n_h$  Values For Cohesionless Soils

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 67.0	Embankment Fill	3,000
Between 67 and 64	Silty Sand	2,000
Below 64	Glacial Till	3,500

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table C below. Intermediate values may be obtained by linear interpolation.

**Table C:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.3.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table D

$\sigma'_v$  = effective vertical stress (kPa)

**Table D:**  $\beta$  Values for Driven Piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 67.0	Embankment Fill	20	0.4
Between 67 and 64	Silty Sand	19	0.4
Below 64	Glacial Till	20	0.5

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.



## 6.4 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table E are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table E:** Static Lateral Earth Pressure Coefficients

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Glacial Till	OPSS Granular B Type I / Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	20 / 19
Angle of Internal Friction, $\phi$	35°	32°	30°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.47	0.50
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.31	0.33

## 6.5 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table F.

**Table F:** Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Glacial Till	OPSS Granular B Type I / Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$32^\circ$	$30^\circ$
<b>Yielding Wall</b>			
$K_{AE}$	0.33	0.37	0.40
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36
<b>Non-Yielding Wall</b>			
$K_{AE}$	0.55	0.62	0.66
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil ( $\text{kN/m}^3$ )

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive



pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors provided in Table F are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## **6.6 Approach Embankments**

The original MTO geotechnical report states that the maximum height of approach fills to be on the order of 9.6 m with an embankment slope of 2H:1V. Backup correspondence from 1972 indicates that excavated shale and glacial till materials were to be used as embankment fill with a higher than typical unit weight recommendation (23.5 kN/m<sup>3</sup>). In addition, the 1972 Report indicated that the approach fills will settle approximately 50 mm in elastic compression. The embankment foundation is expected to be stable and no long term settlement problems are expected unless the fills are reconfigured.

For preliminary design purposes new fills greater than 8 m in height should be sloped at 2H:1V and should include a mid-height bench at least 3m in width.

## **6.7 Erosion Control**

Active erosion below the slope paving was noted at both abutments locations at this site.

The slope paving should be repaired where required and maintained and drainage measures should be enhanced beneath the abutments to prevent further erosion below the slope paving.

## **6.8 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native overburden soils reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.



## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

Should the proposed works result in excavations in or on the bedrock for deep foundations, the foundation investigation should also include bedrock coring, sampling and compressive strength testing to determine the condition and strength properties of the bedrock. Bedrock samples should also be submitted for chemical testing to determine the pyritic heave potential.

During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one.

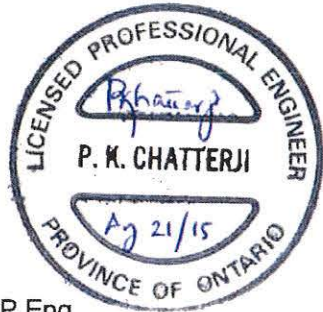
## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.

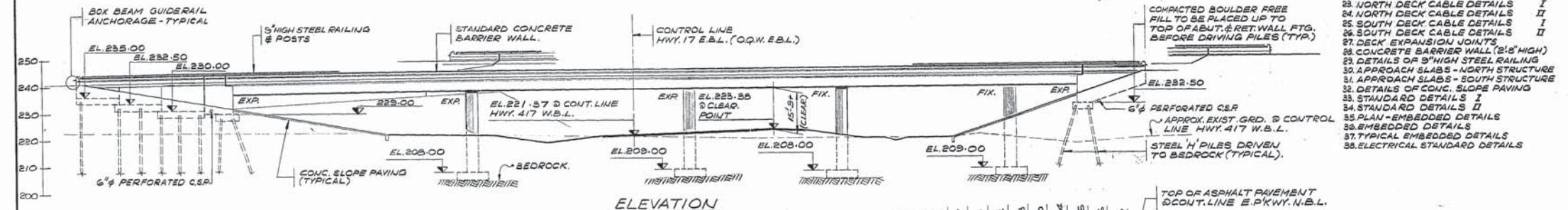
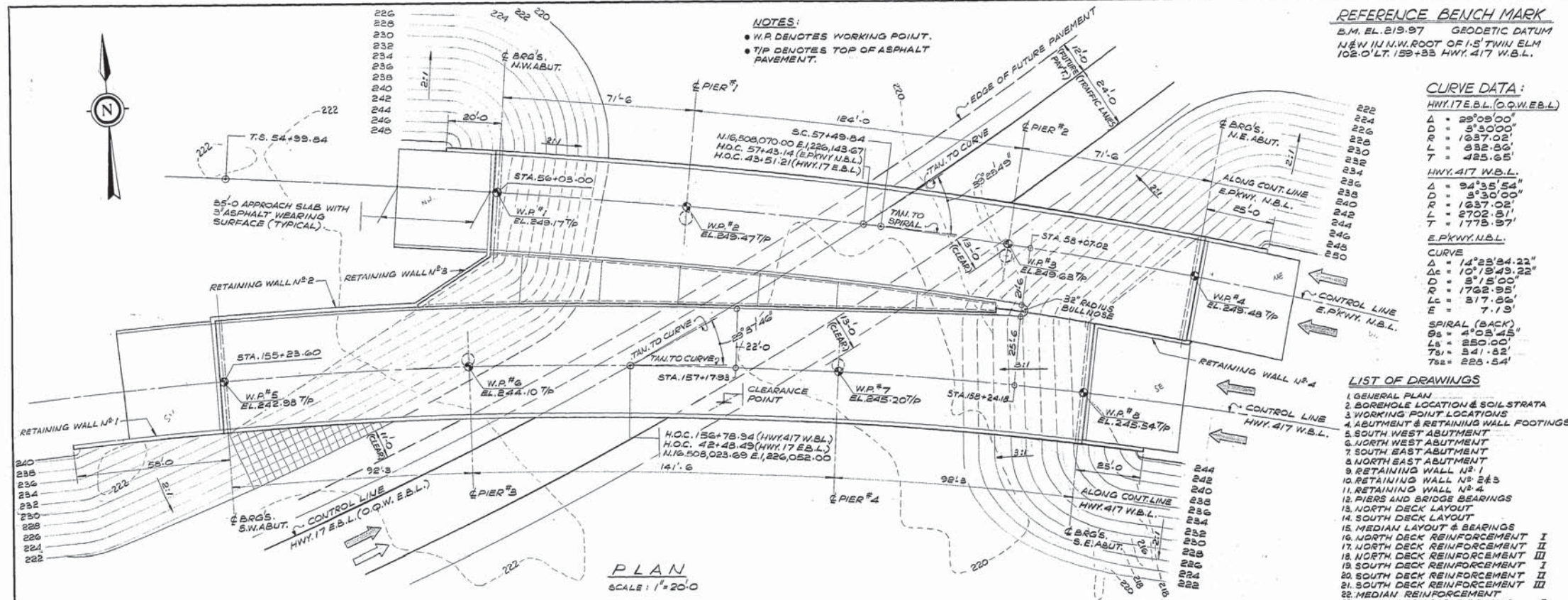


Paul Carnaffan, M. Eng., P. Eng.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, Ph.D., P. Eng.  
Review Principal, Designated MTO Contact

Attachments

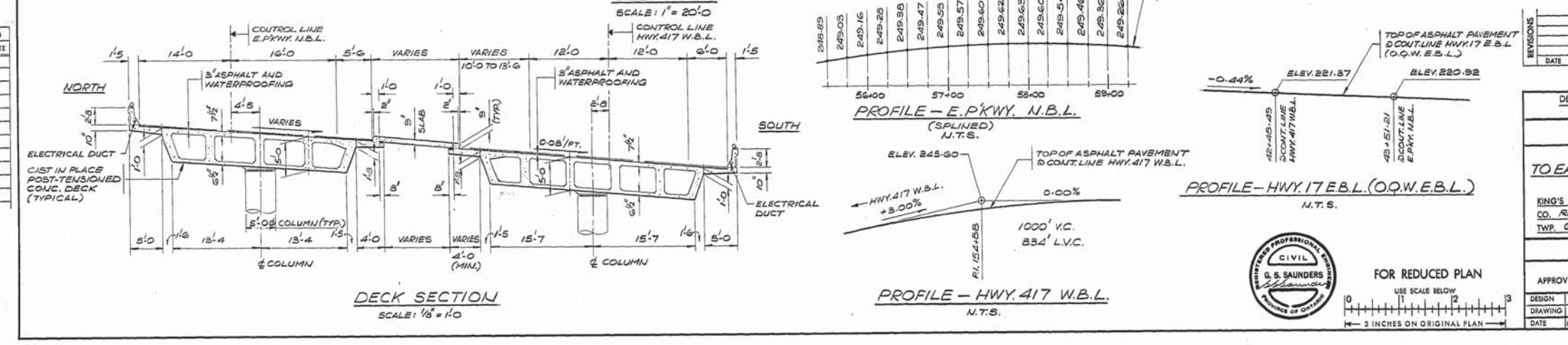


GENERAL NOTES

CLASS OF CONCRETE:  
DECK, MEDIAN, PIER COLUMNS I 5,000 P.S.I.  
BARRIER WALLS II 5,000 P.S.I.  
APPROACH SLABS III 5,000 P.S.I.  
REMAINER I 5,000 P.S.I.

CLEAR COVER ON REINFORCING STEEL:  
FOOTINGS, ABUTMENTS, RETAINING WALLS & PIERS I 4"  
DECK: TOP SLAB - TOP 1 1/2", BOT. 1"  
BOT. SLAB - TOP & BOT. 1"  
WEBB - 1 1/2"  
MEDIAN: TOP - 1 1/2" BOT. - 1"  
APPROACH SLABS: - 2" BARRIER WALLS: - 1 1/2"

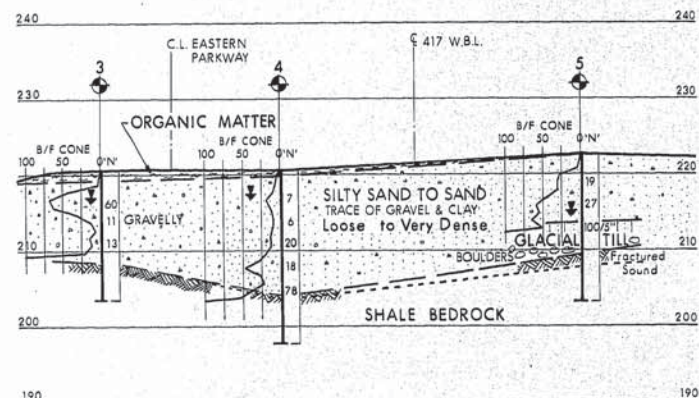
CONSTRUCTION NOTES:  
THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8".  
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED, STRESSED AND CROUTED.



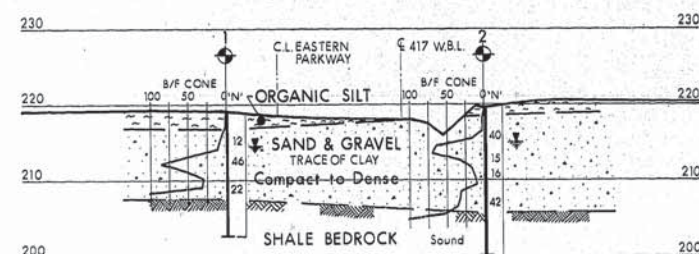
REVISIONS	
DATE	DESCRIPTION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO			
DeLeuw, Cather ENGINEERS & PLANNERS - OTTAWA			
WESTBOUND OVERPASS TO EASTERN PKWAY & TO O.Q.W. W.B.L. BRIDGE N° 7			
KING'S HIGHWAY No. 417		DIST. No. 3	
CO. REG. MUNICIPALITY OF OTTAWA - CARLETON			
TWP. GLOUCESTER		LOT 24 CON. II	
GENERAL PLAN			
APPROVED		SITE No. 3-304A W.P. No. 73-68-02	
DESIGN G.S.S. CHECK L.D.H.		CONTRACT No. 73-192	
DRAWING R.A.P. CHECK G.S.S.		DRAWING No. 3-304A-1	
DATE JUNE '73		LOADING WS20.44	

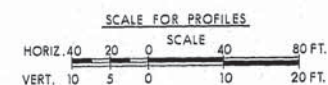
57079 TWP# 56-304A-1-A



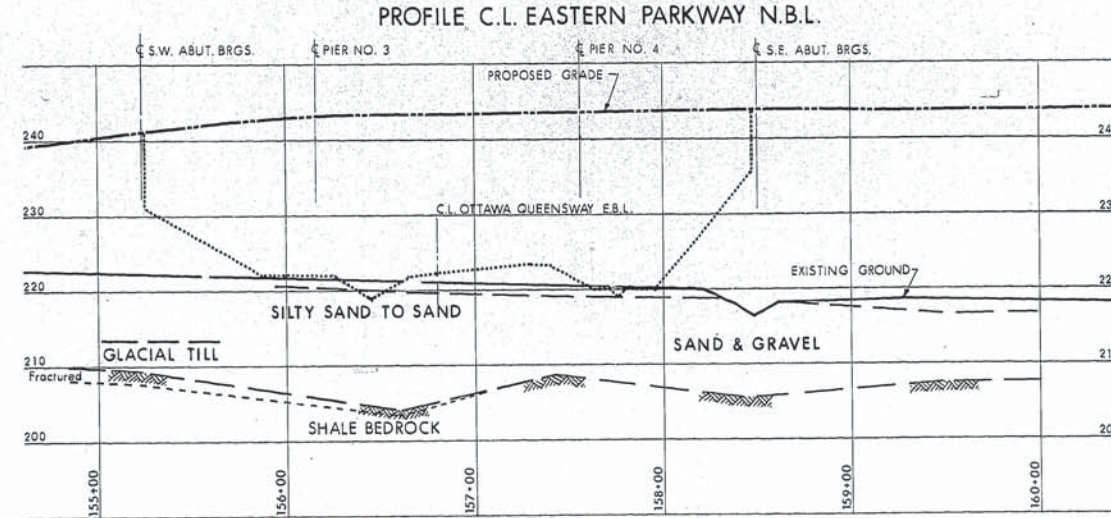
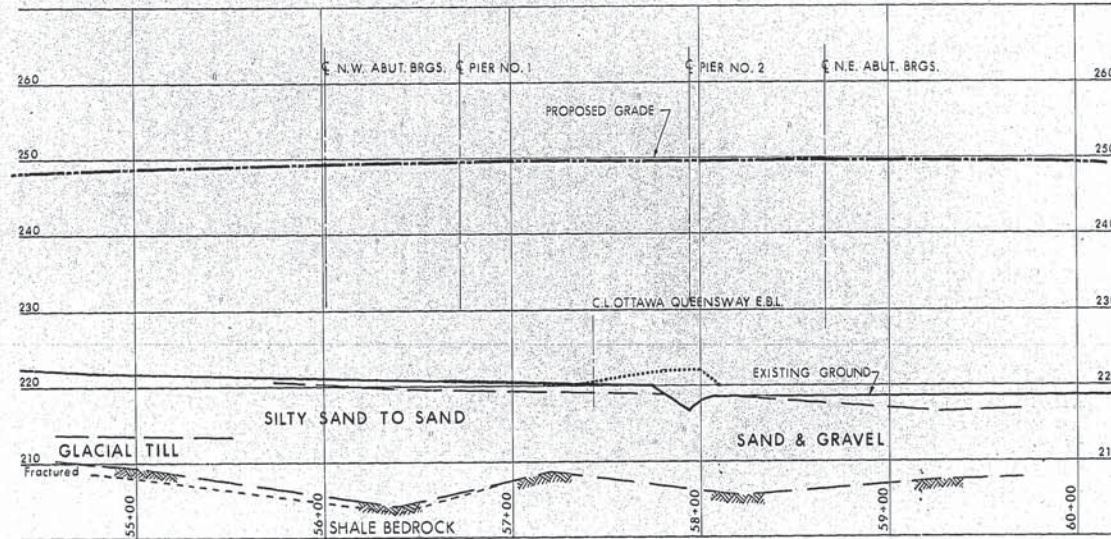
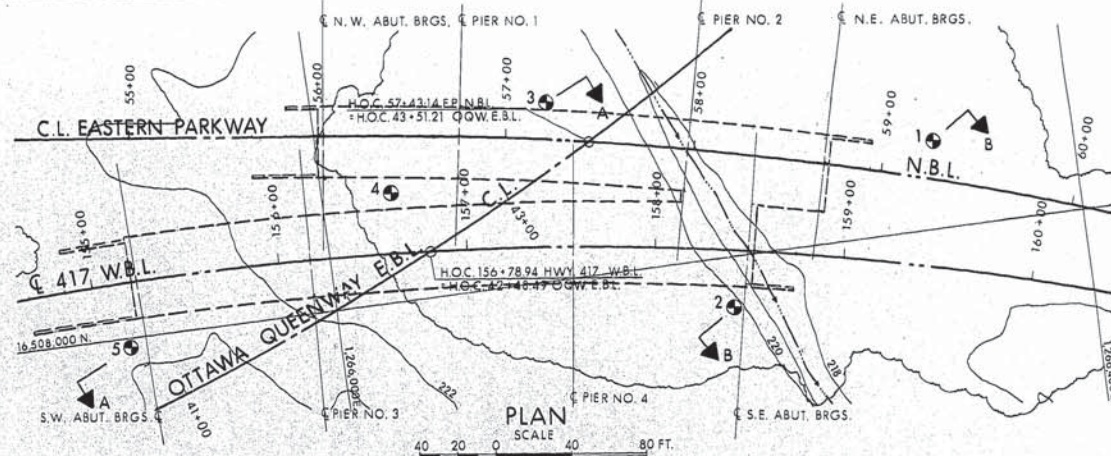
SECTION A-A  
HORIZ. 40 20 0 SCALE 80 FT.  
VERT. 10 5 0 10 20 FT.



SECTION B-B  
HORIZ. 40 20 0 SCALE 80 FT.  
VERT. 10 5 0 10 20 FT.



NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the Ottawa District Office.



PROFILE 417 W.B.L. REF. N° E-5241-1, E-5437-1, & B-56-28



LEGEND				
●	Bore Hole			
⊕	Cone Penetration Test			
⊗	Bore Hole & Cone Test			
↓	Water Levels established at time of field investigation, July & August 1972.			
NO.	ELEVATION	CO-ORDINATES		
		NORTH	EAST	
1	219.1	16,508,046	1,226,328	
2	219.7	507,973	226,210	
3	220.7	508,094	226,126	
4	220.5	508,058	226,037	
5	222.6	507,995	225,889	

NOTE: The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO  
DESIGN SERVICES BRANCH-FOUNDATIONS OFFICE

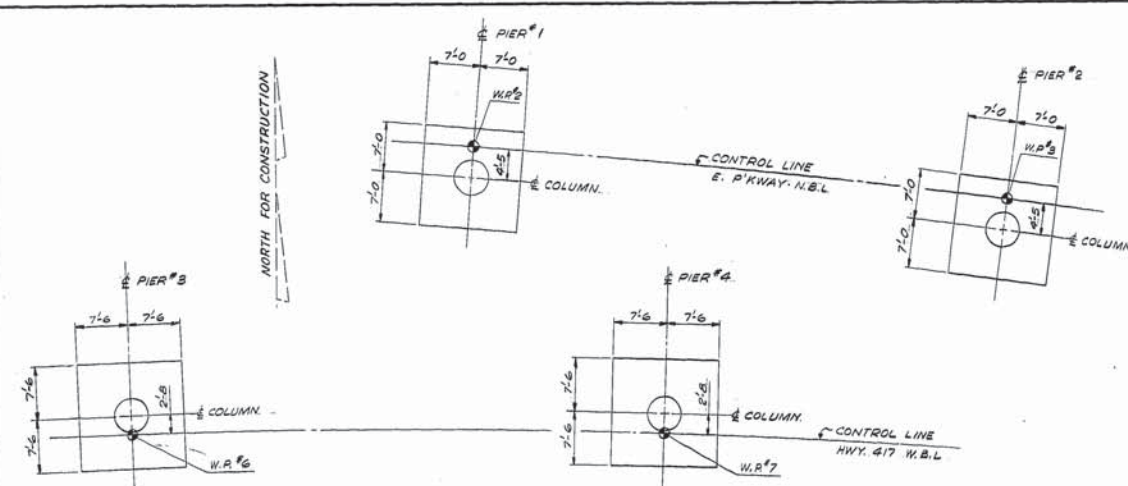
BRIDGE No 7  
EASTERN PARKWAY N.B. & HWY. 417 W.B. OVER OTTAWA QW.E.B.  
HIGHWAY NO. 417 DIST. NO. 9  
CO. REGIONAL MUNICIPALITY OF OTTAWA-CARLETON  
TWP. GLOUCESTER LOT 24 CON. II

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. S.A. CHECKED <i>[initials]</i>	W.P. NO. 13-68-02	DRAWING NO.
DRAWN J.I.G. CHECKED <i>[initials]</i>	W.O. NO. 72-11083	72-11083A
DATE NOV. 21, 1972	SITE NO. 3-304A	BRIDGE DRAWING NO.
APPROVED <i>[signature]</i>	CONT. NO. 73-192	3-304A-2

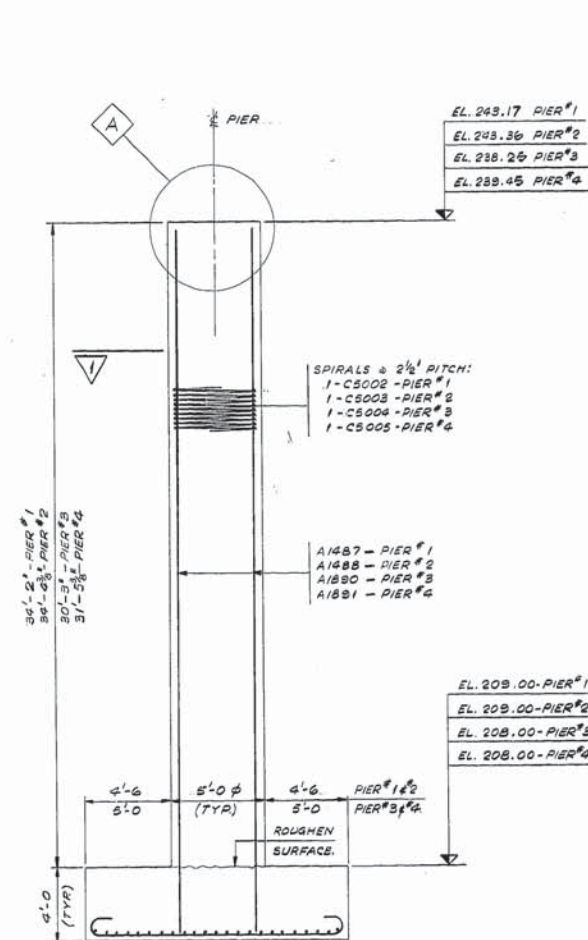
57099

TWP-56-304A-2-A



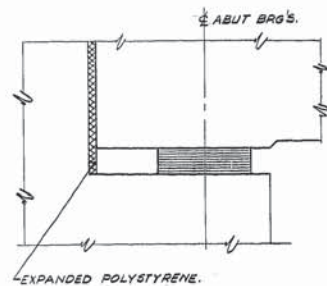
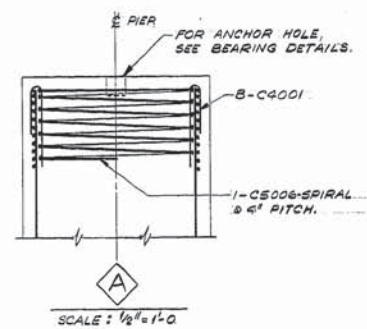
PIER FOOTING LAYOUT

N.T.S.



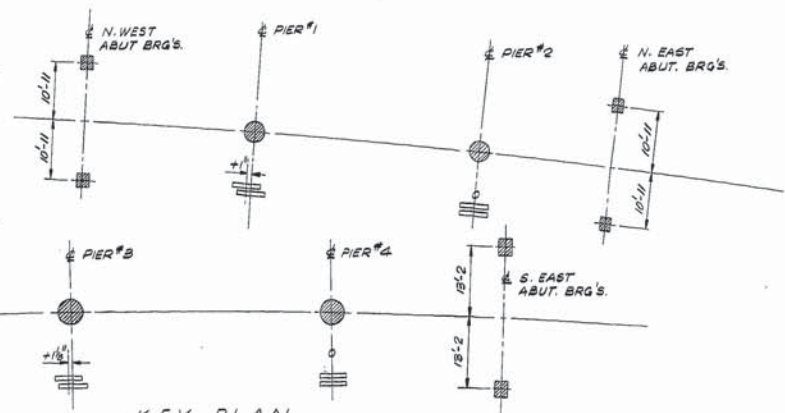
ELEVATION-TYPICAL

SCALE: 1/4" = 1'-0"



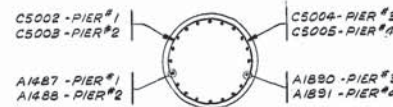
TYP. ELEVATION OF BEARINGS AT ABUTMENTS

SCALE: 3/4" = 1'-0"



KEY PLAN

N.T.S.



PLAN OF PIER FOOTING-TYPICAL

SCALE: 1/4" = 1'-0"

STRUCTURE DESIGN DATA FOR PIER BEARINGS.

LOCATION	BEARING PART NO.	NO. REQ'D	TOTAL MOVEMENT	DIM. 'X'	MAX. LOAD	MIN. LOAD	MAX. SIDE LOAD
PIER #1	RF300/20	1	+3/8", -1/8"	5.36'	1,918 K.	1,587 K.	53 K.
PIER #2	R850	1	-	4.12'	1,861 K.	1,535 K.	51 K.
PIER #3	RF1200/55	1	+3/8", -1/8"	5.86'	2,524 K.	2,156 K.	56 K.
PIER #4	R1200	1	-	4.65'	2,536 K.	2,165 K.	108 K.

PIER BEARINGS TO BE ANDRE ROTA AND ROTAFLON BRIDGE BEARINGS OR EQUAL. TOTAL MOVEMENT INDICATES TOP PLATE SET AWAY FROM PIER #2 OR PIER #4.

STRUCTURE DESIGN DATA FOR ABUT. BEARINGS.

LOCATION	BEARING PART NO.	NO. REQ'D	SIZE	DEAD LOAD	DL + L.L.	MAX. MOVEMENT	MAX. ALLOW. SHEAR	MAX. ALLOW. DEF'L UNDER DL + L.L.
N.W. ABUT.	585/50/36	2	24" x 16" x 5 3/8"	219 K.	307 K.	± 2"	9.9 K./IN.	0.15"
N.E. ABUT.	585/50/33	2	24" x 16" x 3 3/8"	204 K.	292 K.	± 3/8"	17.9 K./IN.	0.10"
S.W. ABUT.	585/54/10	2	24" x 20" x 7 3/8"	347 K.	456 K.	± 2 3/4"	9.6 K./IN.	0.15"
S.E. ABUT.	585/54/44	2	24" x 20" x 3 3/8"	362 K.	471 K.	± 1"	21.3 K./IN.	0.10"

ABUT. BEARINGS TO BE ANDRE BRIDGE BEARINGS OR EQUAL.

## NOTES.

1. PIER FOOTINGS TO BE FOUNDED ON SOUND BEDROCK.
2. FOR MEDIAN SLAB BEARINGS, SEE SHEET NO. 15.

REVISIONS	DATE	BY	DESCRIPTION

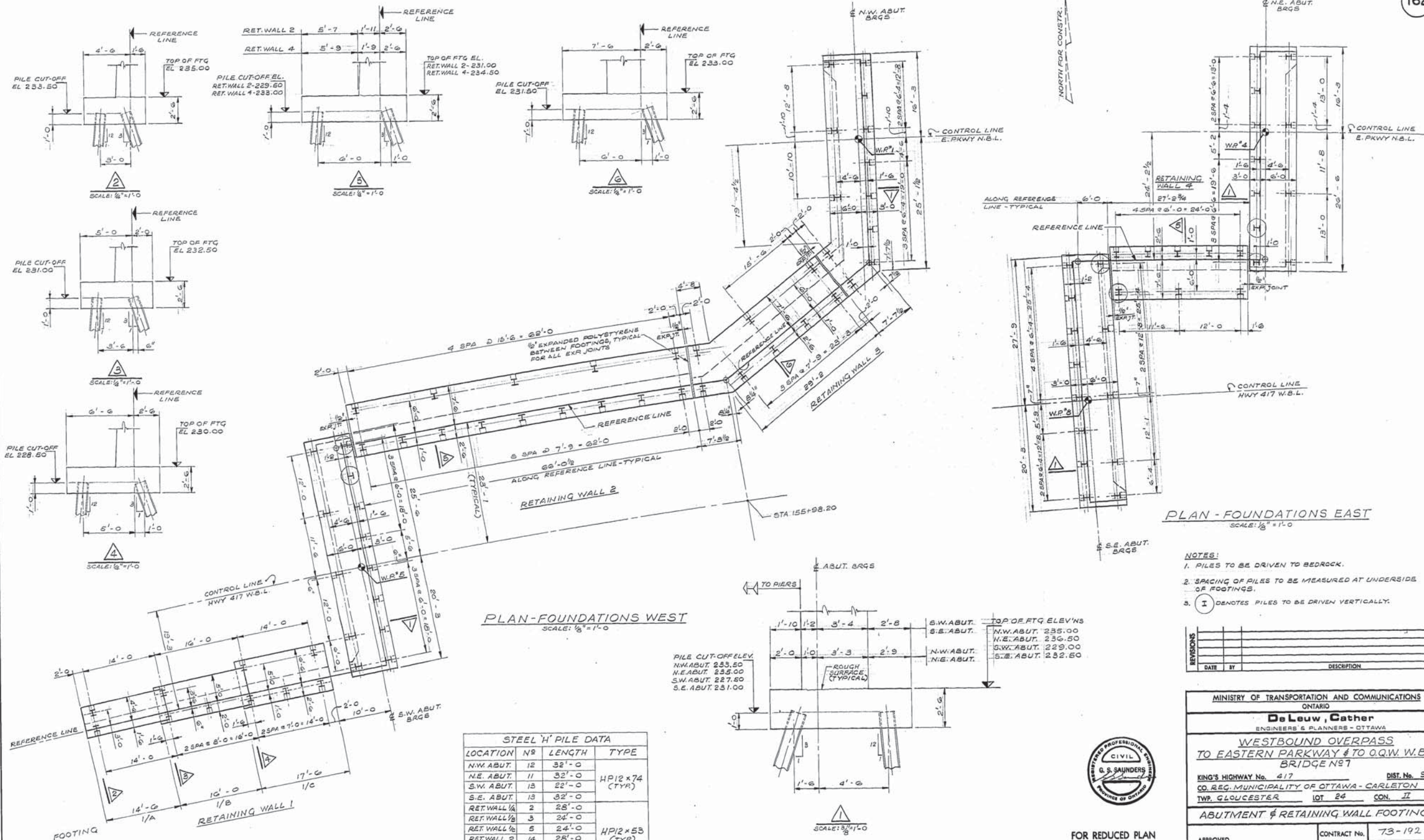
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO			
De Leuw, Cather ENGINEERS & PLANNERS - OTTAWA			
WESTBOUND OVERPASS TO EASTERN PKWAY TO O.Q.W. W.B.L. BRIDGE NO. 7			
KING'S HIGHWAY No. 417		DIST. No. 9	
CO. REG. MUNICIPALITY OF OTTAWA-CARLETON		W.P. No. 13-68-02	
TWP. GLOUCESTER		LOT 24 CON. II	
PIERS AND BRIDGE BEARINGS.			
APPROVED _____		CONTRACT No. 73-192	
DESIGN A.E. CHECK L.D.H.		W.P. No. 13-68-02	
DRAWING S.C. CHECK L.D.H.		SITE No. 3-304A SHEET 12	
DATE JULY 78		LOADING HS20-44	



FOR REDUCED PLAN

USE SCALE BELOW  
10 1 12 13  
3 INCHES ON ORIGINAL PLAN

57082 TWP# 56-304A12-A



PRINT RECORD		
No.	FOR	DATE



**APPENDIX 4**  
**SITE 3-314**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Fred J. Griffiths, Ph.D., P.Eng.  
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 CYRVILLE ROAD UNDERPASS (SITE 3-314) GWP 4074-11-00 GEOCRES 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing Highway 417 Underpass of Cyrville Road in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4043-11-01
- Site No. 3-314
- GEOCRES No. 31G5-114
- Construction Contract 73-192
- Historic W.P. 13-68-04

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa in the Township of Gloucester approximately 600 m north of the Highway 417 / Innes Road Interchange. The bridge carries the Cyrville Road north and southbound (two lanes plus concrete sidewalks) over both the east and westbound lanes of Highway 417. Based on the historic General Layout Drawing (copy attached), the bridge is a 12.8 m wide, 102.4 m long, two-span structure with a post-tensioned cast-in-place concrete deck. The bridge abutments are supported by steel HP12x74 piles driven to bedrock, while the center pier is supported by spread footings founded on bedrock.

The natural terrain in the vicinity of the bridge is generally flat with elevations ranging from 64 to 68 m. The design drawings show that the approach fills were to be constructed by placing as much as 6 m of fill at a 2H:1V (Horizontal:Vertical) slope.



### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-114 dated November 1972. The original investigation was conducted in July 1971 and consisted of six sampled boreholes, four of which were accompanied by dynamic cone penetration tests. Two additional cone-only tests were completed on site. Drawing No. 72-11109A (copy attached) illustrates the locations of the bridge and the investigation boreholes, as well as the soil strata plot for the original investigation. The stratigraphy in the area of the bridge is generally characterized by a silty sand to sandy silt layer, overlaying glacial till, all of which are underlain by shale bedrock.

#### **3.1 Silty Sand to Sandy Silt**

The top of the silty sand to sandy silt layer ranged from 67.2 to 67.9 m in elevation, and the layer had a thickness of 1.8 to 2.4 m. The standard penetration test (STP) 'N' values varied greatly for this deposit ranging from 12 to 66 blows per 0.3 m of penetration; indicating a compact to very dense condition. Gradation test results on samples of this material indicate a gravel content between 0% and 4%, sand content between 22% and 75% and a fines content (combined silt and clay content) ranging from 25% to 78%.

#### **3.2 Glacial Till**

Underlying the silty sand to sandy silt layer is a glacial till deposit. The surface of this deposit ranged from 65.2 to 65.8 m in elevation, and the layer had a thickness of 0.8 to 1.1 m. This layer is described as cohesive and consists of a matrix of clayey silt which binds the sand and gravel particles. The STP 'N' values varied greatly for this deposit ranging from 24 to greater than 100 blows per 0.3 m of penetration; indicating compact to very dense condition. Gradation test results on samples of this material indicate a gravel content between 21% and 29%, sand content between 47% and 49%, and a fines content ranging from 22% to 32%.

#### **3.3 Bedrock**

Beneath the glacial till layer, grey shale bedrock was encountered with surface elevations ranging from 64.1 to 65.0 m. The upper 600 mm of the shale was generally fractured and weathered. At greater depths the shale was described as being in a sound condition. Geological mapping suggests the bedrock at this site is shale of the Billings Formation.

#### **3.4 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 65.8 m to 67.3 m.

### **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in July 2012 for Bridge 3-314 with the report issued November 2013. Condition data outlined in the report for the bridge structure ranged



from poor to good but typically the bridge was rated in fair condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering staff during the week of July 14<sup>th</sup>, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

- Concrete slope paving was installed for erosion control of the abutment slopes
- Signs of shifting and erosion below the slope paving was observed at the east abutment
- Side slopes of the embankment are highly vegetated; no signs of erosion were noted
- Cracks in the pavement were observed at both ends of the approach slabs
- Slight dip was observed in the embankment before the bridge to the east
- No signs of settlement of the middle pier were observed

## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

Polystyrene should be incorporated at both abutments to enhance frost protection of the existing pile caps. The frost cover of the pier footing should also be assessed.

#### **5.2 Seismic Considerations**

This site is classified as a Soil Profile Type I in accordance with the Canadian Highway Bridge Design Code (CHBDC) Section 4.4.6. The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the density of the silty sand/sandy silt and the till at this site, these materials are classified as “not susceptible” to liquefaction during an earthquake event.

#### **5.3 Existing Foundations**

As per the Footings & Pier Column Drawing (copy attached), the bridge abutments are supported by steel HP12x74 piles driven to bedrock. The available construction drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the abutment foundations is 95 tons / 12BP74 pile or approximately 845 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN /



HP310x110 pile driven to bedrock. The SLS condition will not govern for piles end-bearing in or on the bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

The existing bridge pier is supported by a 5.3 m x 5.3 m, footing. Construction notes on the Footings & Pier Column Drawing indicate that the pier footing was to be founded on sound bedrock. The design loads were not provided, however the Foundation Design Report recommended the allowable bearing value of 10 tsf or approximately 960 kPa be used for design of spread footings founded on bedrock.

## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Spread Footings**

Although spread footings within the overburden are possible, effective dewatering of the sand and gravel strata would be required. Given the anticipated relatively shallow depth to the shale bedrock, and the higher geotechnical resistance offered by the bedrock, spread footings within the overburden are not recommended. In addition, footings for potential widenings should match the elevation of the existing foundation element, i.e. be founded on bedrock.

The factored vertical geotechnical resistance of 1500 kPa at ULS may be used for the preliminary design of shallow foundations, founded on or in sound shale bedrock. The SLS condition will not govern for footings in or on the bedrock. The design of any new foundations on bedrock would need to consider interactions with the existing foundations and potential for undermining of the existing structural elements.

Resistance to lateral forces and sliding resistance between concrete and underlying materials should be evaluated using an unfactored coefficient of friction of 0.50 for cast-in-place concrete and 0.45 for pre-cast concrete on sound shale bedrock.

Based on current geological mapping, the shale present at this site may be from the Billings Formation which is susceptible to heaving if allowed to weather in the presence of oxygen. The general mechanism is that oxidation of pyrite within the shale produces sulfuric acid, which in turn reacts with calcite in the shale to form gypsum crystals, which occupy a larger volume than the original materials. A by-product of this chain of reactions also tends to increase sulphate levels which can attack buried concrete structures.

The potentially detrimental effects of shale heaving can be avoided by preventing exposure of the shale to oxygen both during construction and long term. This is typically achieved by limiting exposure of the shale to no more than one day prior to covering with a protective layer such as



shotcrete or a concrete mud slab. Sulphate resistant cement should be used in such applications.

## **6.2 Deep Foundations – Piles**

Although the depth to bedrock is too shallow at the existing pier location for piles to be practical, driven piles are considered to be suitable for the support of abutment foundations perched within the approach embankments. It should be noted that the bedrock surface elevation ranges from 64.1 m to 65.0 m.

### **6.2.1 Axial Resistance**

Steel piles (Grade 350 W steel) end-bearing on sound shale bedrock at this site may be designed on the basis of the following factored vertical geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

### **6.2.2 Pile Tips**

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### **6.2.3 Integral Abutment Considerations**

As per the Footings and Pier Column Drawing the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### **6.2.4 Pile Spacing**

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### **6.2.5 Downdrag**

The overburden at this site is relatively thin and incompressible. Downdrag on existing and new piles is not considered a design issue at this site.

## 6.2.7 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formula for cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where:

- $k_h$  = coefficient of horizontal subgrade reaction
- $n_h$  = coefficient related to soil density given in Table A
- $z$  = depth of pile embedment (m)
- $B$  = pile width perpendicular to load direction (m)

**Table A:**  $n_h$  Values for Cohesionless Soils

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 67.0	Embankment Fill	3,000
Between 67.0 and 65	Sandy Silt to Silty Sand	2,000
Below 65	Glacial Till	3,500

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

## 6.2.9 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

$C$  = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)

**Table C:**  $\beta$  Values for Driven Piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 67.0	Embankment Fill	20	0.4
Between 67.0 and 65	Sandy Silt to Silty Sand	19	0.4
Below 65	Glacial Till	20	0.5

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

## 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$



where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D: Static Lateral Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Glacial Till	OPSS Granular B Type I / Sandy Silt & Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	20 / 19
Angle of Internal Friction, $\phi$	35°	32°	30°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.47	0.5
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.31	0.33

#### 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.



The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E:** Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & OPSS Granular B Type II	Native Glacial Till	OPSS Granular B Type I / Sandy Silt & Silty Sand
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	21	19	20 / 19
Angle of Internal Friction, $\phi$	35°	32°	30°
<b>Yielding Wall</b>			
$K_{AE}$	0.33	0.37	0.40
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36
<b>Non-Yielding Wall</b>			
$K_{AE}$	0.55	0.62	0.66
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth, d (kPa)

d = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

H = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors provided in Table E are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.



## **6.5 Approach Embankments**

Based on the original Foundation Design Report, the embankment soils consist of compact fill to a maximum height of 6 m above the original ground surface constructed with a 2H:1V embankment slope. The embankment foundation is expected to be stable and no long term settlement problems are expected unless the fills are reconfigured.

## **6.6 Erosion Control**

Active erosion below the slope paving was noted at the east abutment location at this site.

The slope paving should be repaired where required and maintained and drainage measures should be enhanced beneath the abutments to prevent further erosion below the slope paving.

## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native overburden soils reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one.



Should the proposed works result in excavations to bedrock, samples of the bedrock should be acquired during the investigation and submitted for chemical testing to determine the pyritic heave potential.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.

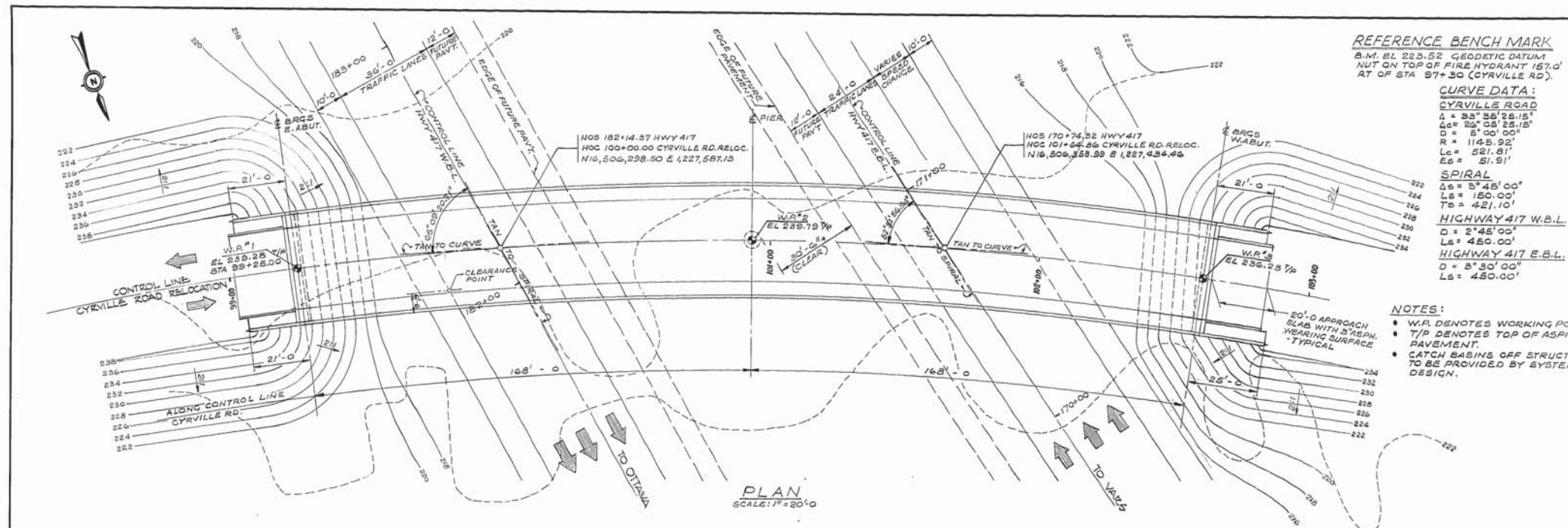


Fred J. Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

Attachments



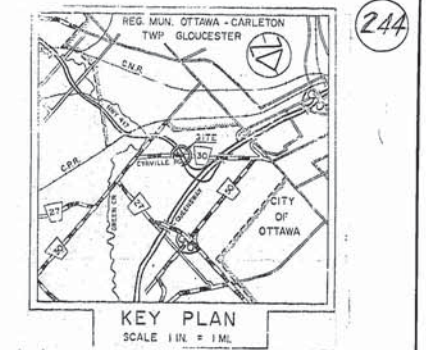
**REFERENCE BENCH MARK**  
S.M. EL 225.52 GEODETIC DATUM  
NUT ON TOP OF FIRE HYDRANT 157.0'  
RT OF STA 97+30 (CYRVILLE RD.)

**CURVE DATA:**  
CYRVILLE ROAD  
Δ = 35° 35' 25.15"  
Δc = 25° 05' 25.15"  
D = 5° 00' 00"  
R = 1145.92'  
Lc = 521.51'  
Es = 51.51'

**SPIRAL**  
Δs = 5° 45' 00"  
Ls = 150.00'  
Ts = 421.10'

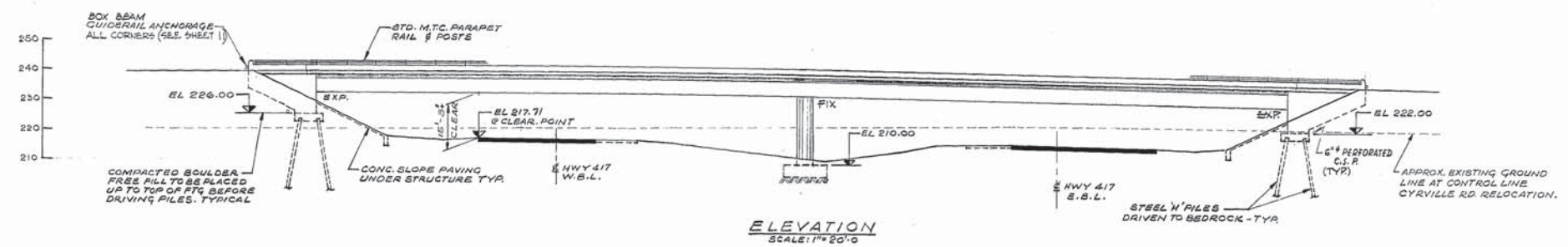
**HIGHWAY 417 W.B.L.**  
D = 2° 45' 00"  
Ls = 450.00'

**HIGHWAY 417 E.B.L.**  
D = 5° 30' 00"  
Ls = 450.00'



- NOTES:**
- W.P. DENOTES WORKING POINT
  - T/P DENOTES TOP OF ASPHALT PAVEMENT.
  - CATCH BASINS OFF STRUCTURE TO BE PROVIDED BY SYSTEMS DESIGN.

- LIST OF DRAWINGS**
- |          |                                   |
|----------|-----------------------------------|
| 3-314-1  | GENERAL LAYOUT                    |
| 2        | BORE HOLE LOCATIONS & SOIL STRATA |
| 3        | FOOTINGS & PIER COLUMN            |
| 4        | EAST ABUTMENT                     |
| 5        | WEST ABUTMENT                     |
| 6        | DECK DETAILS & BEARINGS           |
| 7        | DECK REINFORCEMENT I              |
| 8        | DECK REINFORCEMENT II             |
| 9        | LONGITUDINAL CABLES               |
| 10       | TRANSVERSE CABLES                 |
| 11       | PARAPET WALL DETAILS              |
| 12       | STANDARD STEEL PARAPET RAIL       |
| 13       | 20 FT. APPROACH SLABS             |
| 14       | DETAILS OF CONC. SLOPE PAVING     |
| 15       | STANDARD DETAILS I                |
| 16       | STANDARD DETAILS II               |
| 17       | PLAN-EMBEDDED DETAILS             |
| 3-314-1B | EMBEDDED DETAILS                  |

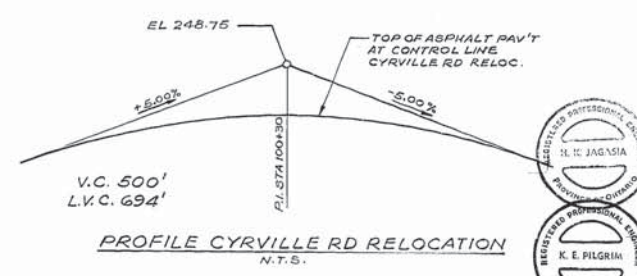
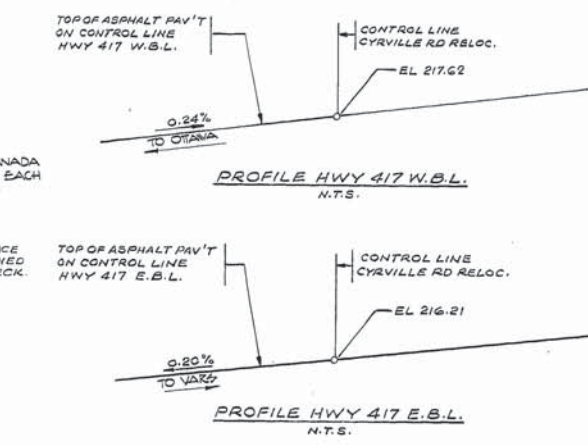
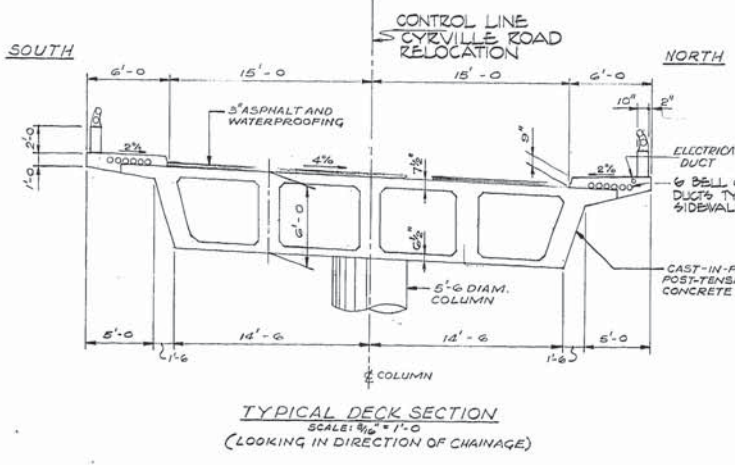


**NOTES:**

**CLASS OF CONCRETE**  
DECK, SIDEWALKS, PARAPET WALLS, 5000 P.S.I.  
PIER COLUMN 3000 P.S.I.  
REMAINDER 3000 P.S.I.

**CLEAR COVER ON REINF. STEEL**  
FOOTINGS & ABUTMENTS - 3"  
SIDEWALKS, APPROACH SLABS & PIER COLUMN - 2"  
DECK: TOP SLAB - TOP 1 1/2" BOTTOM - 1"  
SUBT. TOP & BOT. - 1"  
WEBS - 1/2", PARAPET WALLS - 1 1/2"

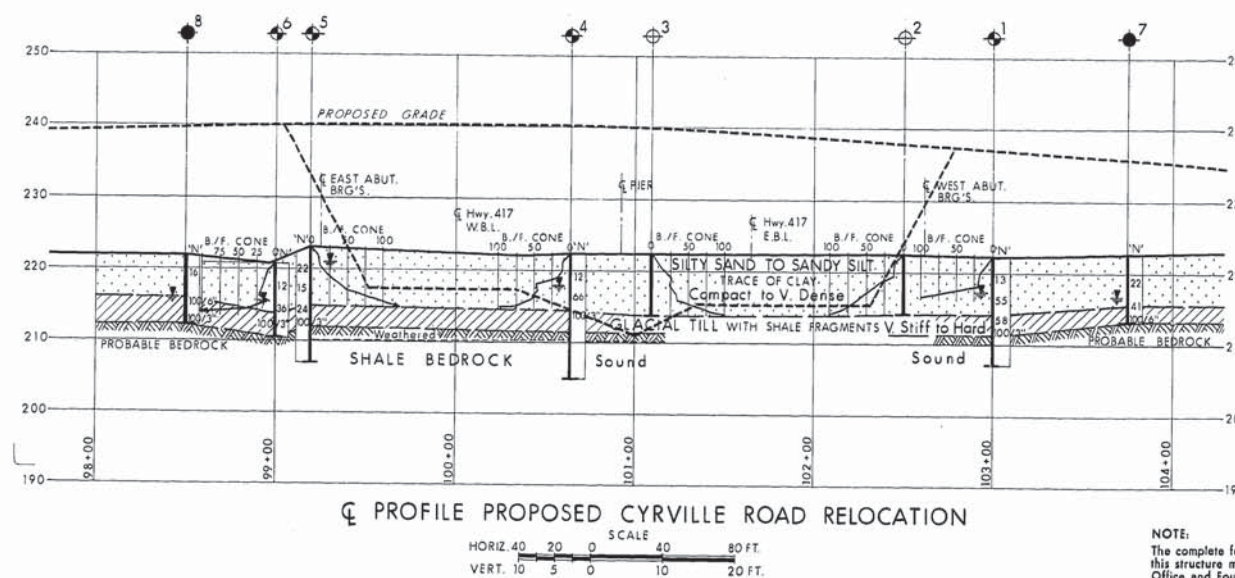
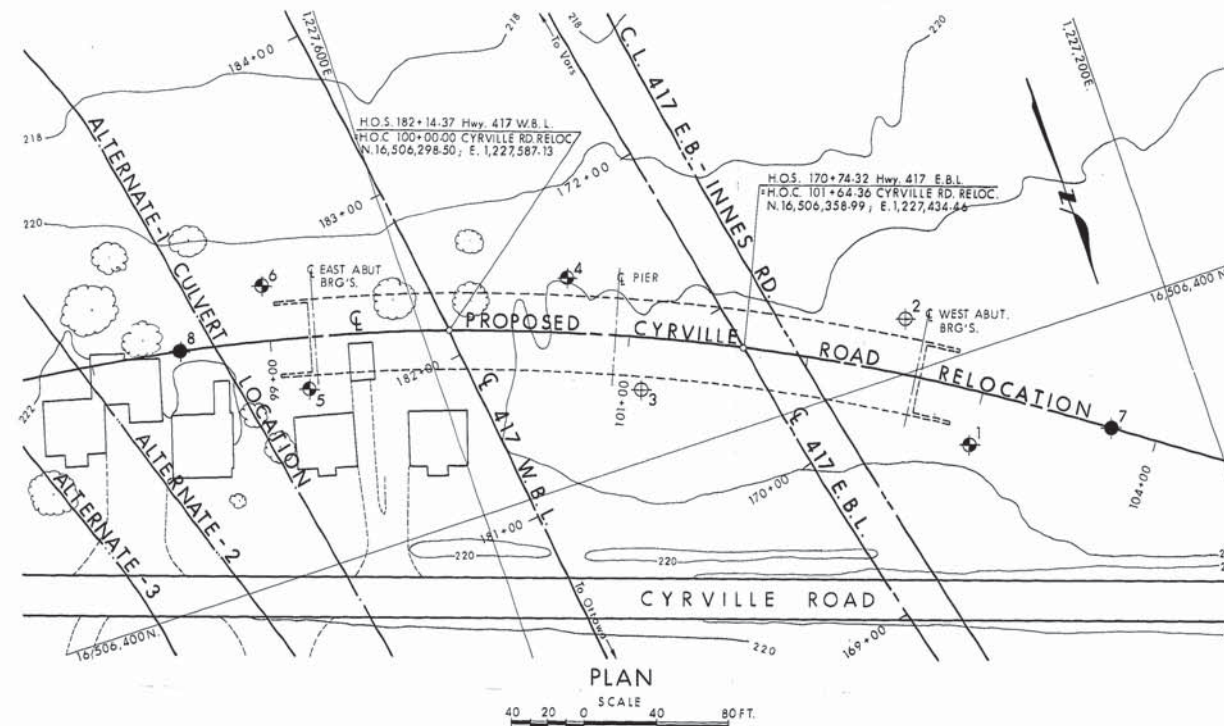
**CONSTRUCTION NOTES**  
THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8".  
NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED, STRESSED AND GROUTED.



REVISIONS	
DATE	DESCRIPTION
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
CYRVILLE ROAD UNDERPASS	
KING'S HIGHWAY No. 417	DIST. No. 9
REQ. MUNICIPALITY OF OTTAWA - CARLETON	
TWP. GLOUCESTER	LOT 24 CON. II
GENERAL LAYOUT	
APPROVED	CONTRACT No. 73-192
DESIGN	W.P. No. 13-68-04
DRAWING	
DATE	

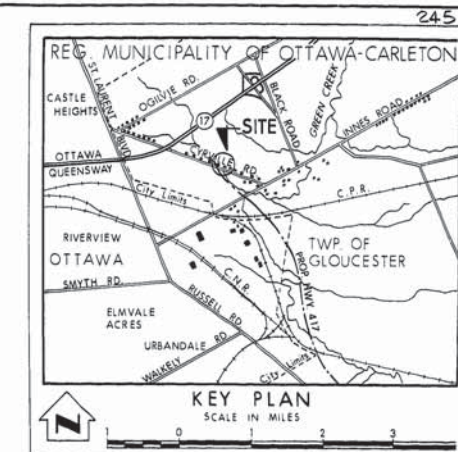


71649 TWP #56-314-1-A 8 A



NOTE:  
The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the OTTAWA District Office.

REF. DELEUW CATHER 284-134-22-4



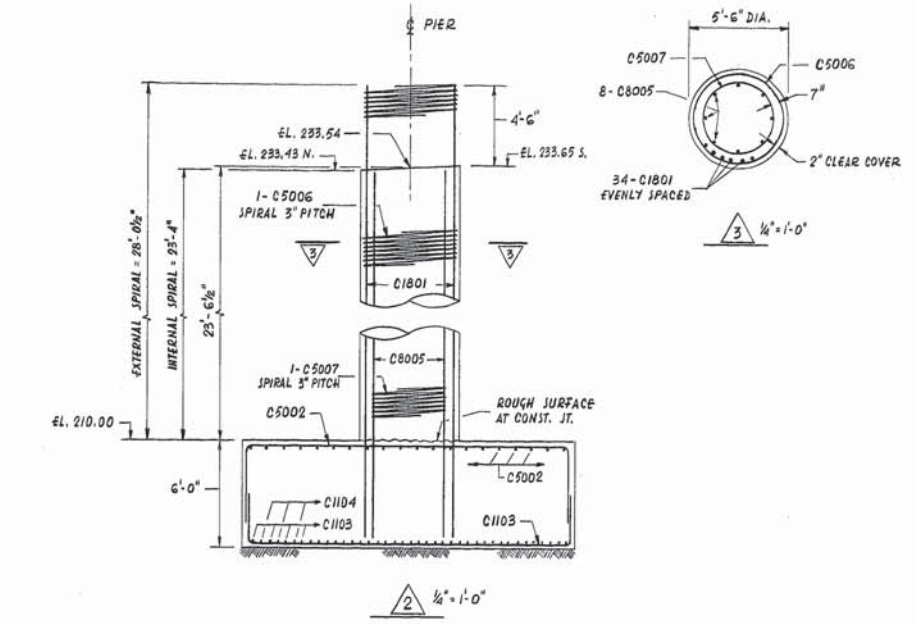
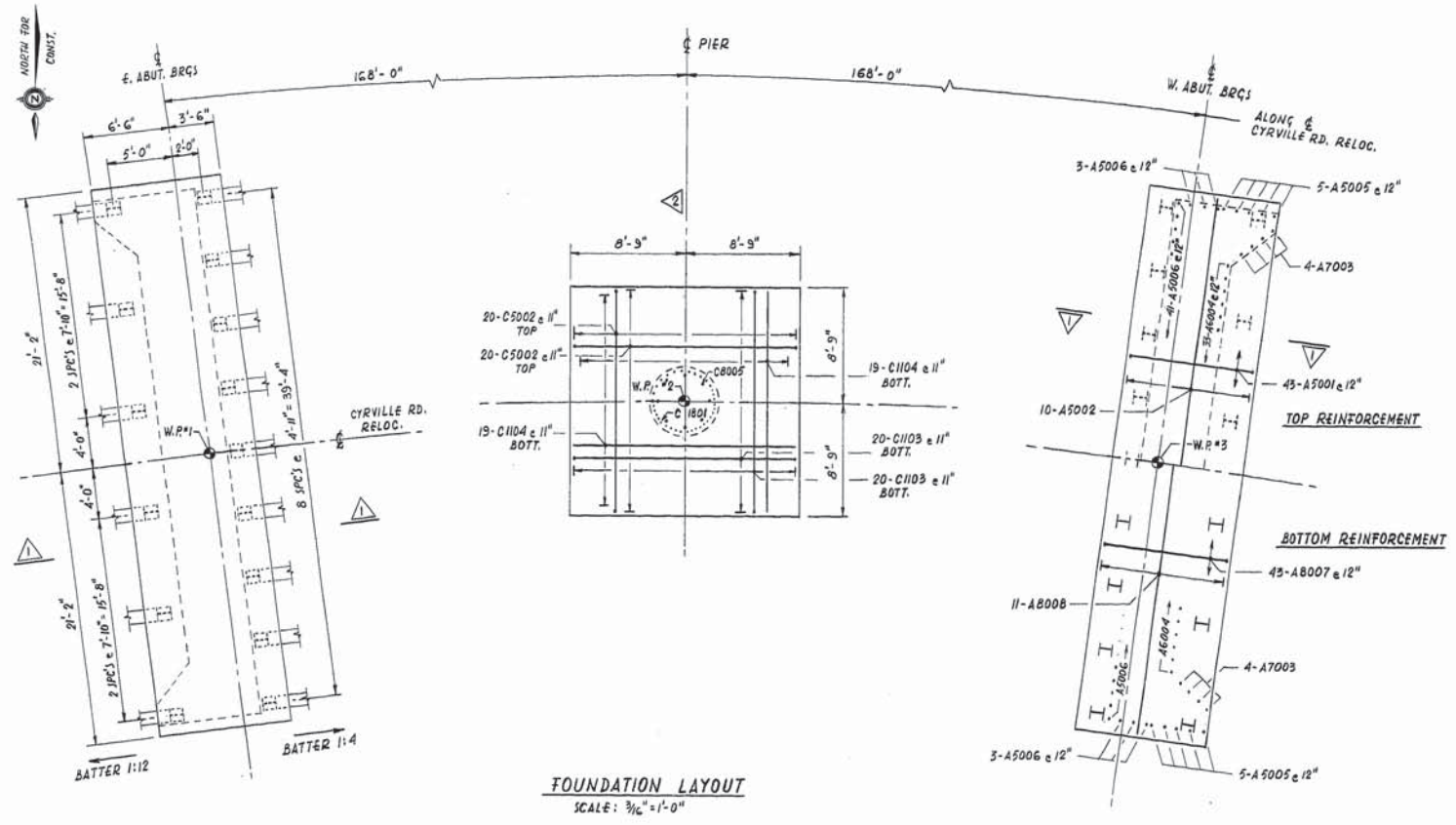
LEGEND			
			Bore Hole
			Cone Penetration Test
			Bore Hole & Cone Test
			Water Levels established at time of field investigation SEPT. 1972
NO.	ELEVATION	CO-ORDINATES NORTH	EAST
1	222.2	16,506,449	1,227,332
2	222.4	16,506,372	1,227,342
3	222.4	16,506,363	1,227,495
4	222.4	16,506,291	1,227,514
5	222.9	16,506,303	1,227,670
6	220.6	16,506,241	1,227,676
7	222.7	16,506,466	1,227,254
8	221.9	16,506,261	1,227,731

NOTE -  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION
1	28/10/77	JIG	BOREHOLE LOCATIONS REVISED

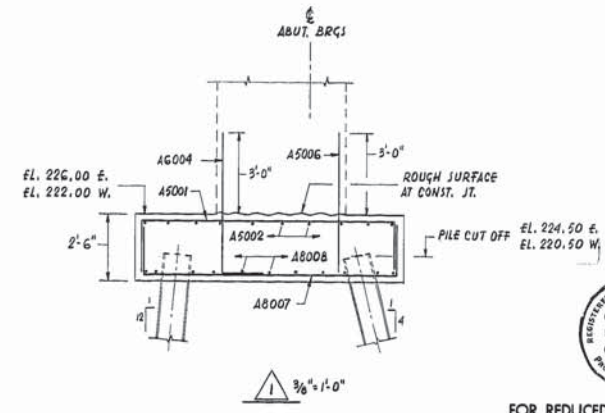
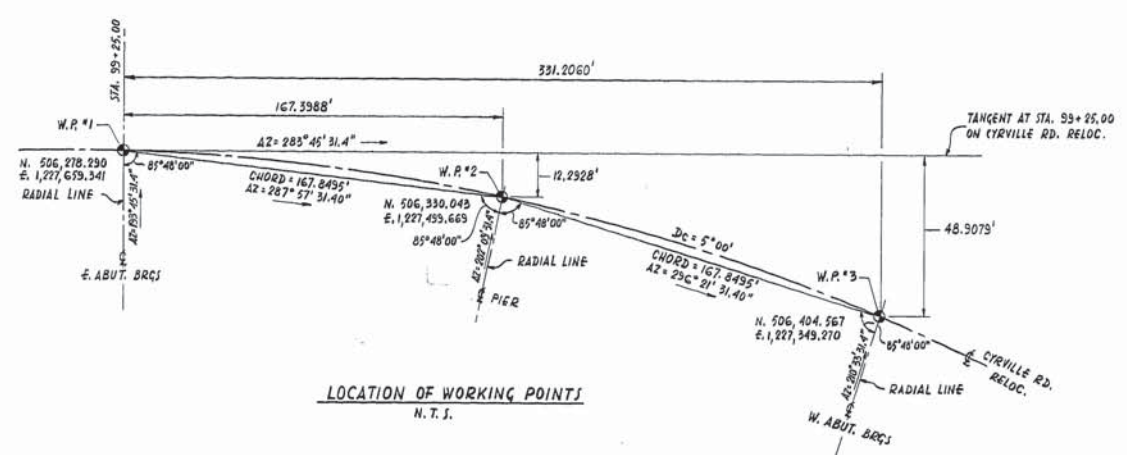
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO DESIGN SERVICES BRANCH-FOUNDATIONS OFFICE			
CYRVILLE ROAD			
HIGHWAY NO. 417	DIST. NO. 9		
CO. REG. MUNICIPALITY OF OTTAWA-CARLETON			
TWP. GLOUCESTER LOT CON.			
BORE HOLE LOCATIONS & SOIL STRATA			
SUBMD. S.A.	CHECKED <i>[Signature]</i>	WP. NO. 13-68-04	DRAWING NO.
DRAWN J.I.G.	CHECKED <i>[Signature]</i>	WD. NO. 72-11109	72-11109 A
DATE OCT. 25, 1972	SITE NO. 3-314	BRIDGE DRAWING NO.	
APPROVED <i>[Signature]</i>	CONT. NO. 73-192	3-314-2	
TWP. 56-314-2-A			

71657



PILE DATA				
LOCATION	BATTER	Nº REQ'D	TYPE	LENGTH
EAST ABUT.	1:12	6	HP 12x74	17'-0"
	1:4	9		
WEST ABUT.	1:12	6	HP 12x74	12'-0"
	1:4	9		

- NOTES:
- DIMENSIONS, REINF. & PILE LAYOUT SIMILAR FOR BOTH ABUTMENTS, EXCEPT AS SHOWN.
  - SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTING.
  - BOTTOM REINF. TO BE SPACED TO AVOID PILES.
  - PILES TO BE DRIVEN TO BEDROCK.
  - PIER FOOTING TO BE FOUND ON SOUND BEDROCK.



REVISIONS	
DATE	DESCRIPTION
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
CYRVILLE ROAD UNDERPASS	
KING'S HIGHWAY No. 417	DIST. No. 9
REG. MUN. OF OTTAWA-CARLETON	
TWP. GLOUCESTER	LOT 24 CON. II
FOOTINGS AND PIER COLUMN	
APPROVED	CONTRACT No. 73-192
DESIGN H.P.	CHECK J.K.D.
DRAWING R.K.	CHECK K.D.
DATE 1/8/1973	LOADING 1/20-44
	W.P. No. 13-68-04
	SITE No. 3-314
	SHEET 3



71658 TWP. 56-314-3-A



**APPENDIX 5**  
**SITE 3-305**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Kenton Power, P.Eng.  
(Reviewed by Fred Griffiths, P.Eng. and  
P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 INNES ROAD UNDERPASS (SITE 3-305) GWP 4074-11-00 GEOCRES 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing Highway 417 Underpass of Innes Road in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4156-10-01
- Site No. 3-305
- GEOCRES No. 31G5-81
- Construction Contract 73-191
- Historic W.P. 13-68-05

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa, in the Township of Gloucester approximately 350 m east of the intersection of Star Top Road and Innes Road. The two-span structure with center pier and abutments carries both the Innes Road east and westbound lanes (four lanes in total plus a concrete median and sidewalks) and the on and off ramps for the Highway 417 / Innes Road Interchange over both the east and westbound lanes of Highway 417. Prior to construction of the bridge, Green Creek flowed from south to north at a location further to the west. The creek was diverted to an area of relatively flat ground at an approximate elevation of 63.4 m.

Based on the historic General Plan Drawing (copy attached) the bridge is approximately 22 m wide, and 85 m long with a cast-in-place concrete deck. The bridge abutments are supported by



steel HP12x74 piles driven to bedrock. The center pier includes three columns, each supported by a spread footing founded on bedrock.

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-81 dated February 1972. The investigation consisted of five sampled boreholes designated 1 to 5; all of which were accompanied by dynamic cone penetration tests. Two additional dynamic cone penetration tests were also carried out opposite both Boreholes 1 and 4. Drawing No. 71-11127A (copy attached) illustrates the locations of the bridge, the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by a silty sand stratum, underlain by shale bedrock.

#### **3.1 Silty Sand to Sandy Silt**

The top of the silty sand to sandy silt layer ranged from 63.9 m to 65.7 m, in elevation with a thickness of 3.4 m to 5.1 m. The standard penetration test (SPT) 'N' values ranged from 7 to 79 blows per 0.3 m of penetration indicating a loose to very dense condition; but typically compact.

Seams of clayey silt were noted within this unit. Atterberg Limit Testing on these materials indicated low plasticity.

Gradation test results on three samples of the silty sand material indicate a gravel content between 0% and 24%, sand content between 58% and 87% and fines content (combined silt and clay content) between 13% and 18%. Gradation test results on two samples for the sandy silt material indicated a gravel content between 2% and 7%, a sand content between 26% and 37%, a silt content between 58% and 64%, and clay content of 3% for both samples tested.

The moisture content of the samples tested ranged from 7% to 30%.

#### **3.2 Bedrock**

A grey shale bedrock was encountered beneath the silty sand to sandy silt stratum in all five boreholes as proven by BX size coring in Boreholes 1 to 4; and split spoon refusal on probable bedrock in Borehole 5. The bedrock surface elevation ranged from 59.6 m to 62.2 m.

Bedrock core recovery ranged from 91.5% to 99.5%. The bedrock was described to be in sound condition. Geological mapping suggests that this site is at the boundary between the Billings and Carlsbad Formations.

#### **3.3 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 61.9 m to 62.8 m.



#### 4 SITE OBSERVATIONS

A structure inspection was conducted by MTO in June 2010 for Bridge 3-305 with the report issued January 2011. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering Ltd. staff during the week of July 14, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

##### West Abutment:

- Concrete slope paving was installed at the site for erosion control of the abutment slopes
- Abutment slope was measured at approximately 25° and was noted as being in generally good condition
- Rust staining and spalling of the concrete were noted on the underside of the bridge deck and on the abutment foundations
- Vegetation on the embankment side slopes was noted with no obvious signs of settlement or erosion

##### East Abutment:

- Concrete slope paving was installed at site for erosion control of abutment slopes
- Abutment slope was measured at approximately 26° and was noted as being in generally good condition with only slight settlement noted at the bottom of the slope
- Rust staining and spalling of the concrete were noted on the underside of the bridge deck and on the abutment foundation
- Vegetation on the embankment side slopes was noted with no obvious signs of settlement or erosion

##### Pier:

- Could not be safely accessed and as such the pier was visually inspected from the west abutment
- No obvious signs of foundation settlement were observed
- Ditchline was vegetated and some ponded water was noted
- Slight rust staining of the pier columns was noted but no concrete spalling

##### Bridge and Road Surface:

- Concrete spalling of the sidewalks, barriers and sides of bridge deck was noted
- Frequent longitudinal and transverse cracking of the asphalt surface was noted
- Some potholes, patching and crack repair of the asphalt surface were noted
- A slight dip in the road west of the bridge was observed



## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

Where insufficient earth cover is provided, polystyrene insulation may be used to enhance frost protection.

#### **5.2 Seismic Considerations**

This site is classified as a Soil Profile Type I in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC). The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the reported density of the silty sand/sandy silt and the measured groundwater at the site at the time of the investigation, these materials are classified as “not susceptible” to liquefaction during the design earthquake event.

#### **5.3 Existing Foundations**

As per the Footings and Pier Columns Drawing the bridge abutments were designed to be supported on steel HP12x74 piles driven to bedrock. The available construction drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the abutment foundations is 100 tons / HP12x74 pile or approximately 890 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN/pile for HP310x110 piles driven to bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

The existing pier is supported on three, 3.4 m x 3.4 m footings founded on bedrock. The Foundation Design Report recommended that spread footings founded in the sand stratum not be used due to the likelihood of excess settlements. The design loads for the pier foundations were also not indicated on the design drawings, however the Foundation Design Report recommends an allowable bearing pressure of up to 15 tsf or approximately 1,400 kPa for pier foundations founded on or in the sound shale bedrock.



## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Shallow Foundations**

Although spread footings within the overburden are possible, effective dewatering of the sand and gravel strata would be required. Given the anticipated relatively shallow depth to the shale bedrock, and the higher geotechnical resistance offered by the bedrock, spread footings within the overburden are not recommended. In addition, footings for potential widenings should match the elevation of the existing foundation element, i.e. be founded on bedrock.

The factored vertical geotechnical resistance of 1500 kPa at ULS may be used for the preliminary design of shallow foundations, founded on or in sound shale bedrock. The SLS condition will not govern for footings in or on the bedrock. The design of any new foundations on bedrock would need to consider interactions with the existing foundations and potential for undermining of the existing structural elements.

Resistance to lateral forces and sliding resistance between concrete and underlying materials should be evaluated using an unfactored coefficient of friction of 0.50 for cast-in-place concrete and 0.45 for pre-cast concrete on sound shale bedrock.

Based on current geological mapping, the shale present at this site may be from the Billings Formation which is susceptible to heaving if allowed to weather in the presence of oxygen. The general mechanism is that oxidation of pyrite within the shale produces sulfuric acid, which in turn reacts with calcite in the shale to form gypsum crystals, which occupy a larger volume than the original materials. A by-product of this chain of reactions also tends to increase sulphate levels which can attack buried concrete structures.

The potentially detrimental effects of shale heaving can be avoided by preventing exposure of the shale to oxygen both during construction and long term. This is typically achieved by limiting exposure of the shale to no more than one day prior to covering with a protective layer such as shotcrete or a concrete mud slab. Sulphate resistant cement should be used in such applications.

### **6.2 Deep Foundations – Piles**

Although the depth to bedrock is too shallow at the existing pier location for piles to be practical, driven piles are considered to be suitable for the support of abutment foundations perched within the approach embankments. It should be noted that the bedrock surface elevation ranges from 59.6 m to 62.2 m.



### 6.2.1 Axial Resistance

Steel piles (Grade 350W steel) end-bearing on sound bedrock at this site may be designed on the basis of the following factored, vertical, geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

### 6.2.2 Pile Tips

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### 6.2.3 Integral Abutment Considerations

As per the Footings and Pier Columns Drawing the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### 6.2.4 Pile Spacing

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### 6.2.5 Downdrag

In light of the cohesionless makeup of the existing overburden soils at this site, downdrag on existing and new piles is not considered a design issue.

### 6.2.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formula for cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where:

- $k_h$  = coefficient of horizontal subgrade reaction
- $n_h$  = coefficient related to soil density given in Table A
- $z$  = depth of pile embedment (m)
- $B$  = pile width perpendicular to load direction (m)

**Table A:**  $n_h$  values for cohesionless soils.

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 62	Embankment Fill	3,000
Below 62	Sandy Silt to Silty Sand	2,000

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B below. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.2.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

- $C$  = pile circumference (m)
- $q_s$  = soil unit shaft friction
- $\Delta z$  = subdivided segment of the embedded length (m)
- $W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)

**Table C:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 62	Embankment Fill	20	0.4
Below 62	Sandy Silt to Silty Sand	19	0.4

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D: Static Lateral Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Sandy Silt & Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.5
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33

#### 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4. of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where A is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Sandy Silt & Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$
<b>Yielding Wall</b>		
$K_{AE}$	0.33	0.40
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36
<b>Non-Yielding Wall</b>		
$K_{AE}$	0.55	0.66
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43



The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in the table above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## 6.5 Approach Embankments

The General Plan Drawing indicates that the maximum height of the approach fills was approximately 7.3 m from proposed Highway 417 elevation with an embankment slope of 2H:1V (Horizontal:Vertical). Design drawings indicate that the abutment backfill is to consist of compacted boulder free fill. In addition, the Foundation Design Report states that the approach fills will settle approximately 38 mm in elastic compression. The embankment foundation is expected to be stable and no long term settlement problems are expected unless the fills are reconfigured.

## 6.6 Erosion Control

No active erosion below the slope paving was noted at either abutments locations at this site. Slight settlement of the slope paving was noted at the bottom of the slope of the east abutment.

The slope paving should be repaired where required and maintained and drainage measures should be enhanced beneath the abutments to prevent future erosion below the slope paving.



## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native overburden soils reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one.

Should the proposed works result in excavations to bedrock, samples of the bedrock should be acquired during the investigation and submitted for chemical testing to determine the pyritic heave potential.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.



Kenton C. Power, P.Eng.  
Geotechnical Engineer



Fred Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer

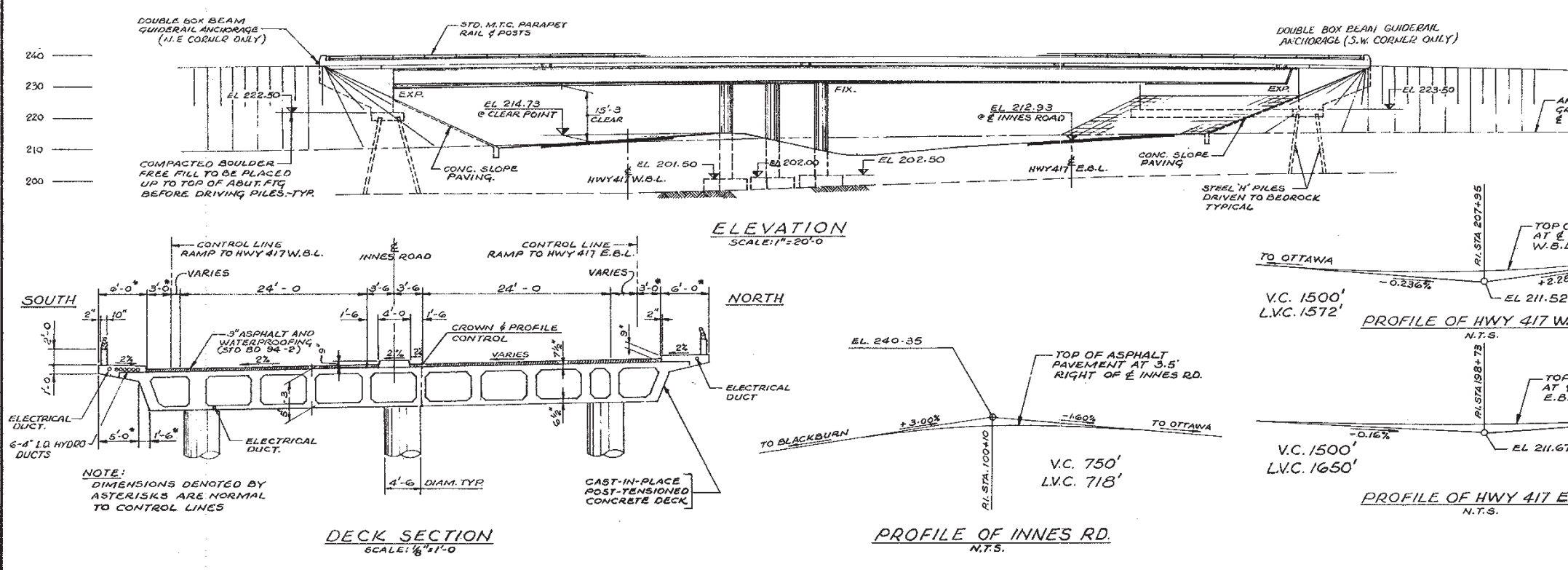
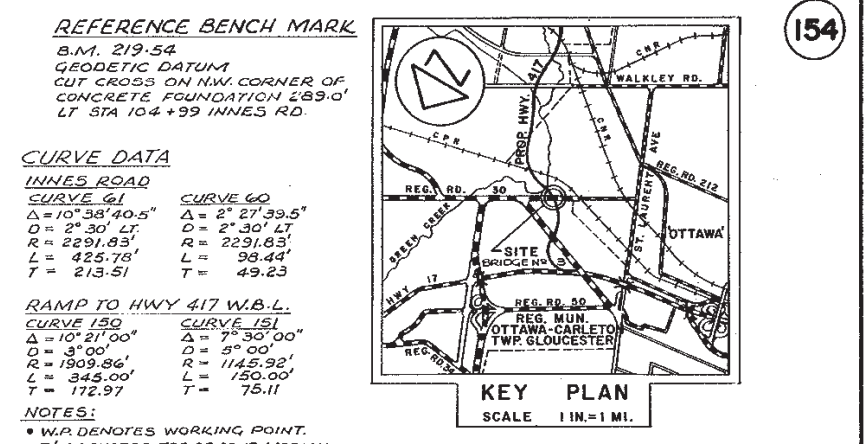
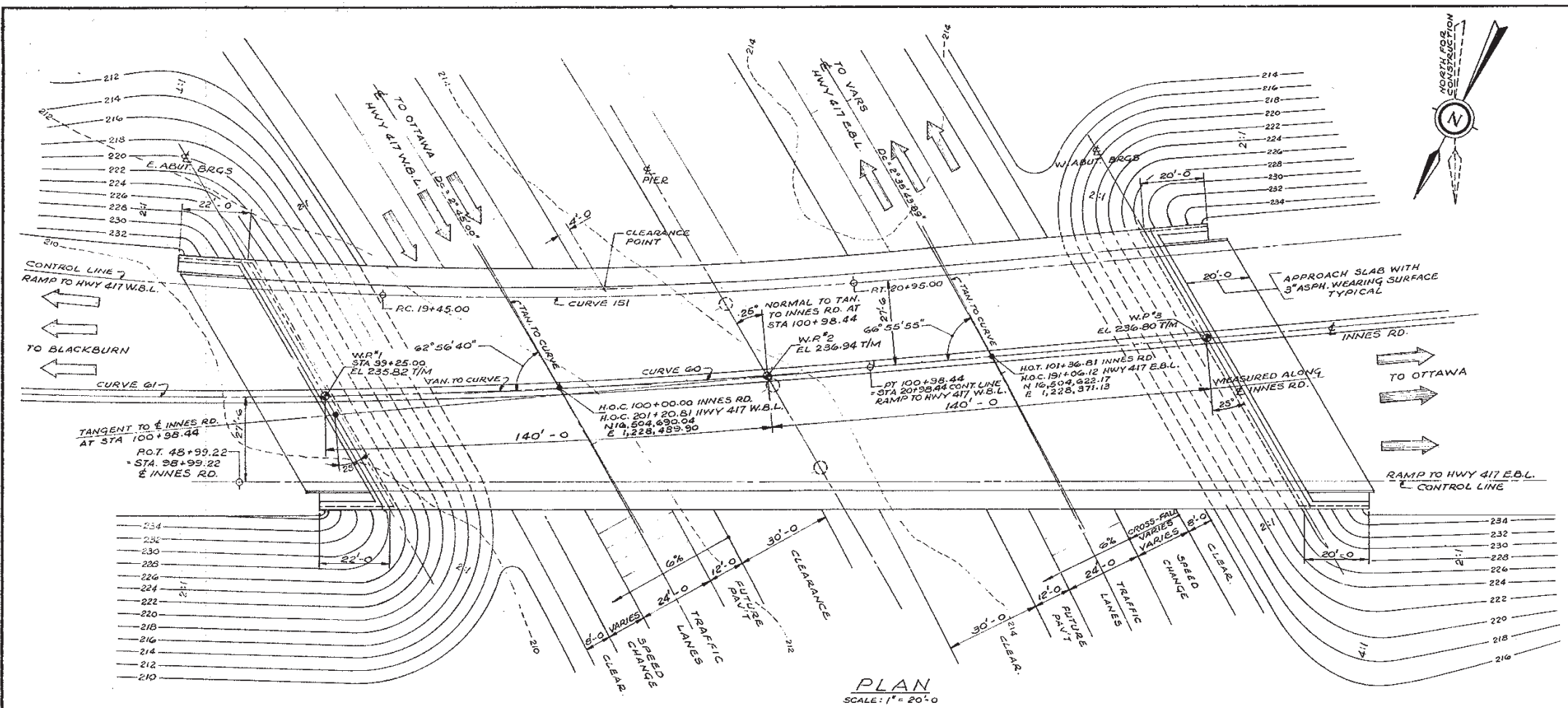


P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

## Attachments

Client: McIntosh Perry  
File No.: 19-3405-3  
E file: 3-305 tel

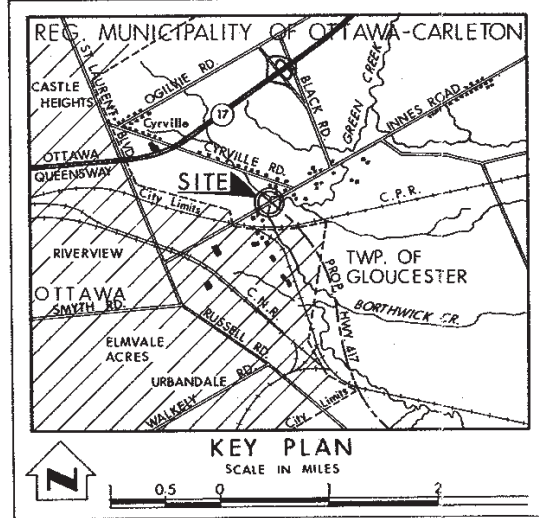
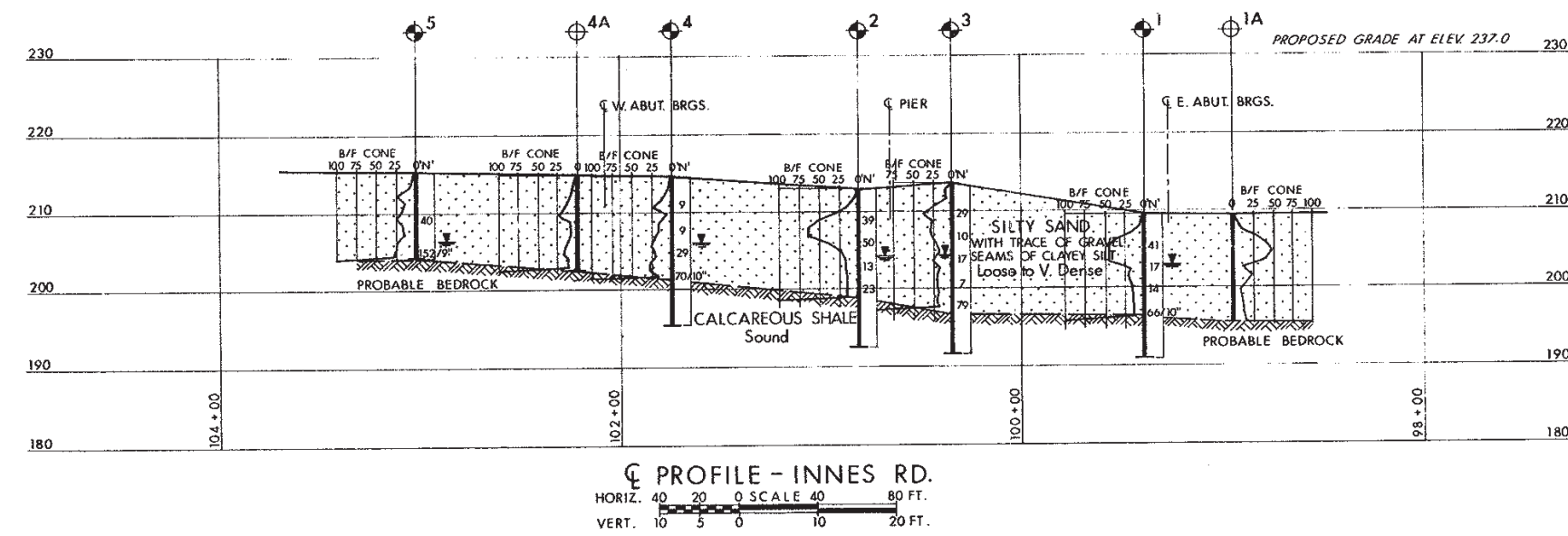
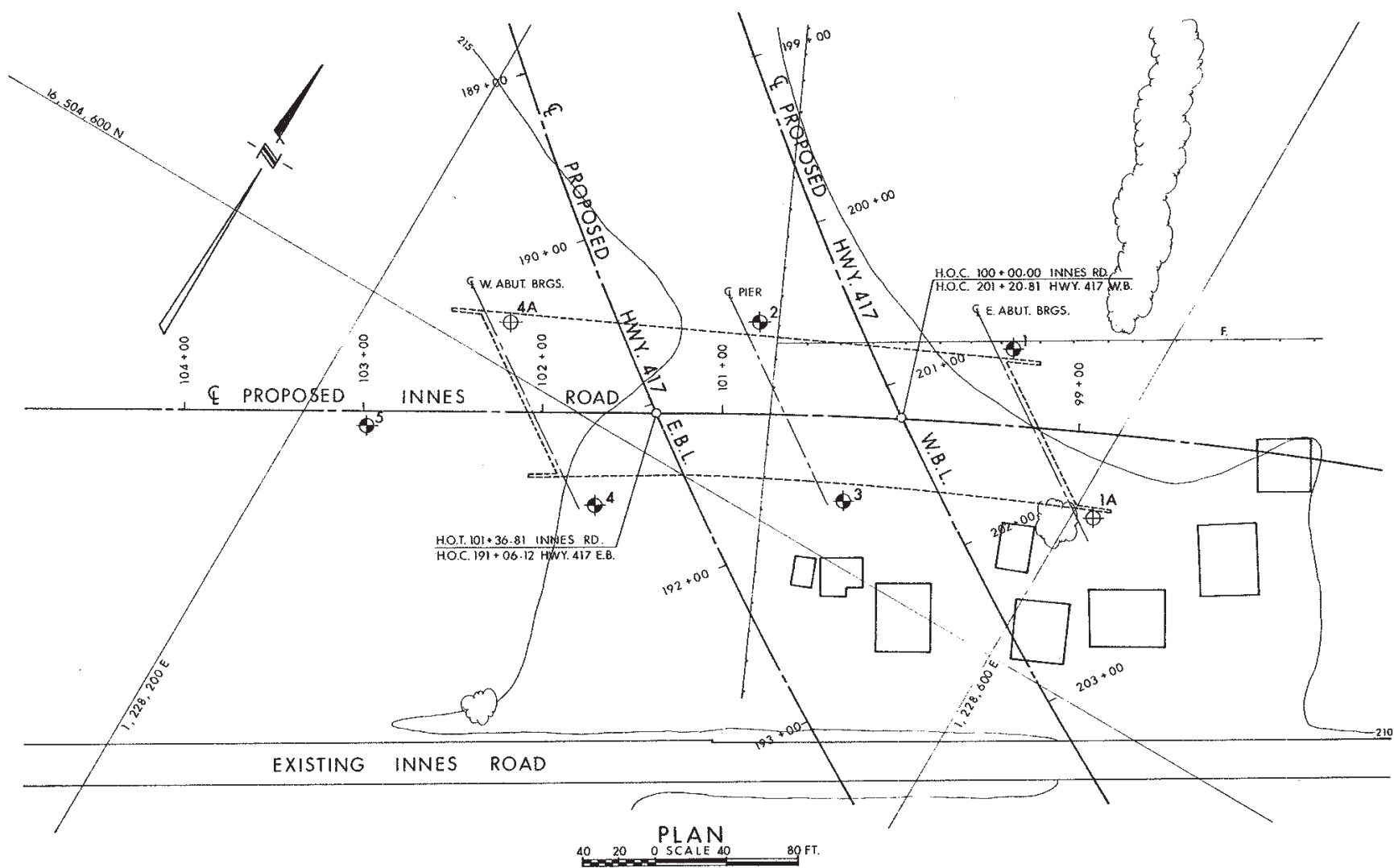
Date: August 21, 2015  
Page 12



GENERAL NOTES  
CLASS OF CONCRETE:  
DECK, SIDEWALKS & MEDIAN - 5000 P.S.I.  
PARAPETS - 5000 P.S.I.  
PIER COLUMNS - 4000 P.S.I.  
REMAINDER - 3000 P.S.I.  
CLEAR COVER ON REINFORCING STEEL:  
FOOTINGS, ABUTMENTS & PIERS - 3"  
DECK: TOP SLAB - TOP 1 1/2", BOT. 1"  
BOT. SLAB - TOP & BOT. 1"  
INTERIOR WEBS & DIAPHRAGMS - 1 1/2"  
OUTSIDE FACE OF EXTERIOR WEBS - 1 1/2"  
SIDEWALKS & MEDIAN - 2"  
EXCEPT AS NOTED.  
CONSTRUCTION NOTES:  
THE CONTRACTOR IS RESPONSIBLE FOR  
FINISHING THE BEARING SEATS DEAD LEVEL TO  
THE SPECIFIED ELEVATIONS WITH A TOLERANCE  
OF  $\pm 1/8"$ .  
NO CONCRETE SHALL BE PLACED ABOVE THE  
ABUTMENT BEARING SEATS UNTIL THE  
CONCRETE IN THE DECK HAS BEEN PLACED,  
STRESSED & GROUTED.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO  
De Leuw, Cather  
CONSULTING ENGINEERS - OTTAWA  
BRIDGE NO. 3  
INNES ROAD INTERCHANGE  
(1.0 MILE EAST OF O.Q.W.)  
KING'S HIGHWAY No. 417 DIST. No. 9  
CO. REG. MUNICIPALITY OF OTTAWA - CARLETON  
TWP. GLOUCESTER LOT 23 & 24 CON. 2  
GENERAL PLAN  
SITE No. 3-305 W.P. No. 13-68-05  
APPROVED: [Signature] BRIDGE ENGINEER  
CONTRACT No. 73-1  
DESIGN: G.S.S. CHECK: L.D.H.  
DRAWING: K.A.B. CHECK: G.S.S.  
DATE: FEB. 73 LOADING: HS20-44 DRAWING No. 3-305-1



LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation. Nov. & Dec. 1971.		
NO.	ELEVATION	CO - ORDINATES	
		NORTH	EAST
1	209.6	16,504,750	1,228,523
1A	209.5	16,504,689	1,228,612
2	213.0	16,504,690	1,228,400
3	213.7	16,504,625	1,228,484
4	214.7	16,504,572	1,228,341
4A	215.0	16,504,616	1,228,277
5	215.4	16,504,523	1,228,236

NOTE -  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS	DATE	BY	DESCRIPTION

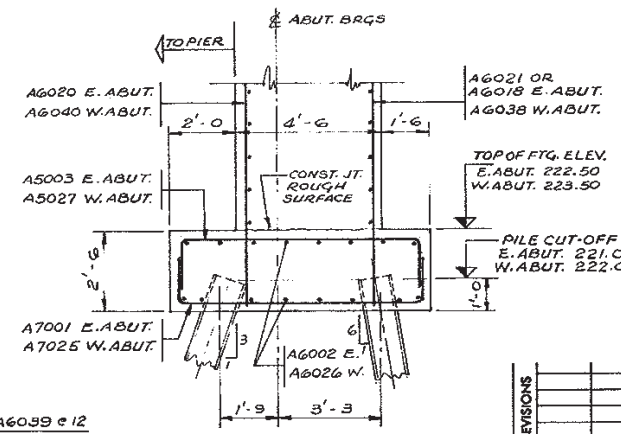
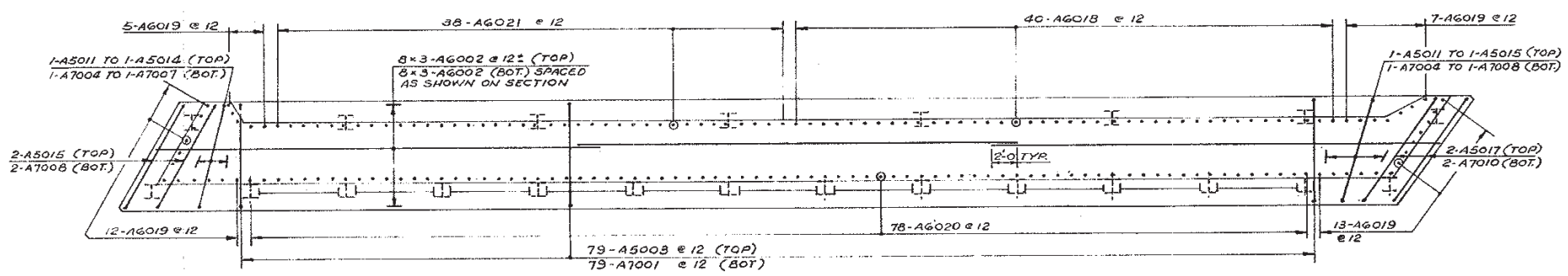
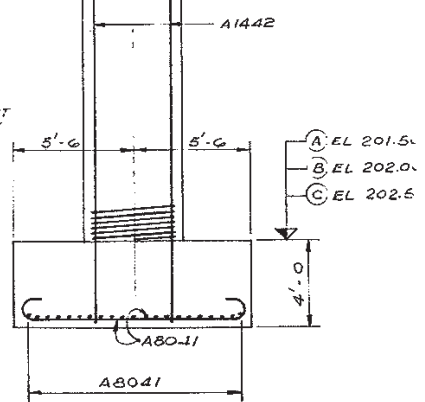
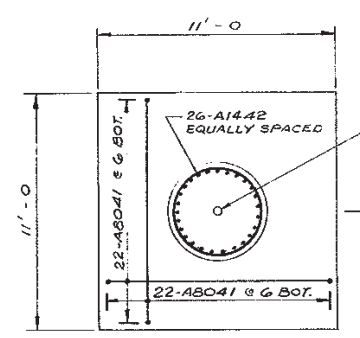
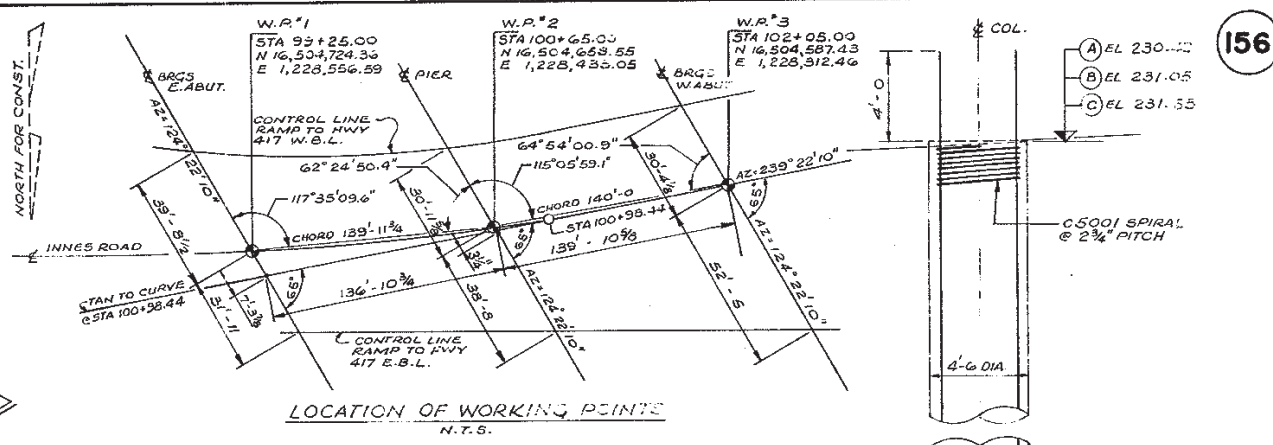
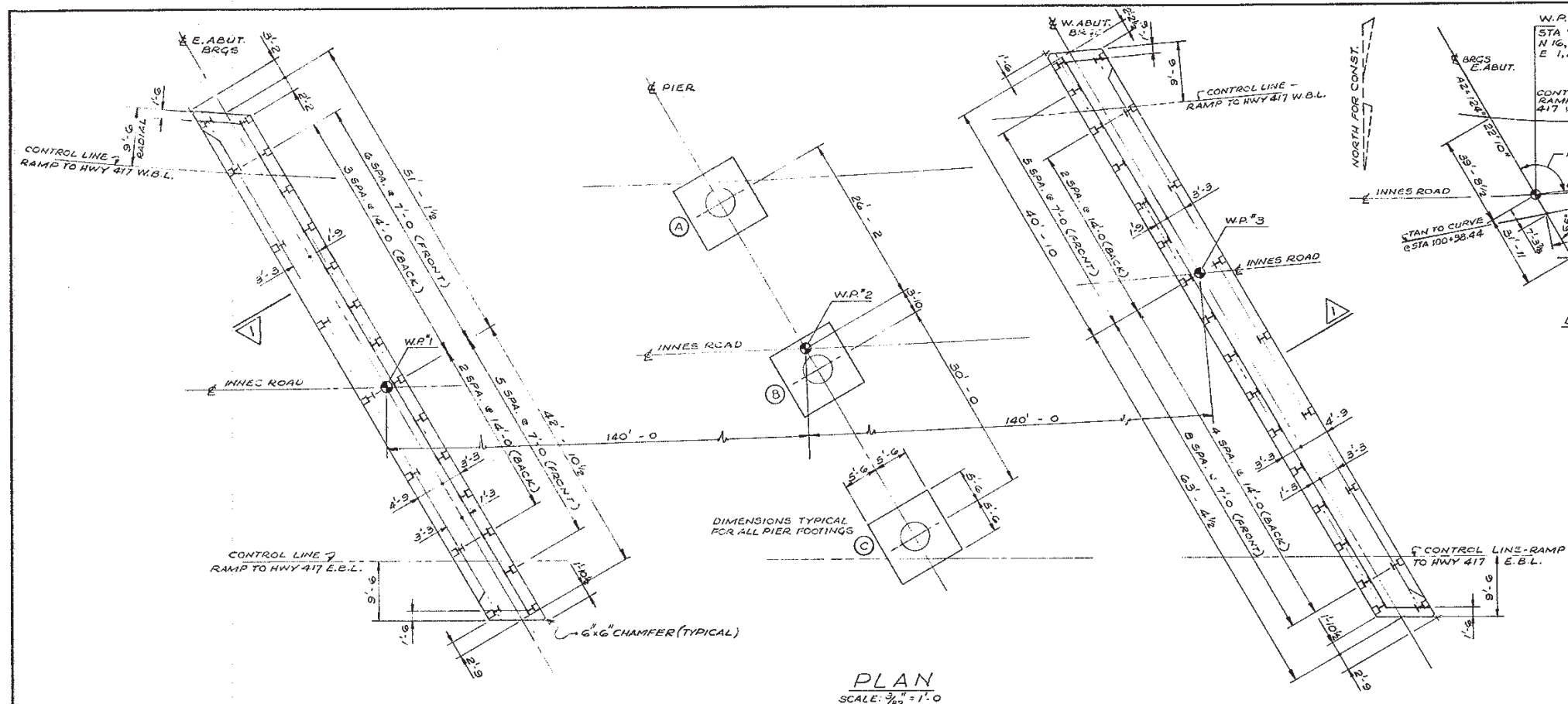
DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS  
DESIGN SERVICES BRANCH - FOUNDATION OFFICE

INNES ROAD  
HIGHWAY NO. 417 DIST. NO. 9  
REG. MUNICIPALITY OF OTTAWA - CARLETON  
TWP. LOT CCN.

BORE HOLE LOCATIONS & SOIL STRATA

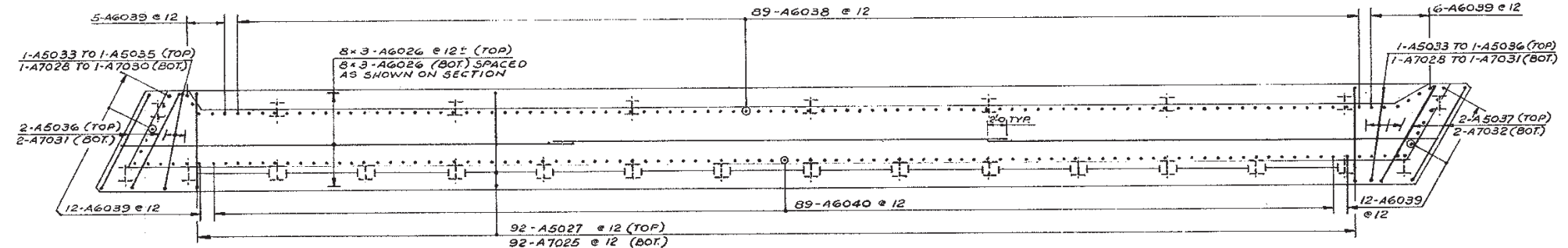
SUBMD. S.A.	CHECKED	W.P. NO. 13 - 68 - 05	DRAWING NO.
DRAWN S.R.	CHECKED	JOB NO. 71 - 11127	71 - 11127 A
DATE JANUARY 5, 1972.	SITE NO. 3 - 305	BRIDGE DRAWING NO.	
APPROVED	CONT. NO. 73 - 191		3 - 305 - 2

NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the OTTAWA District Office.

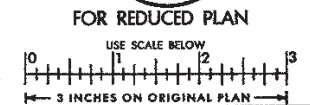


- NOTES:
- PILES TO BE DRIVEN TO BEDROCK
  - SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTINGS

REVISIONS	DATE	BY	DESCRIPTION



STEEL H'PILE DATA		
LOCATION	NO	LENGTH
E. ABUT.	22	25'-0"
W. ABUT.	25	30'-0"
		HP 12 x 74



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO

**DeLeuw, Cather**  
ENGINEERS & PLANNERS - OTTAWA

**BRIDGE No. 3**  
**INNES ROAD INTERCHANGE**  
(1.0 MILES EAST OF O.S.W.)

KING'S HIGHWAY No. 417 SITE No. 3-305 W.P. No. 13-68-05  
CO. REG. MUNICIPALITY OF OTTAWA - CARLETON  
TWP. GLOUCESTER LOT 23 & 24 CON. 2

**FOOTINGS & PIER COLUMNS**

APPROVED: *[Signature]* CONTRACT No. 7-7  
DESIGN: G.S.S. CHECK: L.D.H. CONTRACT No. 7-7  
DRAWING: K.A.B. CHECK: G.S.S. DRAWING No. 3-305-3  
DATE: FEB. '73 LOADING: HS20-44



**APPENDIX 6**  
**SITE 3-311/1**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Kenton Power, P.Eng.  
(Reviewed by Fred Griffiths, P.Eng. and  
P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 EASTBOUND BRIDGE OVER GREEN CREEK (SITE 3-311/1) GWP 4074-11-00 GEOCRES 31G5-263**

#### **PART 1: FACTUAL INFORMATION**

##### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing overpass structure carrying eastbound traffic on Highway 417 over Green Creek in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4023-11-00
- Site No. 3-311/1
- GEOCRES No. 31G5-85
- Construction Contract 73-191
- Historic W.P. 13-68-09

##### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa, in the Township of Gloucester approximately 270 m south-east of the Highway 417 / Innes Road Interchange. The single span structure with abutments, carries the eastbound lanes of Highway 417 (two lanes in total plus paved shoulders) and the west-south on ramp for the Innes Road Interchange over Green Creek.

Based on the historic General Plan Drawing (copy attached) the bridge is approximately 24.4 m long with a width varying from 30.4 to 31.9 m with a concrete pre-stressed girder structure. Both of the bridge abutments were to be supported by steel HP12x74 piles driven to bedrock. The natural terrain in the vicinity of the bridge was generally flat. Prior to construction of the bridge,



Green Creek flowed from south to north at a location further to the west. The creek was re-aligned from its original course as part of the construction works at this site.

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-85 dated August 1972. The investigation consisted of two sampled boreholes designated 8 and 9; both accompanied by dynamic cone penetration tests (DCPT). Two additional DCPT test designated 5 and 6 were also carried out opposite both Boreholes 8 and 9. Drawing No. 72-11067A (copy attached) illustrates the locations of the bridge and the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by a sandy silt stratum at the east abutment (Borehole 8) and a silty sand at the west abutment (Borehole 9), underlain by shale bedrock.

#### **3.1 Sandy Silt**

A sandy silt stratum with occasional clayey silt seams was encountered in Borehole 8. The surface of this deposit ranged from 61.7 m in elevation, and the layer had a thickness of 3.1 m. The standard penetration test (SPT) 'N' values ranged from 4 to 8 blows per 0.3 m of penetration; indicating a loose condition.

The results of a grain size analysis including hydrometer testing completed on a sample of this material indicated a gravel content of 1%, sand content of 38%, silt content of 49%, and clay content of 12%. Atterberg Limits test was carried out on material from the clayey silt seam indicate a clay of low plasticity.

The moisture content of the sample tested was 24%.

#### **3.2 Silty Sand**

A silty sand stratum with occasional clayey silt seams was encountered in Borehole 9. The surface of this deposit was at 64.1 m in elevation, and the layer had a thickness of 7.0 m. SPT 'N' values range from 9 to 30 blows per 0.3 m of penetration, indicating a loose to compact condition; but typically compact.

The results of a grain size analysis including one with hydrometer testing completed on two samples of this material indicated a gravel content between 3% and 18%, sand content between 60% and 71%, and a fines content (combined silt and clay content) of 11% and 37%.

No moisture content testing was conducted on this material.

#### **3.3 Bedrock**

A shale bedrock was encountered beneath the silty sand and sandy silt strata in both Boreholes 8, and 9 as proven by BX size coring, and DCPT refusal on assumed bedrock in Boreholes 5 and 6. The bedrock surface elevation ranged from 57.1 m to 58.6 m.



Bedrock core recovery was 100% in both boreholes. The bedrock was described to be in sound condition. Geological mapping suggests that this site is near the boundary between the Billings and Carlsbad Formations.

### **3.4 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 61.0 m to 63.0 m. The water level of Green Creek at the time of the investigation is indicated on the General Plan Drawing as 60.1 m.

## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in August 2012 for Bridge 3-311/1 with the report issued October 2013. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in good condition. The report recommended major rehabilitation of the bridge structure components within 6 to 10 years of the inspection.

The site was inspected by Thurber Engineering Ltd. staff during the week of July 14, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

North and South Abutments:

- Gabions were installed for erosion protection of the abutment slopes
- Abutment slopes were measured at approximately 2H:1V (Horizontal:Vertical)
- Slight settlement of the gabions was noted along the top of the slope, otherwise the slope was described as being in generally good condition
- No obvious signs of scouring by creek flow or erosion of the toe of the abutment slope were observed
- Vegetation on the embankment side slopes was noted with no obvious signs of settlement or erosion
- Rust staining and spalling of the concrete was noted on the abutment walls

Bridge and Road Surface:

- Concrete spalling of the barriers and sides of the bridge deck was noted
- Frequent longitudinal and transverse cracking of the asphalt surface was noted
- Some potholes, patching and crack repair of the asphalt surface were noted



## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

Where insufficient earth cover is provided, polystyrene insulation may be used to enhance frost protection.

#### **5.2 Seismic Considerations**

This site is best classified as a Soil Profile Type I in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC).

The site specific peak ground acceleration (PGA) value with a 10% probability of exceedance in 50 Years of 0.123 was used in the evaluation of seismic considerations. The PGA value was obtained from the Natural Resources Canada 2010 National Building Code (NBC) Seismic Hazard Calculator obtained May 2015. A copy of the NBC Seismic Hazard Calculation Data sheet is attached for reference.

The susceptibility to liquefaction of the loose silty sand and sandy silt at this site has been evaluated using the site specific PGA value obtained from the 2010 NBC. Based on existing information these soils were found to be not susceptible to liquefaction during the design earthquake event, with Factors of Safety over 1.0.

#### **5.3 Existing Foundations**

As per the Foundation Layout and Details Drawing the bridge abutments were designed to be supported on steel HP12x74 piles driven to bedrock. The available construction drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the abutment foundations is 95 tons / HP12x74 pile or approximately 845 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN/pile for HP310x110 piles driven to bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

### **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.



## **6.1 Shallow Foundations**

Although spread footings within the overburden are possible, effective dewatering of the sand and gravel strata would be required. Given the anticipated relatively shallow depth to the shale bedrock, and the higher geotechnical resistance offered by the bedrock, spread footings within the overburden are not recommended.

The factored vertical geotechnical resistance of 1500 kPa at ULS may be used for the preliminary design of shallow foundations, founded on or in sound shale bedrock. The SLS condition will not govern for footings in or on the bedrock. The design of any new foundations on bedrock would need to consider interactions with the existing foundations and potential for undermining of the existing structural elements.

Resistance to lateral forces and sliding resistance between concrete and underlying materials should be evaluated using an unfactored coefficient of friction of 0.50 for cast-in-place concrete and 0.45 for pre-cast concrete on sound shale bedrock.

Based on current geological mapping, the shale present at this site may be from the Billings Formation which is susceptible to heaving if allowed to weather in the presence of oxygen. The general mechanism is that oxidation of pyrite within the shale produces sulfuric acid, which in turn reacts with calcite in the shale to form gypsum crystals, which occupy a larger volume than the original materials. A by-product of this chain of reactions also tends to increase sulphate levels which can attack buried concrete structures.

The potentially detrimental effects of shale heaving can be avoided by preventing exposure of the shale to oxygen both during construction and long term. This is typically achieved by limiting exposure of the shale to no more than one day prior to covering with a protective layer such as shotcrete or a concrete mud slab. Sulphate resistant cement should be used in such applications.

## **6.2 Deep Foundation – Piles**

Based on the review of existing subsurface stratigraphy at the site driven piles are considered to be suitable for the support of abutment foundations perched within the approach embankments. It should be noted that the bedrock surface elevation ranges from 57.1 m to 58.6 m.

### **6.2.1 Axial Resistance**

Steel piles (Grade 350 W steel) end-bearing on sound bedrock at this site may be designed on the basis of the following factored, vertical, geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### 6.2.3 Integral Abutment Considerations

As per the Foundation Layout and Details Drawing the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### 6.2.4 Pile Spacing

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### 6.2.5 Downdrag

In light of the cohesionless makeup of the existing overburden soils at this site, downdrag on existing and new piles is not considered a design issue.

### 6.2.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formula for cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where:

- $k_h$  = coefficient of horizontal subgrade reaction
- $n_h$  = coefficient related to soil density given in Table A
- $z$  = depth of pile embedment (m)
- $B$  = pile width perpendicular to load direction (m)

**Table A:**  $n_h$  values for cohesionless soils.

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 62.5	Embankment Fill	3,000
Below 62.5	Sandy Silt to Silty Sand	2,000



The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B below. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.2.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)

**Table C:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 62.5	Embankment Fill	20	0.4
Below 62.5	Sandy Silt to Silty Sand	19	0.4



A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D:** Static Lateral Earth Pressure Coefficients

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Sandy Silt & Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.5
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33



## 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E:** Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Sandy Silt & Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$
<b>Yielding Wall</b>		
$K_{AE}$	0.33	0.40
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36
<b>Non-Yielding Wall</b>		
$K_{AE}$	0.55	0.66
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil ( $\text{kN/m}^3$ )

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.



The factors provided in Table E are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## **6.5 Approach Embankments**

The General Plan Drawing states that the maximum height of the approach fills was approximately 6.8 m from stream bed to approach fill with an embankment slope of 2H:1V. Design drawings indicate that the abutment backfill is to consist of compacted boulder free fill. The Foundation Design Report states that settlement of the approach fills will be elastic in nature and negligible after the completion of the fills. The embankment foundation is expected to be stable and no long term settlement problems are expected unless the fills are reconfigured.

## **6.6 Erosion Control**

The gabions should be maintained to prevent erosion and scouring of the abutment slopes.

## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native overburden soils reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.



## **THURBER**

Based on a preliminary assessment of existing data the native materials at this site were found to be not susceptible to liquefaction during the design earthquake event. During detailed design the potential for and the implications of liquefaction should be confirmed. This may include drilling foundation boreholes in the approaches to determine the condition of the embankment fill and underlying native silt and sand.

During detailed design, the need for vibration monitoring will also need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one.

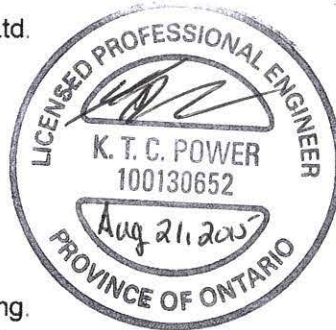
Should the proposed works result in excavations to bedrock, samples of the bedrock should be acquired during the investigation and submitted for chemical testing to determine the pyritic heave potential.



## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.



Kenton C. Power, P.Eng.  
Geotechnical Engineer



Fred Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer

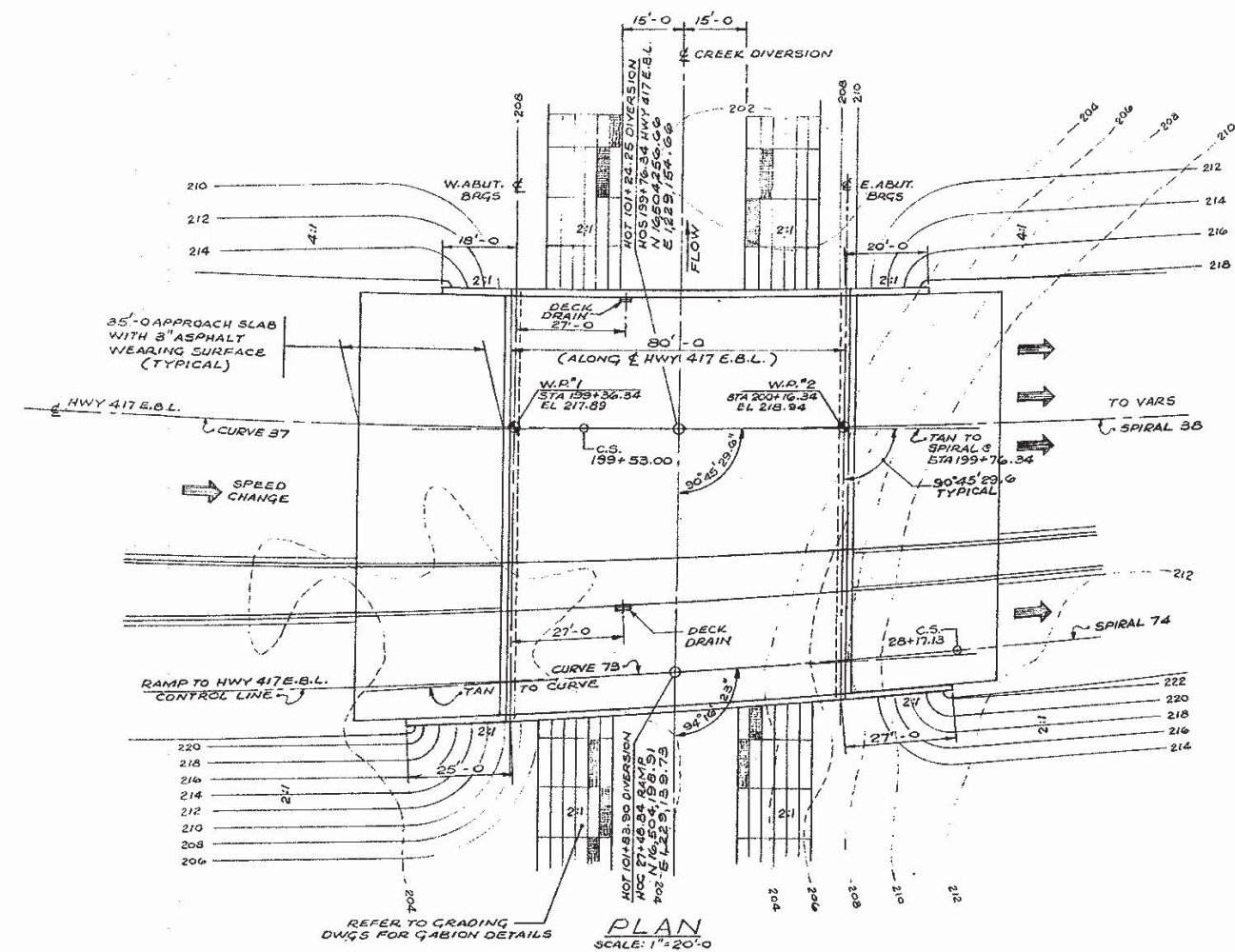


P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

## Attachments

Client: McIntosh Perry  
File No.: 19-3405-3  
E file: 3-311\_1 tel

Date: August 21, 2015  
Page 12



NOTES:  
 • W.P. DENOTES WORKING POINT.  
 • T/P DENOTES TOP OF PAVEMENT.

#### REFERENCE BENCH MARK

BENCH MARK 218.75  
 GEODETIC DATUM  
 CUT CROSS ON N.E. CORNER OF HWY BRIDGE  
 540.0' RT OF 202+15 E.B.L.

#### CURVE DATA

HWY 417 E.B.L.  
 CIRCULAR CURVE 37  
 $\Delta = 59^{\circ}02'22.40''$   
 $D = 2^{\circ}35'43.89''$   
 $R = 2207.483'$   
 $T = 1249.941'$   
 $L = 2274.668'$   
 $E = 329.313'$

SPIRAL 38  
 $\theta_s = 5^{\circ}50'23.75''$   
 $L_s = 450.00'$   
 $L_t = 300.163'$   
 $S_t = 150.149'$

#### RAMP INNES RD TO HWY 417 E.B.L.

CIRCULAR CURVE 73  
 $\Delta = 10^{\circ}00'00.00''$   
 $D = 2^{\circ}30'00.00''$   
 $R = 2291.831'$   
 $T = 200.509'$   
 $L = 400.00'$   
 $E = 8.754'$

SPIRAL 74  
 $\theta_s = 2^{\circ}30'00.00''$   
 $L_s = 200.00'$   
 $L_t = 133.547'$   
 $S_t = 66.679'$

#### LIST OF DRAWINGS

1. GENERAL PLAN.
2. BOREHOLE LOCATION & SOIL STRATA.
3. FOUNDATION LAYOUT & DETAILS.
4. WEST ABUTMENT.
5. EAST ABUTMENT.
6. PRESTRESSED GIRDERS & BEARINGS.
7. DECK DETAILS.
8. CONCRETE BARRIER WALL (2'-8" HIGH).
9. DETAILS OF 9" HIGH STEEL RAILING.
10. APPROACH SLABS.
11. STANDARD DETAILS I.
12. STANDARD DETAILS II.
13. PLAN - EMBEDDED DETAIL.
14. EMBEDDED DETAILS.

#### GENERAL NOTES

##### CLASS OF CONCRETE:

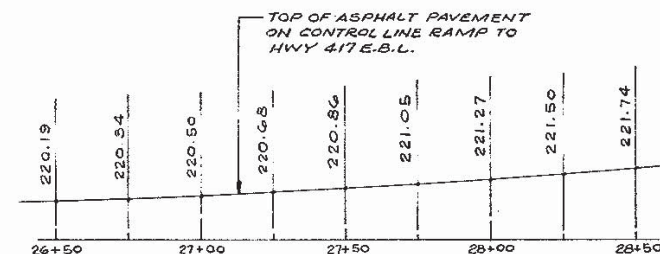
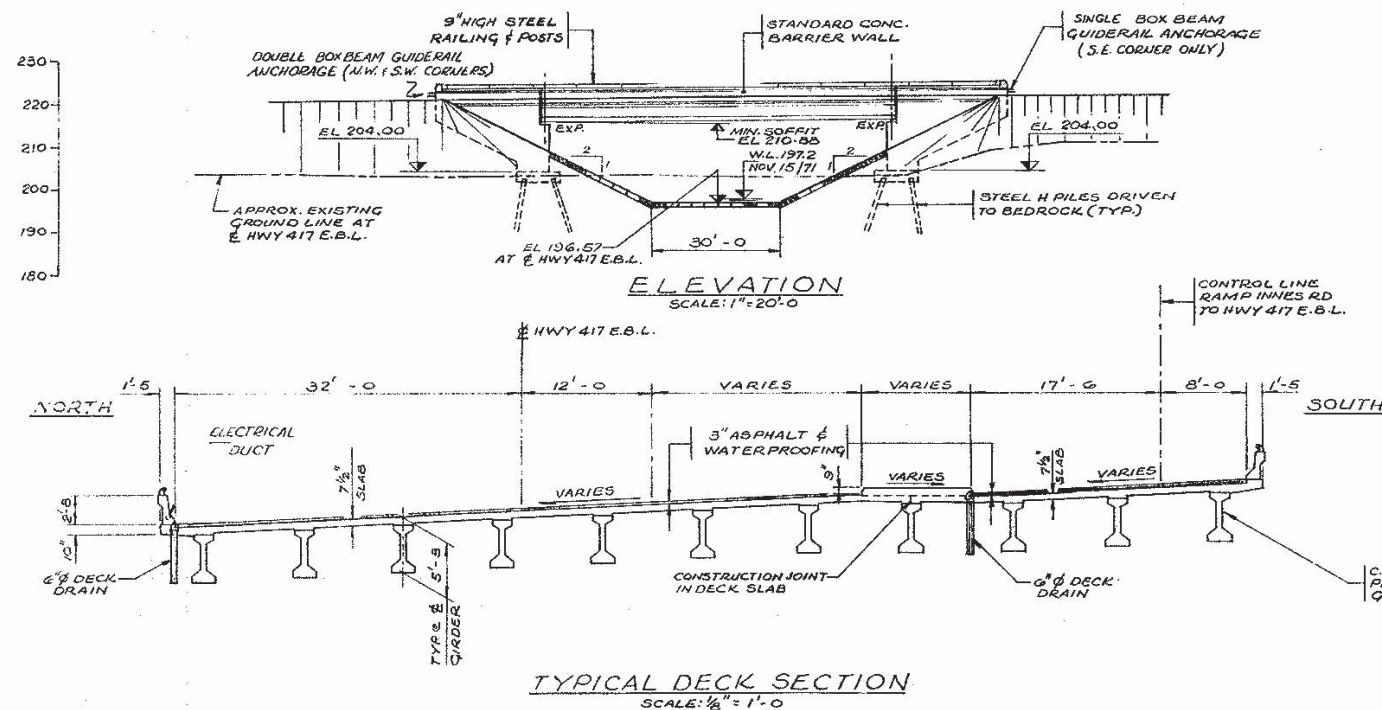
PRESTRESSED CONCRETE GIRDERS - 5000 PSI  
 APPROACH SLABS - 4000 PSI  
 DECK, DIAPHRAGMS, MEDIAN & BARRIER WALLS - 4000 PSI  
 REMAINDER - 3000 PSI

##### CLEAR COVER ON REINFORCING STEEL:

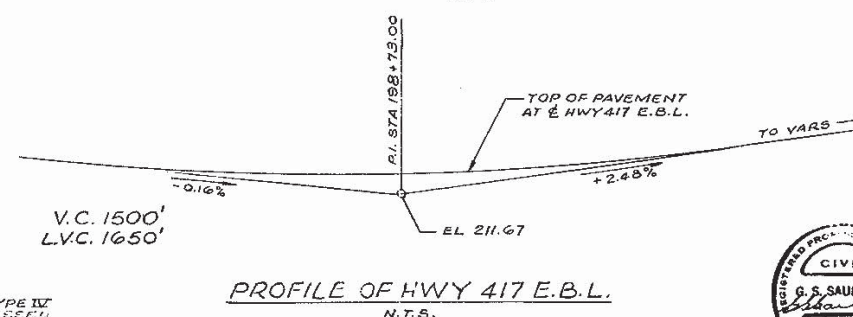
FOOTINGS & ABUTMENTS - 3"  
 DECK: TOP - 1 1/2", BOTTOM - 1", DIAPHRAGMS - 1 1/2"  
 BARRIER WALLS - 1 1/2"  
 APPROACH SLABS & MEDIAN - 2"

##### CONSTRUCTION NOTES

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF  $\pm 1/8"$ .  
 NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.



PROFILE OF RAMP TO HWY 417 E.B.L.  
 (SPLINED)  
 N.T.S.

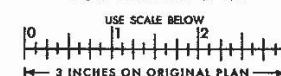


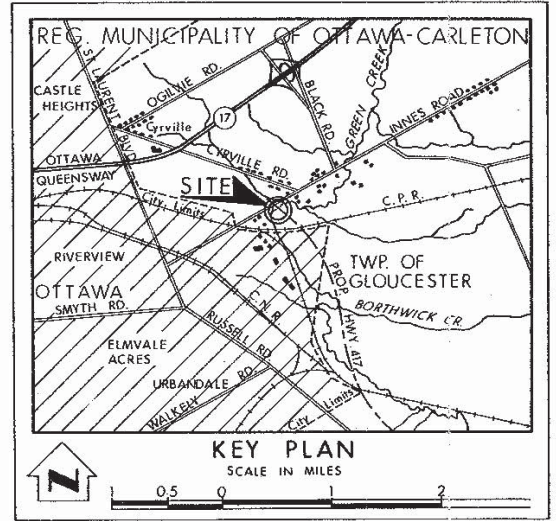
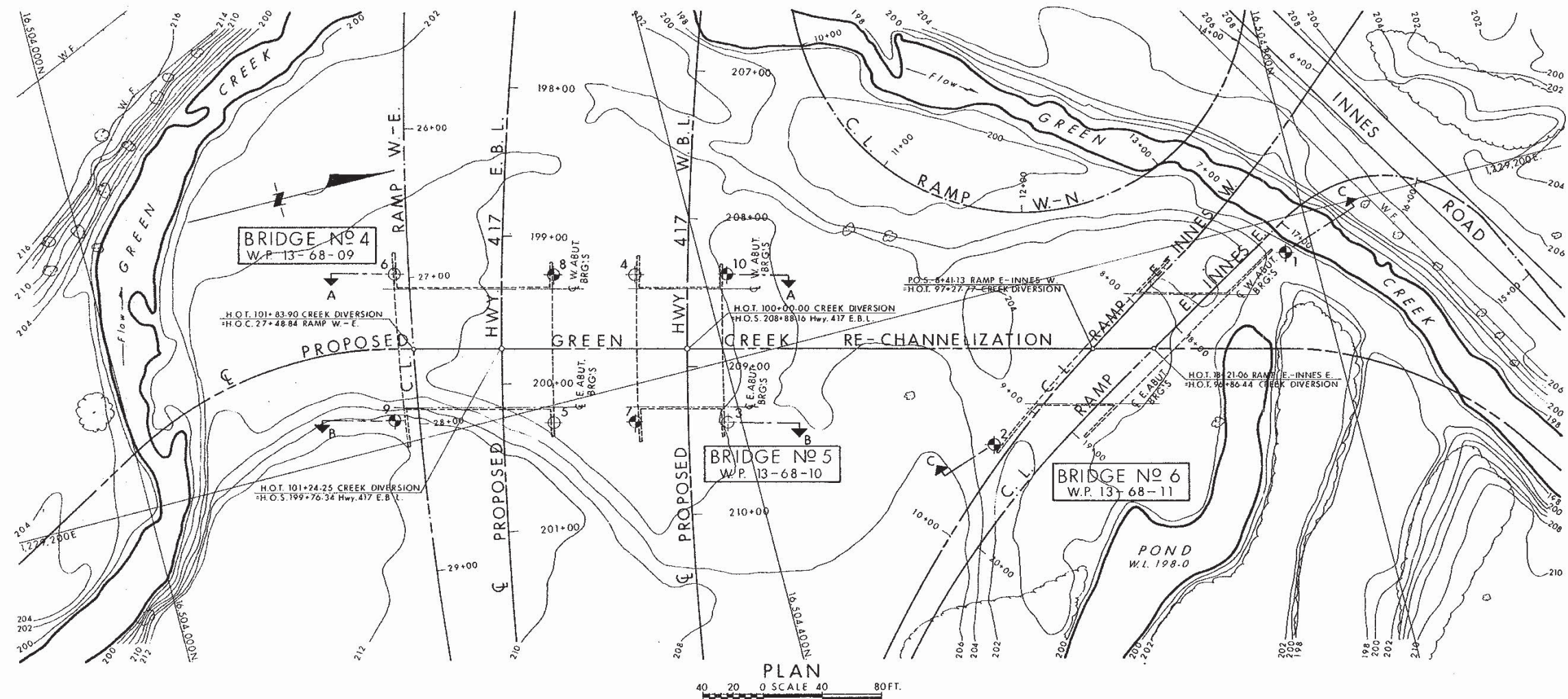
PROFILE OF HWY 417 E.B.L.  
 N.T.S.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO			
De Leuw, Cather ENGINEERS & PLANNERS - OTTAWA			
GREEN CREEK UNDER HWY 417 E.B.L. IMMEDIATELY EAST OF INNES RD. BRIDGE NO. 4			
KING'S HIGHWAY No. 417		DIST. No. 9	
CO. REG. MUNICIPALITY OF OTTAWA-CARLETON			
TWP. GLOUCESTER		LOT 23 CON. III	
GENERAL PLAN			
APPROVED		SITE No. 3-3118	
DESIGN G.S.S. CHECK L.D.H.		CONTRACT No. 73-66-09	
DRAWING K.A.B. CHECK G.S.S.		DRAWING No. 3-3118-1	
DATE APR. '73		LOADING 4520-46	

FOR REDUCED PLAN



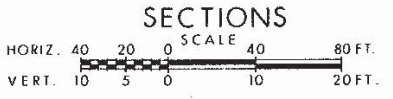
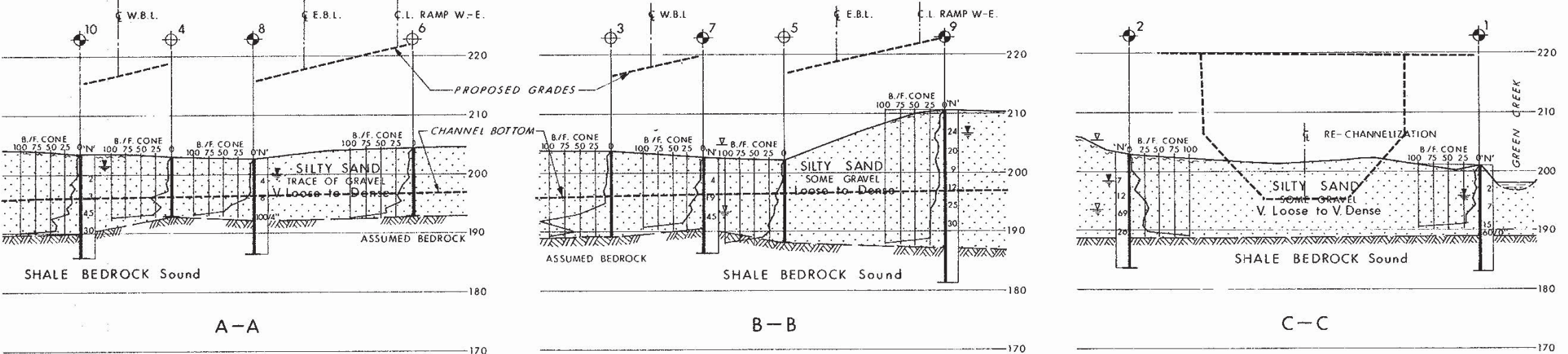


LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation, June 1972		
	Head		
	Artesian Water Levels		
	Encountered		
NO.	ELEVATION	CO - ORDINATES	
		NORTH	EAST
1	201.0	16,504,781	1,229,223
2	202.7	16,504,561	1,229,300
3	203.7	16,504,391	1,229,240
4	203.0	16,504,355	1,229,128
5	202.1	16,504,277	1,229,212
6	204.5	16,504,200	1,229,087
7	202.4	16,504,331	1,229,225
8	202.5	16,504,302	1,229,114
9	210.3	16,504,175	1,229,183
10	203.4	16,504,415	1,229,143

— NOTE —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

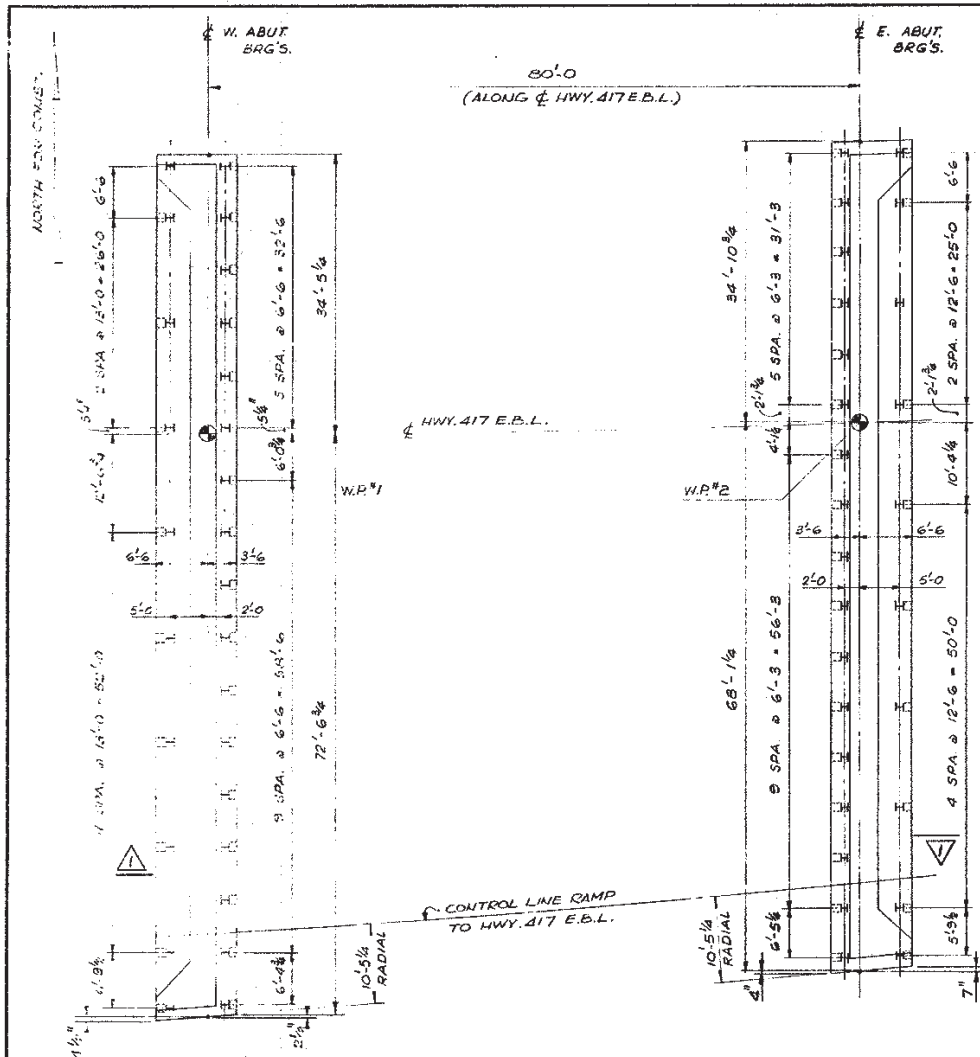
REVISIONS		
DATE	BY	DESCRIPTION



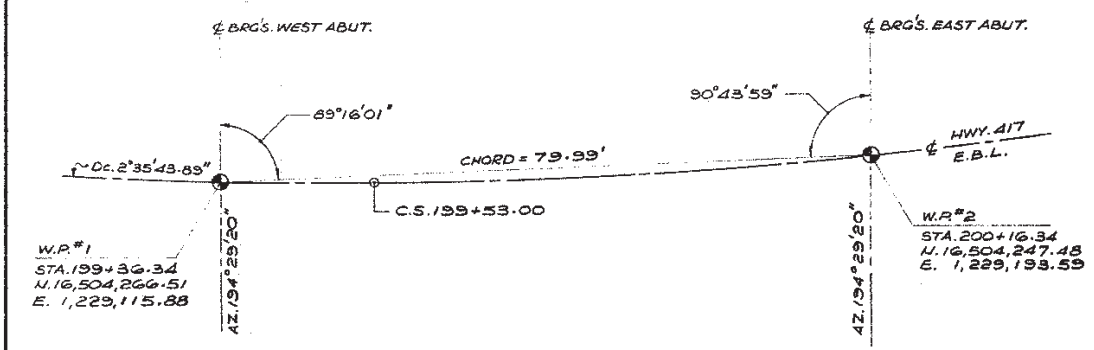
NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the OTTAWA District Office.

REF. NO. B56-34

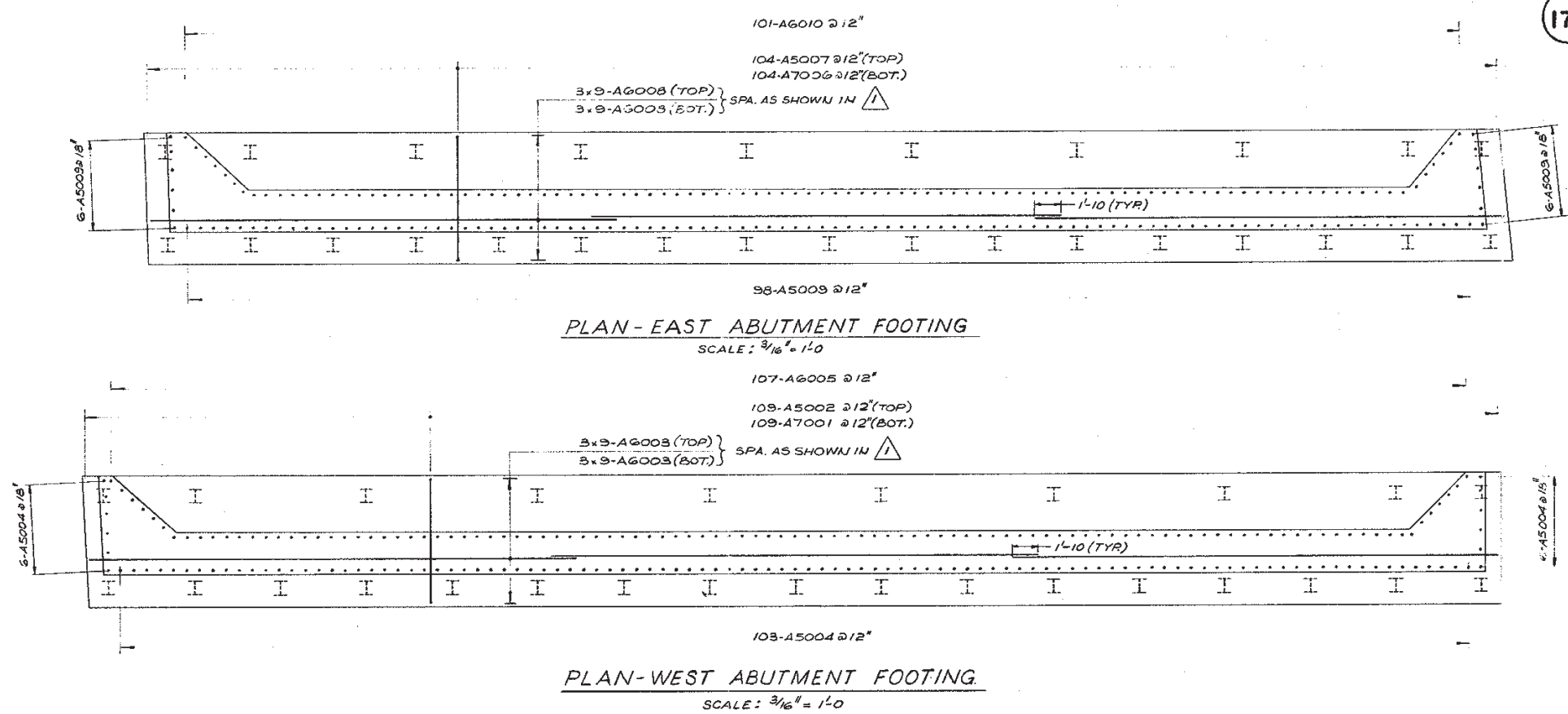
MINISTRY OF TRANSPORTATION & COMMUNICATIONS DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE			
BRIDGE No. 4 GREEN CREEK			
HIGHWAY NO. Prop. 417 E.B.L.		DIST. NO. 9	
REG. MUNICIPALITY OF OTTAWA-CARLETON			
TWP. GLOUCESTER		LOT. CON.	
BORE HOLE LOCATIONS & SOIL STRATA			
SUBMD. S.A.	CHECKED	W.P. NO. 13-68-09	DRAWING NO.
DRAWN	CHECKED	JOB NO. 72-11067	72-11067A
DATE July 27, 1972		SITE NO. 3-311B	BRIDGE DRAWING NO.
APPROVED		CONT. NO. 73-191	3-311B-2
PRINCIPAL FOUNDATION ENGINEER			



PLAN  
SCALE: 3/32" = 1'-0"

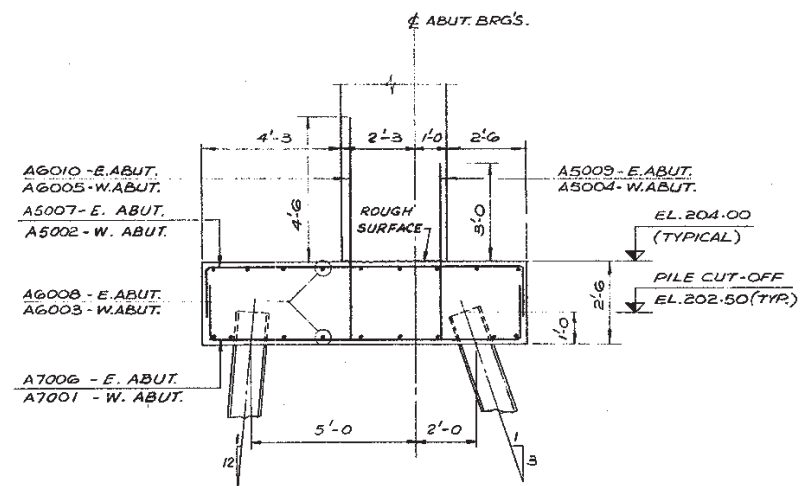


LOCATION OF WORKING POINTS  
N.T.S.



PLAN-EAST ABUTMENT FOOTING

PLAN-WEST ABUTMENT FOOTING



STEEL 'H' PILE DATA			
LOCATION	Nº	LENGTH	TYPE
W. ABUT.	27	14'-0"	HP 12 x 74
E. ABUT.	27	18'-0"	(TYR)

- NOTES**
1. PILES TO BE DRIVEN TO BEDROCK.
  2. SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTING.

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO

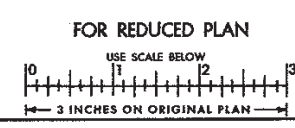
**DeLeuw, Cather**  
ENGINEERS & PLANNERS - OTTAWA

**GREEN CREEK UNDER HWY. 417 E.B.L.**  
IMMEDIATELY EAST OF WINNES ROAD,  
BRIDGE Nº 4

KING'S HIGHWAY Nº. 417 DIST. No. 9  
CO. REG. MUNICIPALITY OF OTTAWA-CARLETON  
TWP. GLOUCESTER LOT 23 CON. III

**FOUNDATION LAYOUT & DETAILS**

APPROVED \_\_\_\_\_ CONTRACT No. 73-191  
DESIGN L.D.H. CHECK A.G. W.P. No. 13-G-3-C  
DRAWING G.C. CHECK G.S.S. SITE No. 3-3113 SHEET 3  
DATE APR 73 LOADING 1/2.20-44



# 2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836  
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: ,

May 05, 2015

Site Coordinates: 45.4131 North 75.6073 West

User File Reference: Green Creek

## National Building Code ground motions:

**2% probability of exceedance in 50 years (0.000404 per annum)**

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.635	0.309	0.138	0.046	0.323

**Notes.** Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. *These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.*

## Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.090	0.249	0.386
Sa(0.5)	0.043	0.123	0.187
Sa(1.0)	0.017	0.056	0.088
Sa(2.0)	0.006	0.018	0.028
PGA	0.039	0.123	0.201

## References

**National Building Code of Canada 2010 NRCC no. 53301**; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3

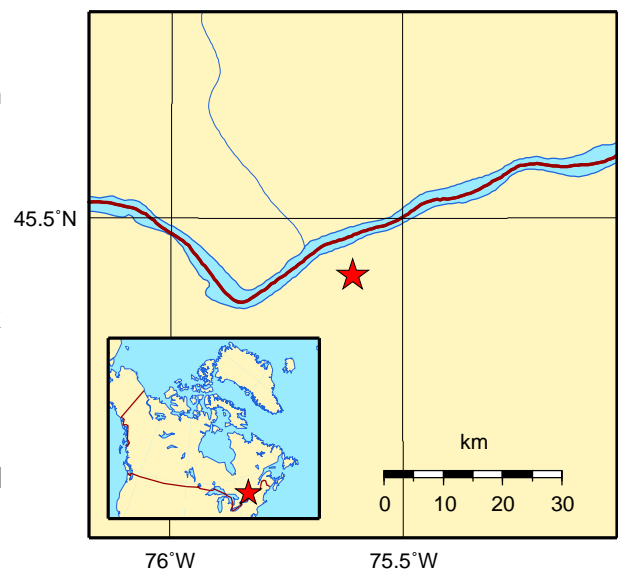
**Appendix C:** Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

**User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543** (in preparation)  
**Commentary J:** Design for Seismic Effects

**Geological Survey of Canada Open File xxxx**  
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites [www.EarthquakesCanada.ca](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information

Aussi disponible en français





**APPENDIX 7**  
**SITE 3-311/2**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Kenton Power, P.Eng.  
(Reviewed by Fred Griffiths, P.Eng. and  
P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 WESTBOUND BRIDGE OVER GREEN CREEK (SITE 3-311/2) GWP 4074-11-00 GEOCRE 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing overpass structure carrying westbound traffic on Highway 417 over Green Creek in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4022-11-01
- Site No. 3-311/2
- GEOCRE No. 31G5-85
- Construction Contract 73-191
- Historic W.P. 13-68-10

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa, in the Township of Gloucester approximately 230 m south-east of the Highway 417 / Innes Road Interchange. The single span structure, carries the westbound lanes of Highway 417 (two lanes in total plus paved shoulders) over Green Creek. Prior to construction of the bridge Green Creek flowed from south to north at a location further to the west. The creek was diverted to an area of relatively flat ground with an approximate elevation of 62.5 m.

Based on the historic General Plan Drawing (copy attached) the bridge is approximately 17.4 m wide, and 24.4 m long with a concrete pre-stressed girder structure. Both of the bridge abutments were to be supported by steel HP12x74 piles driven to bedrock.



### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-85 dated August 1972. The investigation consisted of two sampled boreholes designated 7 and 10; both accompanied by dynamic cone penetration tests. Two additional dynamic cone penetration tests designated 3 and 4 were also carried out opposite both Boreholes 7 and 10. Drawing No. 72-11067A (copy attached) illustrates the locations of the bridge and the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by a silty sand with occasional clayey silt seams underlain by shale bedrock.

#### **3.1 Silty Clay**

A silty clay stratum was encountered at the ground surface of Borehole 10. The surface of this deposit was at elevation 62.0 m with a thickness of 2.9 m. The standard penetration test (SPT) 'N' values range from 2 to 3 blows per 0.3 m of penetration, indicating a soft to firm consistency.

The results of a grain size analysis test including hydrometer testing completed on a sample of this material indicated a gravel content of 1%, sand content of 27%, silt content of 51%, and clay content of 21%. Atterberg Limits test was carried out on the fines within this material indicate a silty clay of low plasticity.

The moisture content of the sample tested was 26%.

#### **3.2 Silty Sand with Gravel**

A silty sand stratum with varying amounts of gravel was encountered at the ground surface of Borehole 7 and beneath the silty clay stratum in Borehole 10. The stratum contained clayey silt seams though the thickness of the seams was not indicated. The surface of this deposit was at 61.7 m and 59.1 m in elevation with a thickness from 1.2 m and 3.5 m. SPT 'N' values range from 4 to 45 blows per 0.3 m of penetration, indicating a loose to dense condition but typically dense.

The results of a grain size analysis including one with hydrometer testing completed on two samples of this material indicated a gravel content between 14% and 41%, sand content between 46% and 50%, and a fines content (combined silt and clay content) between 13% and 36%. Atterberg Limits tests carried out on material from the clayey silt seams indicated a clay of low plasticity.

The moisture content of the samples tested were 9% and 26%.

#### **3.3 Bedrock**

A grey shale bedrock was encountered beneath the silty sand stratum in both Boreholes 7 and 10 as proven by BX size coring; and DCPT refusal on assumed bedrock in Boreholes 3 and 4. The bedrock surface elevation ranged from 57.9 m to 58.2 m.



Bedrock core recovery was 100% in both boreholes. The bedrock was described to be in sound condition. Geological mapping suggests that this site is near the boundary between the Billings and Carlsbad Formations.

### **3.4 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 58.9 m to 61.4 m. An artesian groundwater condition with a head of 0.8 m was encountered in Borehole 7 once the borehole penetrated the lower granular silty sand stratum at elevation 59.1 m.

The water level of Green Creek at the time of the investigation is indicated on the General Plan Drawing as 60.2 m.

## **4 SITE OBSERVATIONS**

A detailed deck condition survey was carried out by exp. Services Inc. in June 2012 for Bridge 3-311/2 with the report issued September 2013. Condition data outlined in the report for the bridge structure ranged from fair to good but typically the bridge was rated in good condition.

The site was inspected by Thurber Engineering Ltd. staff during the week of July 14, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

North and South Abutments:

- Gabions were installed at the site for erosion protection of the abutment slopes
- Vegetation growth over the gabions was noted
- Abutment slopes were measured at approximately 2H:1V (Horizontal:Vertical)
- Abutment slopes were noted to be in generally good condition with no signs of erosion or settlement
- No obvious signs of scouring by creek flow or erosion of the toe of the abutment slopes were observed
- Vegetation on the embankment side slopes was noted with no obvious signs of settlement or erosion
- Rust staining and spalling of the concrete was noted on the abutment walls

Bridge and Road Surface:

- Concrete spalling of the barriers and sides of the bridge deck was noted
- Frequent longitudinal and transverse cracking of the asphalt surface was noted on both the bridges and approaches
- Some potholes, patching and crack repair of the asphalt surface were noted on both the bridges and approaches
- A slight dip in the pavement before the abutments was observed



## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

Where insufficient earth cover is provided, polystyrene insulation may be used to enhance frost protection.

#### **5.2 Seismic Considerations**

This site is best classified as a Soil Profile Type I in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC).

The site specific peak ground acceleration (PGA) value with a 10% probability of exceedance in 50 Years of 0.123 was used in the evaluation of seismic considerations. The PGA value was obtained from the Natural Resources Canada 2010 National Building Code (NBC) Seismic Hazard Calculator obtained May 2015. A copy of the NBC Seismic Hazard Calculation Data sheet is attached for reference.

The susceptibility to liquefaction of the loose silty sand and the soft to firm silty clay at this site has been evaluated using the site specific PGA value obtained from the 2010 NBC. Based on existing information these soils were found to be not susceptible to liquefaction during the design earthquake event, with Factors of Safety over 1.0.

#### **5.3 Existing Foundations**

As per the Foundation Layout and Details Drawing the bridge abutments were designed to be supported on steel HP12x74 piles driven to bedrock. The available construction drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the design piles is 95 tons / HP12x74 pile or approximately 845 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN/pile for HP310x110 piles driven to bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

### **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary



recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

## **6.1 Shallow Foundations**

Although spread footings within the overburden are possible, effective dewatering of the sand and gravel strata would be required. Given the anticipated relatively shallow depth to the shale bedrock, and the higher geotechnical resistance offered by the bedrock, spread footings within the overburden are not recommended.

The factored vertical geotechnical resistance of 1500 kPa at ULS may be used for the preliminary design of shallow foundations, founded on or in sound shale bedrock. The SLS condition will not govern for footings in or on the bedrock. The design of any new foundations on bedrock would need to consider interactions with the existing foundations and potential for undermining of the existing structural elements.

Resistance to lateral forces and sliding resistance between concrete and underlying materials should be evaluated using an unfactored coefficient of friction of 0.50 for cast-in-place concrete and 0.45 for pre-cast concrete on sound shale bedrock.

Based on current geological mapping, the shale present at this site may be from the Billings Formation which is susceptible to heaving if allowed to weather in the presence of oxygen. The general mechanism is that oxidation of pyrite within the shale produces sulfuric acid, which in turn reacts with calcite in the shale to form gypsum crystals, which occupy a larger volume than the original materials. A by-product of this chain of reactions also tends to increase sulphate levels which can attack buried concrete structures.

The potentially detrimental effects of shale heaving can be avoided by preventing exposure of the shale to oxygen both during construction and long term. This is typically achieved by limiting exposure of the shale to no more than one day prior to covering with a protective layer such as shotcrete or a concrete mud slab. Sulphate resistant cement should be used in such applications.

## **6.2 Deep Foundation – Piles**

Based on the review of existing subsurface stratigraphy at the site driven piles are considered to be suitable for the support of abutment foundations perched within the approach embankments. It should be noted that the bedrock surface elevation ranges from 57.9 m to 58.2 m.

### **6.2.1 Axial Resistance**

Steel piles (Grade 350 W steel) end-bearing on sound bedrock at this site may be designed on the basis of the following factored, vertical, geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.



### 6.2.2 Pile Tips

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### 6.2.3 Integral Abutment Considerations

As per the Foundation Layout and Details Drawing each of the existing abutments are supported by a pile group that includes two rows of piles battered in opposite directions to each other. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### 6.2.4 Pile Spacing

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### 6.2.5 Downdrag

In light of the cohesionless makeup of the existing overburden soils at this site, downdrag on existing and new piles is not considered a design issue.

### 6.2.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formula for cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where:

$k_h$  = coefficient of horizontal subgrade reaction

$n_h$  = coefficient related to soil density given in Table A

$z$  = depth of pile embedment (m)

$B$  = pile width perpendicular to load direction (m)

**Table A:**  $n_h$  values for cohesionless soils.

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 62	Embankment Fill	3,000
Below 62	Native Silty Sand	2,000

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B below. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.2.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)

**Table C:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 62	Embankment Fill	20	0.4
Below 62	Native Silty Sand	19	0.4

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

### 6.3 Static Lateral Earth Pressure Coefficients

Existing and anticipated lateral earth pressures acting on the structure should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

The coefficient of lateral earth pressure at-rest ( $K_0$ ) should be used unless the abutment walls can rotate enough to fully mobilize the active condition. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D: Static Lateral Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Silty Sand	Silty Clay
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19	17
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$	$27^\circ$
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.5	0.55
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33	0.38

#### 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Silty Sand	Silty Clay
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19	17
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$	$27^\circ$
<b>Yielding Wall</b>			
$K_{AE}$	0.33	0.40	0.45
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36
<b>Non-Yielding Wall</b>			
$K_{AE}$	0.55	0.66	0.74
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$



where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors provided in Table E are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## **6.5 Approach Embankments**

The General Plan Drawing indicates that the maximum height of the approach fills was approximately 6.4 m from stream bed to approach fill with an embankment slope of 2H:1V. Design drawings indicate that the abutment backfill is to consist of compacted boulder free fill. The Foundation Design Report states that settlement of the approach fills will be elastic in nature and negligible after the completion of the fills. The embankment foundation is expected to be stable and no long term settlement problems are expected unless the fills are reconfigured

## **6.6 Erosion Control**

The gabions should be maintained to prevent erosion and scouring of the abutment slopes.

## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native overburden soils reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.



Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

Based on a preliminary assessment of existing data the native materials at this site were found to be not susceptible to liquefaction during the design earthquake event. During detailed design the potential for and the implications of liquefaction should be confirmed. This may include drilling foundation boreholes in the approaches to determine the condition of the embankment fill and underlying native materials.

During detailed design, the need for vibration monitoring will also need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one.

Should the proposed works result in excavations to bedrock, samples of the bedrock should be acquired during the investigation and submitted for chemical testing to determine the pyritic heave potential.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.



Kenton C. Power, P.Eng.  
Geotechnical Engineer



Fred Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

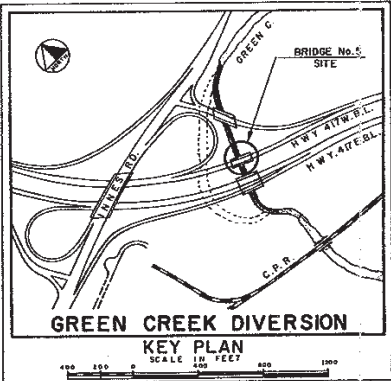
## Attachments

Client: McIntosh Perry  
File No.: 19-3405-3  
E file: 3-311\_2 tel

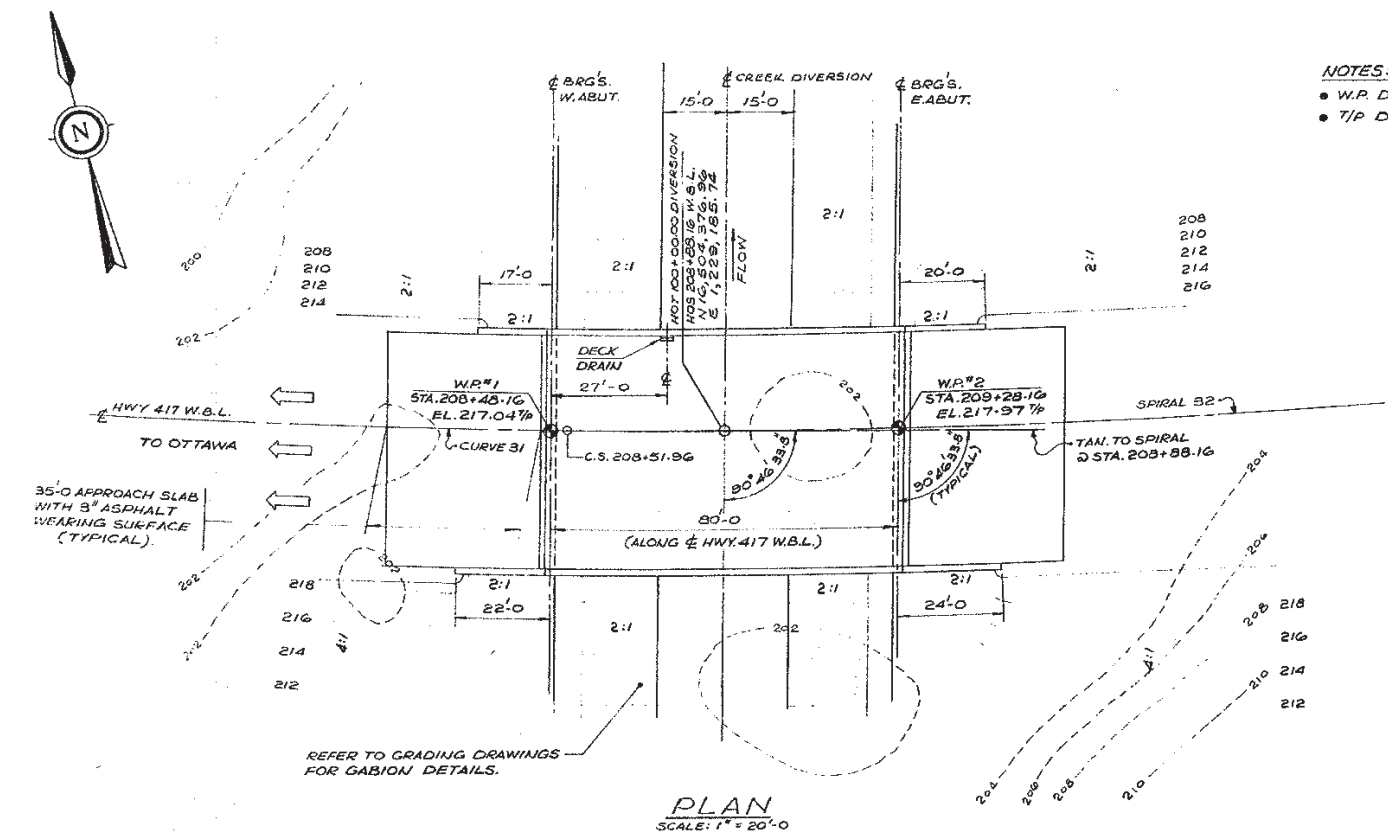
Date: August 21, 2015  
Page 12

REFERENCE BENCH MARK  
BENCH MARK 218.75  
GEODETIC DATUM  
CUT CROSS ON N.E. CORNER OF RWY BRIDGE  
540.0' RT OF 202+15 E.B.L.

CURVE DATA  
HWY 417 W.B.L.  
CIRCULAR CURVE 31 SPIRAL 32  
 $\Delta = 61^\circ 56' 13.00''$   $\theta S = 6^\circ 11' 15.00''$   
 $D = 2^\circ 45' 00.00''$   $L_S = 450.00'$   
 $R = 2083.483'$   $LT = 300.183'$   
 $T = 1250.523'$   $ST = 150.187'$   
 $L = 2252.253'$   
 $E = 346.375'$



NOTES:  
• W.P. DENOTES WORKING POINT.  
• T.P. DENOTES TOP OF PAVEMENT.



- LIST OF DRAWINGS
1. GENERAL PLAN.
  2. BOREHOLE LOCATIONS & SOIL STRATA.
  3. FOUNDATION LAYOUT & DETAILS.
  4. WEST ABUTMENT.
  5. EAST ABUTMENT.
  6. PRESTRESSED GIRDERS & BEARINGS.
  7. DECK DETAILS.
  8. CONCRETE BARRIER WALL (2'-8" HIGH).
  9. DETAILS OF 9" HIGH STEEL RAILING.
  10. APPROACH SLABS.
  11. STANDARD DETAILS I.
  12. STANDARD DETAILS II.
  13. PLAN - EMBEDDED DETAIL.
  14. EMBEDDED DETAILS.

GENERAL NOTES

CLASS OF CONCRETE:

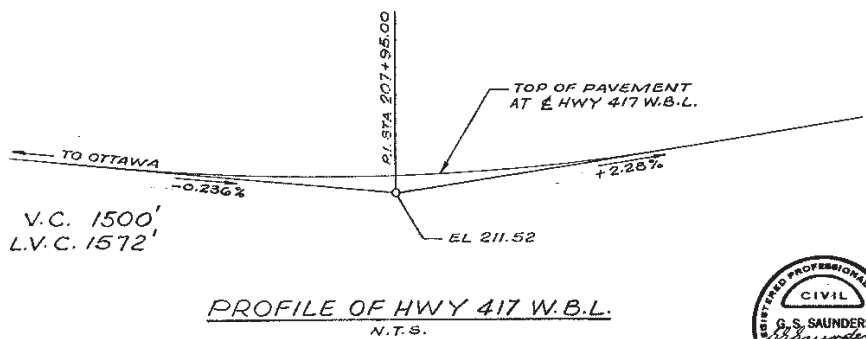
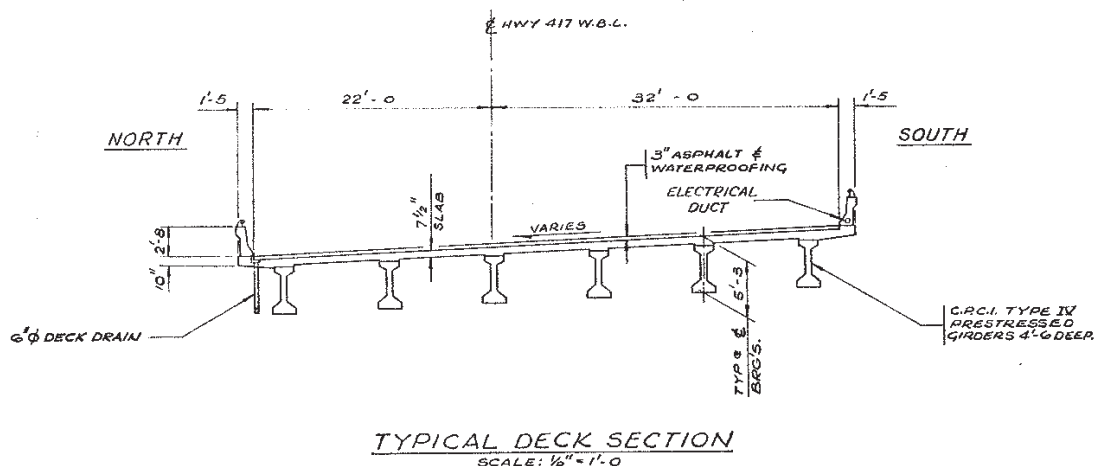
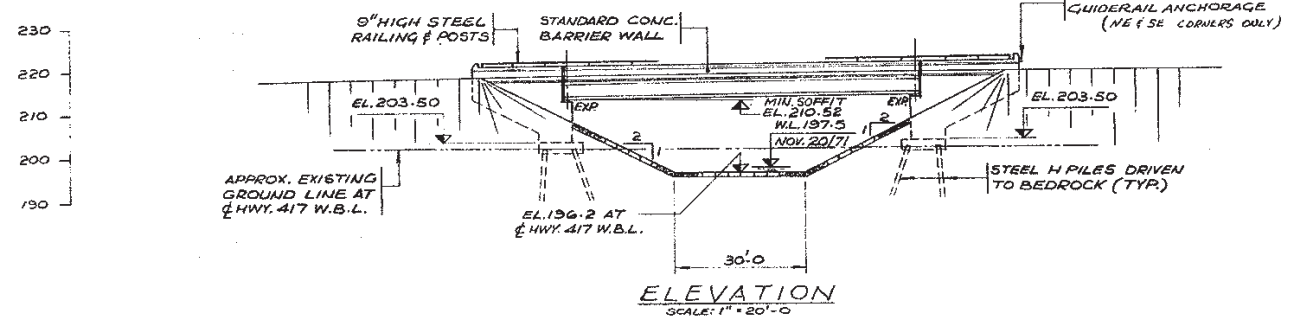
PRESTRESSED CONCRETE GIRDERS	-5000 I. S.I.
APPROACH SLABS	-4000 I. S.I.
DECK, DIAPHRAGMS & BARRIER WALLS	-4000 I. S.I.
REMAINDER	-3000 I. S.I.

CLEAR COVER ON REINFORCING STEEL:

FOOTINGS & ABUTMENTS	-3"
DECK: TOP - 1 1/2", BOTTOM - 1", DIAPHRAGMS - 1 1/2"	
BARRIER WALLS	-1 1/2"
APPROACH SLABS	-2"

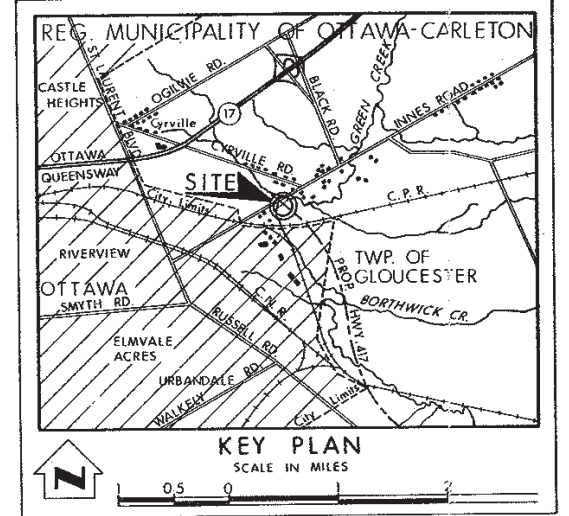
CONSTRUCTION NOTES

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF  $\pm 1/8"$ . NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.



REVISIONS	
DATE	DESCRIPTION
DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO <b>De Leuw, Cather</b> ENGINEERS & PLANNERS - OTTAWA <b>GREEN CREEK UNDER HWY 417 W.B.L.</b> IMMEDIATELY EAST OF INNES RD. BRIDGE No. 5 KING'S HIGHWAY No. 417 DIST. No. 9 CO. REG. MUNICIPALITY OF OTTAWA-CARLETON TWP. GLOUCESTER LOT 23 CON. III <b>GENERAL PLAN</b> APPROVED: <b>3-311A</b> W.P. No. <b>13-68-10</b> DESIGN: <b>G.S.S.</b> CHECK: <b>L.O.H.</b> CONTRACT: <b>13-68-10</b> DRAWING: <b>K.A.B.</b> CHECK: <b>G.S.S.</b> DRAWING: <b>3-311A-1</b> DATE: <b>APR. '73</b> LOADING: <b>HS20-44</b>	





LEGEND				
	Bore Hole			
	Cone Penetration Test			
	Bore Hole & Cone Test			
	Water Levels established at time of field investigation, June 1972			
	Head			
	Artesian Water Levels			
	Encountered			
NO.	ELEVATION	CO - ORDINATES		
		NORTH	EAST	
1	201.0	16,504,781	1,229,223	
2	202.7	16,504,561	1,229,300	
3	203.7	16,504,391	1,229,240	
4	203.0	16,504,355	1,229,128	
5	202.1	16,504,277	1,229,212	
6	204.5	16,504,200	1,229,087	
7	202.4	16,504,331	1,229,225	
8	202.5	16,504,302	1,229,114	
9	210.3	16,504,175	1,229,183	
10	203.4	16,504,415	1,229,142	

**NOTE**

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

DATE	BY	DESCRIPTION

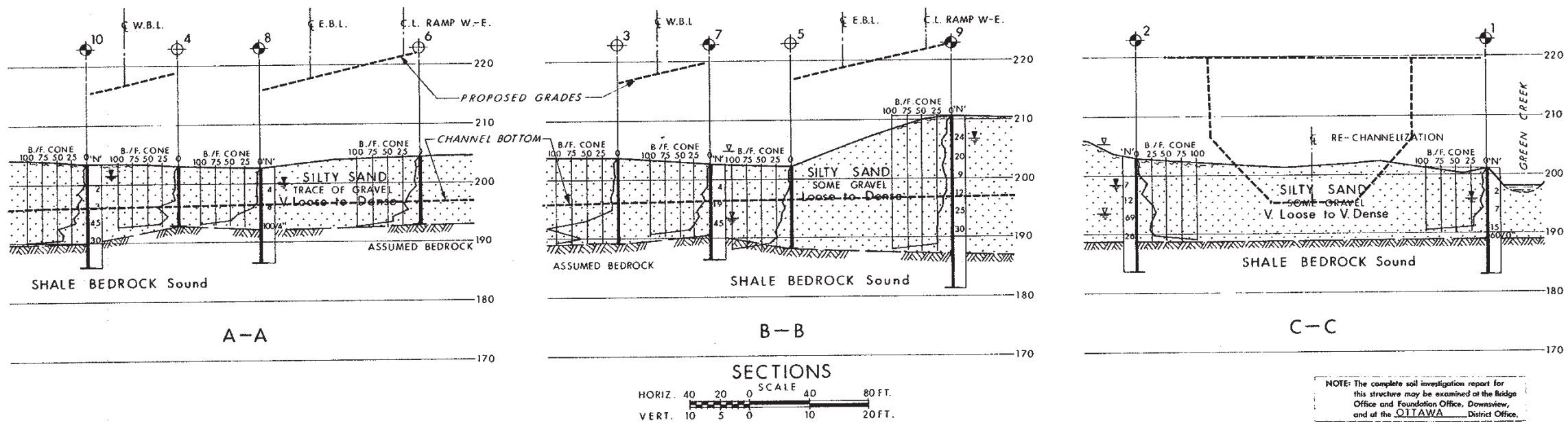
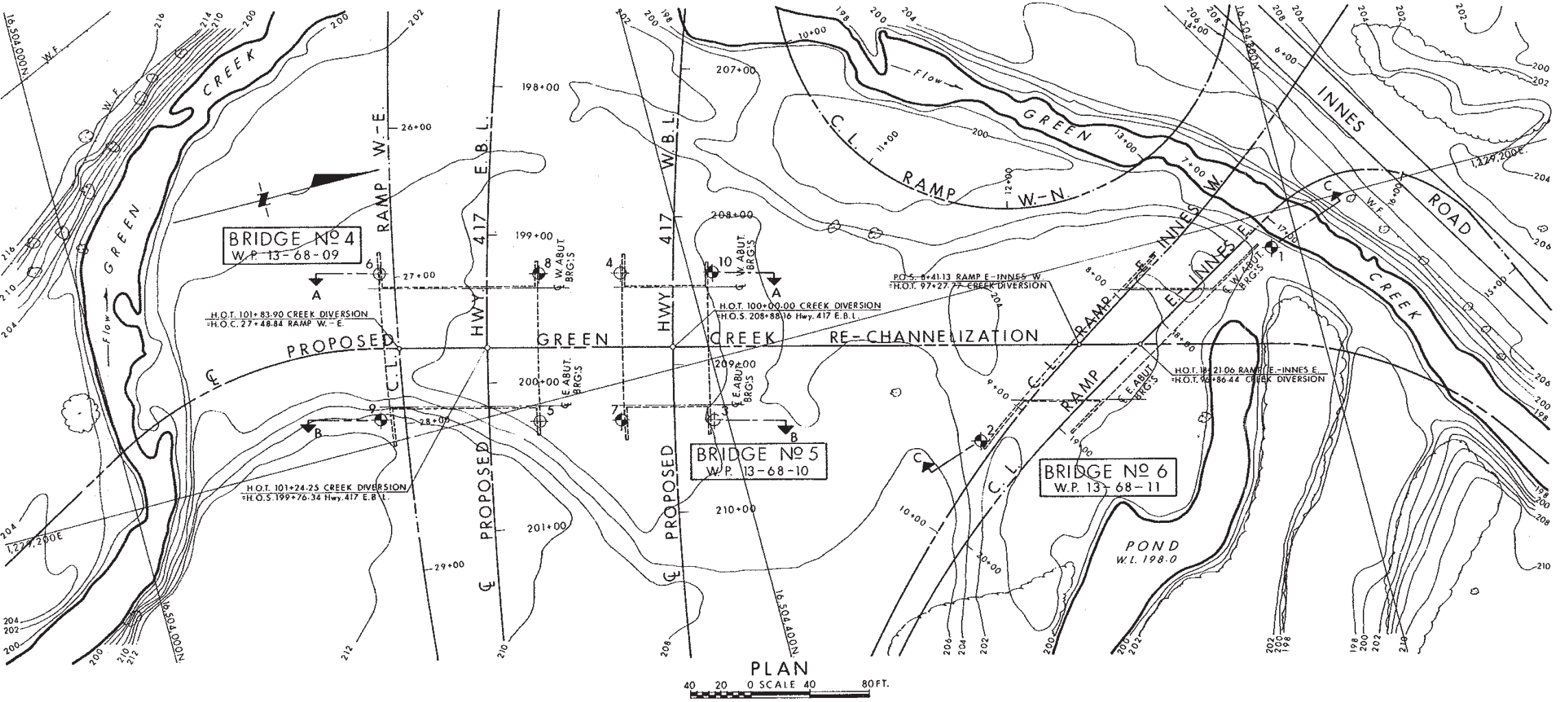
MINISTRY OF TRANSPORTATION & COMMUNICATIONS  
DESIGN SERVICES BRANCH - FOUNDATIONS OFFICE

**BRIDGE No. 5  
GREEN CREEK**

HIGHWAY NO. Prop. 417 W.B.L. DIST. NO. 9  
REG. MUNICIPALITY OF OTTAWA-CARLETON  
TWP. GLOUCESTER LOT CON.

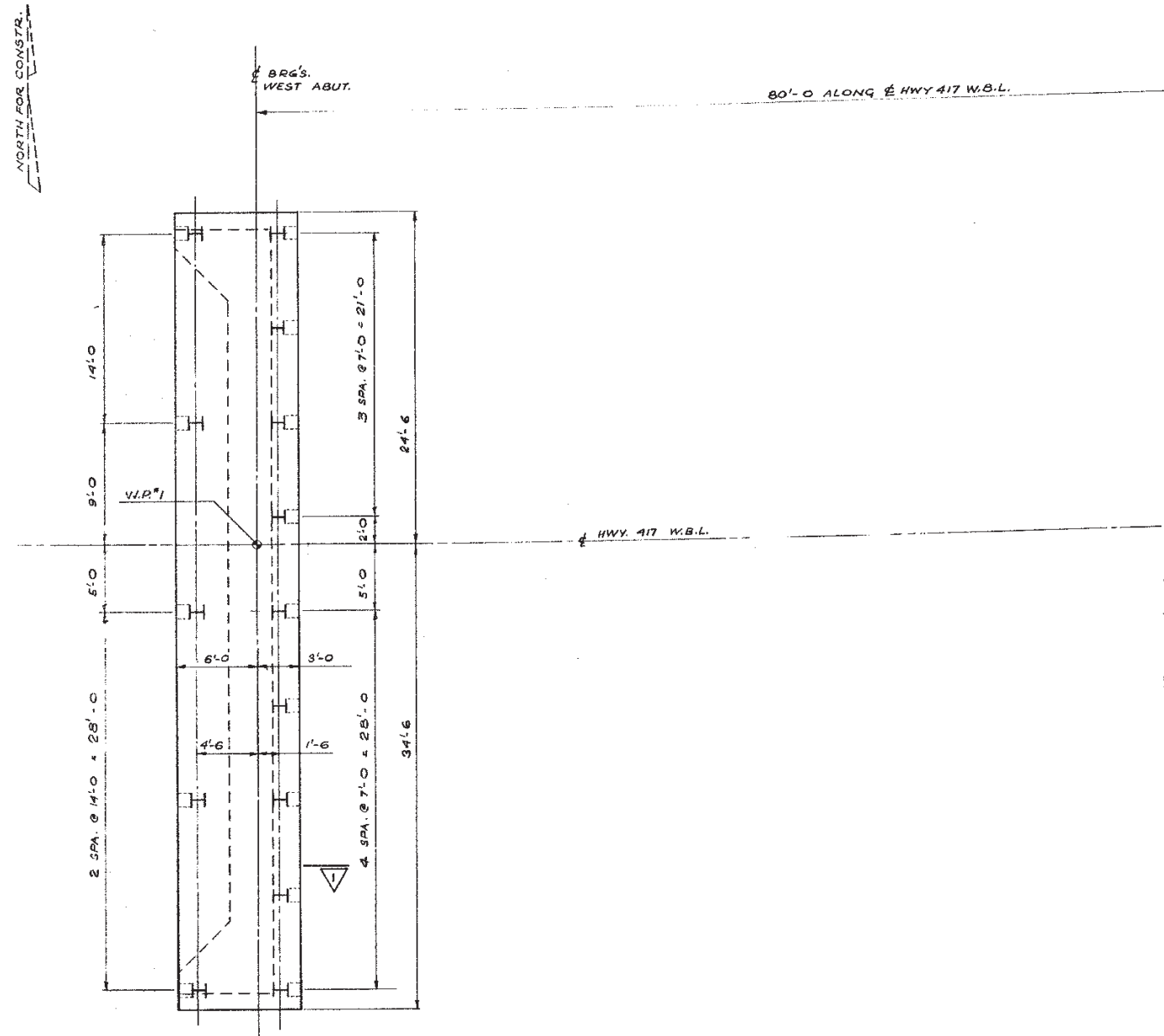
**BORE HOLE LOCATIONS & SOIL STRATA**

SUBMD. S.A. CHECKED <input checked="" type="checkbox"/>	W.P. NO. 13-68-10	DRAWING NO. <b>72-11067A</b>
DRAWN <input checked="" type="checkbox"/>	JOB NO. 72-11067	BRIDGE DRAWING NO. <b>3-311A-2</b>
DATE July 27, 1972	SITE NO. 3-311A	
APPROVED	CONT. NO. 72-191	



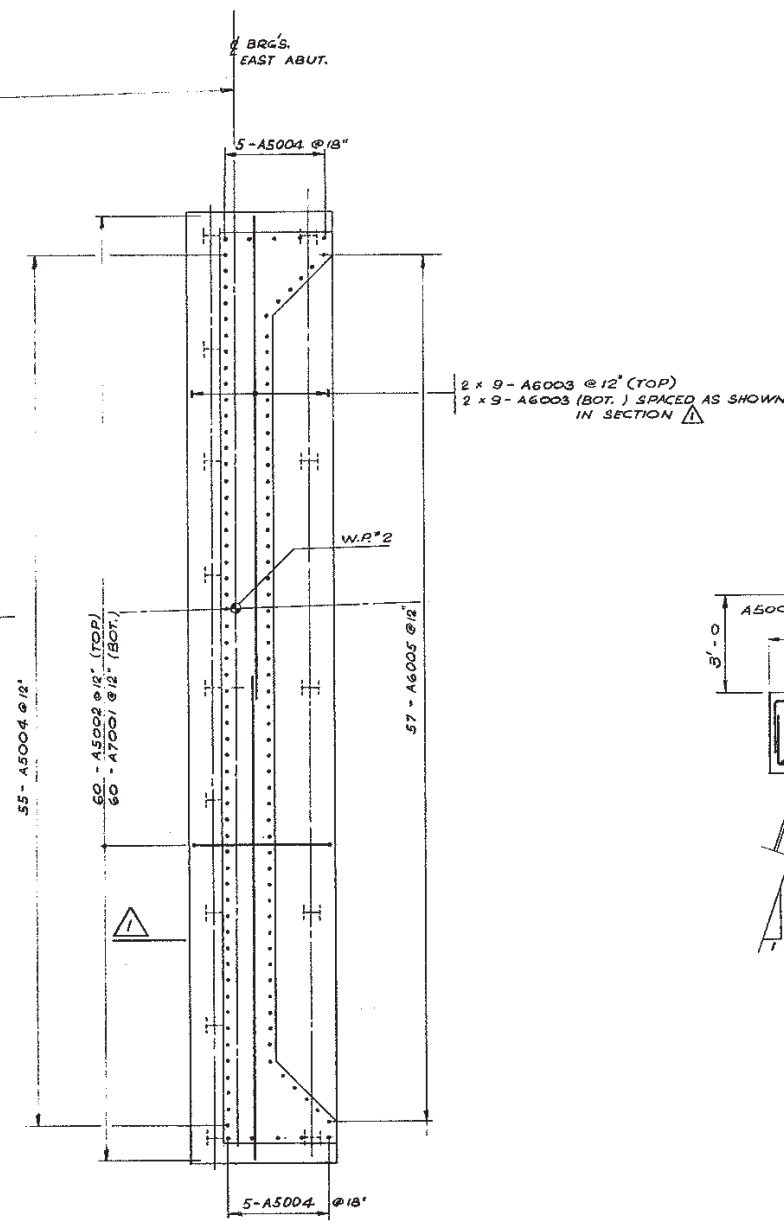
NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the OTTAWA District Office.

REF. NO. B56-34



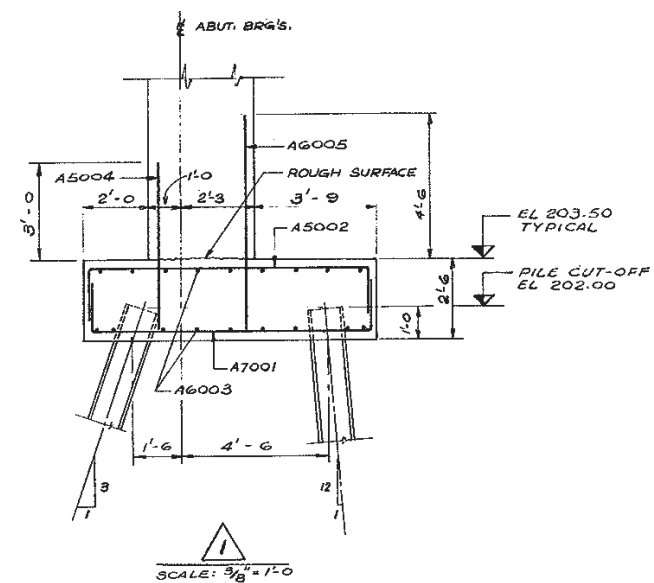
DIMENSIONS & PILE SPACING

PLAN  
FOOTINGS SIMILAR FOR BOTH ABUTMENTS  
SCALE: 3/16" = 1'-0"



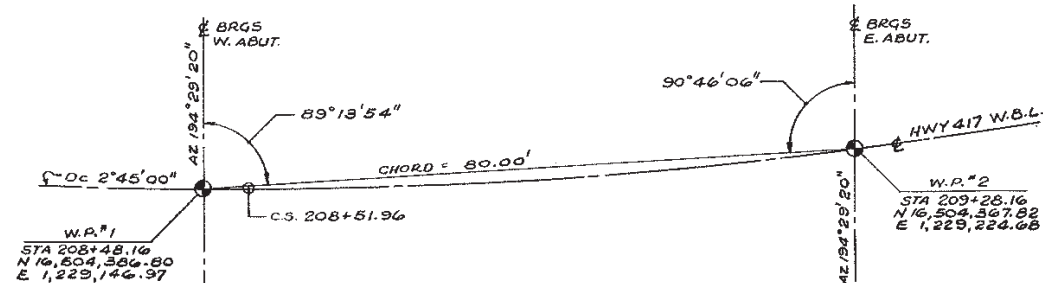
REINFORCEMENT

STEEL 'H' PILE DATA			
LOCATION	NO	LENGTH	TYPE
W. ABUT.	14	16'-0	HP 12 x 74 (1" YR)
E. ABUT.	14	16'-0	



SCALE: 3/8" = 1'-0"

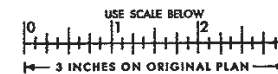
- NOTES:
1. PILES TO BE DRIVEN TO BEDROCK
  2. SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTING.



LOCATION OF WORKING POINTS  
N.T.S.



FOR REDUCED PLAN



REVISIONS	
DATE	DESCRIPTION
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
De Leuw, Cather ENGINEERS & PLANNERS - OTTAWA	
GREEN CREEK UNDER HWY 417 W.B.L. IMMEDIATELY EAST OF INNES RD. BRIDGE NO 5	
KING'S HIGHWAY No. 417	DIST. No. 9
CO. REG. MUNICIPALITY OF OTTAWA-CARLETON	
TWP. GLOUCESTER	LOT 23 CON. III
FOUNDATION LAYOUT & DETAILS	
APPROVED	CONTRACT No. 73-191
DESIGN L.D.H. CHECK A.G.	W.P. No. 13-68-10
DRAWING F.Z. CHECK G.S.S.	SITE No. 3-311A SHEET 3
DATE APR. '73	LOADING HS20-44

# 2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836  
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: ,

May 05, 2015

Site Coordinates: 45.4131 North 75.6073 West

User File Reference: Green Creek

## National Building Code ground motions:

**2% probability of exceedance in 50 years (0.000404 per annum)**

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.635	0.309	0.138	0.046	0.323

**Notes.** Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. *These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.*

## Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.090	0.249	0.386
Sa(0.5)	0.043	0.123	0.187
Sa(1.0)	0.017	0.056	0.088
Sa(2.0)	0.006	0.018	0.028
PGA	0.039	0.123	0.201

## References

**National Building Code of Canada 2010 NRCC no. 53301**; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3

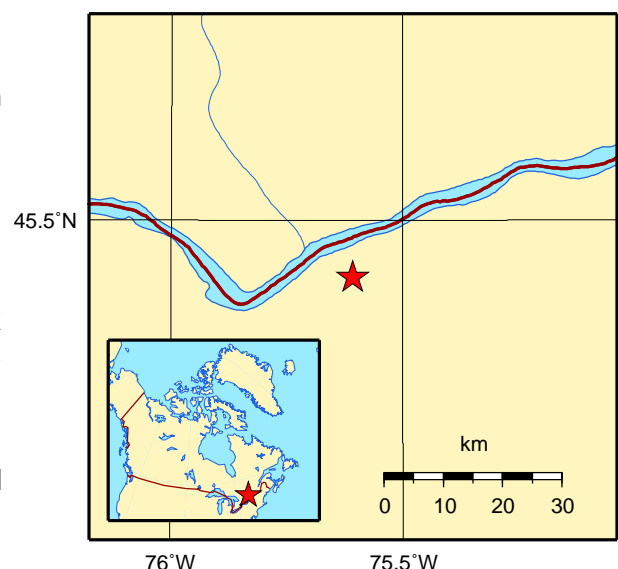
**Appendix C:** Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

**User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543** (in preparation)  
**Commentary J:** Design for Seismic Effects

**Geological Survey of Canada Open File xxxx**  
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites [www.EarthquakesCanada.ca](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information

Aussi disponible en français





**PRELIMINARY FOUNDATION DESIGN  
HIGHWAY 417 – 15 BRIDGES  
CITY OF OTTAWA, ONTARIO  
GWP No. 4074–11–00  
VOLUME 2 OF 2**

**GEOCRES No. 31G5-263**

**SUBMITTED TO  
McINTOSH PERRY CONSULTING ENGINEERS LTD.**

**August 21, 2015  
File: 19-3405-3**



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#### **APPENDIX 13**

SITE 3-301/2

#### **APPENDIX 14**

SITE 3-265/1

#### **APPENDIX 15**

SITE 3-265/2



## **APPENDIX 8**

SITE 3-310



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Kenton Power, P.Eng.  
(Reviewed by Fred Griffiths, P.Eng. and  
P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 S-EW OFF RAMP BRIDGE OVER GREEN CREEK (SITE 3-310) GWP 4074-11-00 GEOCRETS 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing bridge carrying westbound traffic from Highway 417 over Green Creek to Innes Road in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4010-11-01
- Site No. 3-310
- GEOCRETS No. 31G5-85
- Construction Contract 73-191
- Historic W.P. 13-68-11

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa, in the Township of Gloucester approximately 130 m south-east of the Highway 417 / Innes Road Interchange. The single span structure, carries the westbound traffic that is exiting Highway 417 using the S-EW ramp towards Innes Road. The bridge consist of one lane with paved shoulders at the south end and widens at the north end to accommodate two lanes with paved shoulders. Prior to construction of the bridge, Green Creek flowed from south to north at a location further to the west. The creek was diverted to an area of relatively flat ground with an approximate elevation between 61.6 m and 62.8 m.



Based on the historic General Plan Drawing (copy attached) the bridge is approximately 27 m long with a width varying from 11.6 m to 13.5 m, with a concrete pre-stressed girder structure. Both of the bridge abutments were to be supported by steel HP12x74 piles driven to bedrock.

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-85 dated August 1972. The original investigation was conducted in June 1972 for the initial location of the bridge and consisted of one sampled borehole designated Borehole 2 accompanied by a dynamic cone penetration test. Additional fieldwork was undertaken in April 1974, as a new bridge location had been selected. The additional field work consisted of two sampled boreholes designated 11 and 13 both accompanied by dynamic cone penetration tests. Two additional dynamic cone penetration tests designated 12 and 14 were also carried out opposite both Boreholes 11 and 13. An addendum to the original report was issued May 1974 to incorporate findings of the second investigation.

Drawing No. 72-11067A (copy attached) illustrates the locations of the bridge and the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by a silty sand with occasional clayey silt seams underlain by shale bedrock.

#### **3.1 Silty Sand with Gravel**

A silty sand stratum with varying amounts of gravel was encountered in all sampled boreholes. The stratum contained clayey silt seams though the thickness of the seams were not indicated. The surface of this deposit ranged from 61.2 m and 62.6 m in elevation, and the layer had a thickness of 3.7 m and 5.3 m. The standard penetration test 'N' values ranged from 7 to 69 blows per 0.3 m of penetration, indicating a loose to very dense condition but typically compact.

The results of a grain size analysis on a sample of this material indicated a gravel content of 27%, a sand content of 71%, and a fines content (combined silt and clay content) of 2%.

The moisture content of the sample tested was 9%.

#### **3.2 Bedrock**

A grey shale bedrock was encountered beneath the silty sand stratum in Boreholes 2, 11 and 13 as proven by BX size coring, and DCPT refusal on assumed bedrock in Boreholes 12 and 14. The bedrock surface ranged in elevation from 57.2 m to 57.5 m.

Bedrock core recovery ranged between 90% and 100%. The bedrock was described to be in sound condition. Geological mapping suggests that this site is near the boundary between the Billings and Carlsbad Formations.



### **3.3 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 57.5 m to 60.5 m. An artesian groundwater condition was encountered in Boreholes 2 and 13 once the borehole penetrated the lower granular stratum. The artesian head stabilized at an elevation of 62.8 m, approximately 0.8 m above the existing ground surface at the time of the investigation.

The water level of Green Creek at the time of the investigation is indicated on the General Plan Drawing as 60.2 m. The new creek bed was to be established at elevation 59.2 m.

## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in July 2012 for Bridge 3-310 with the report issued September 2013. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering Ltd. staff during the week of July 14, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

North and South Abutments:

- Gabions were installed for erosion control of the abutment slopes
- Vegetation growth over the gabions was noted
- Abutment slopes were measured at approximately 30°
- Abutment slopes were noted to be in generally good condition with no signs of erosion or settlement
- No obvious signs of scouring by creek flow or erosion of the toe of the abutment slopes were noted
- Vegetation on the embankment side slopes was noted with no obvious signs of settlement or erosion
- Rust staining and spalling of the concrete was noted on the abutment walls

Bridge and Road Surface:

- Concrete spalling of the barriers and sides of the bridge deck was noted
- Frequent longitudinal and transverse cracking of the asphalt surface was noted
- Some potholes, patching and crack repair of the asphalt surface were noted



## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

Where insufficient earth cover is provided, polystyrene insulation may be used to enhance frost protection.

#### **5.2 Seismic Considerations**

This site is best classified as a Soil Profile Type I in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC). The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the reported density of the silty sand and the measured groundwater at the site at the time of the investigation, this materials are classified as “not susceptible” to liquefaction during the design earthquake event

#### **5.3 Existing Foundations**

As per the Foundation Layout and Details Drawing the bridge abutments were designed to be supported on steel HP12x74 piles driven to bedrock. The available construction drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the piles is 95 tons / HP12x74 pile or approximately 845 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN / HP310x110 pile driven to bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

### **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.



## **6.1 Shallow Foundations**

Although spread footings within the overburden are possible, effective dewatering of the sand and gravel strata would be required. Given the anticipated relatively shallow depth to the shale bedrock, and the higher geotechnical resistance offered by the bedrock, spread footings within the overburden are not recommended.

The factored vertical geotechnical resistance of 1500 kPa at ULS may be used for the preliminary design of shallow foundations, founded on or in sound shale bedrock. The SLS condition will not govern for footings in or on the bedrock. The design of any new foundations on bedrock would need to consider interactions with the existing foundations and potential for undermining of the existing structural elements.

Resistance to lateral forces and sliding resistance between concrete and underlying materials should be evaluated using an unfactored coefficient of friction of 0.50 for cast-in-place concrete and 0.45 for pre-cast concrete on sound shale bedrock.

Based on current geological mapping, the shale present at this site may be from the Billings Formation which is susceptible to heaving if allowed to weather in the presence of oxygen. The general mechanism is that oxidation of pyrite within the shale produces sulfuric acid, which in turn reacts with calcite in the shale to form gypsum crystals, which occupy a larger volume than the original materials. A by-product of this chain of reactions also tends to increase sulphate levels which can attack buried concrete structures.

The potentially detrimental effects of shale heaving can be avoided by preventing exposure of the shale to oxygen both during construction and long term. This is typically achieved by limiting exposure of the shale to no more than one day prior to covering with a protective layer such as shotcrete or a concrete mud slab. Sulphate resistant cement should be used in such applications.

## **6.2 Deep Foundation – Piles**

Based on the review of existing subsurface stratigraphy at the site driven piles are considered to be suitable for the support of abutment foundations perched within the approach embankments. It should be noted that the bedrock surface elevation ranges from 57.2 m to 57.5 m.

### **6.2.1 Axial Resistance**

Steel piles (Grade 350 W steel) end-bearing on sound shale bedrock at this site may be designed on the basis of the following factored, vertical, geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.



### 6.2.2 Pile Tips

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### 6.2.3 Integral Abutment Considerations

As per the Foundation Layout and Details Drawing the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### 6.2.4 Pile Spacing

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### 6.2.5 Downdrag

In light of the cohesionless makeup of the existing overburden soils at this site, downdrag on existing and new piles is not considered a design issue.

### 6.2.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formula for cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where:

$k_h$  = coefficient of horizontal subgrade reaction

$n_h$  = coefficient related to soil density given in Table A

$z$  = depth of pile embedment (m)

$B$  = pile width perpendicular to load direction (m)

**Table A:**  $n_h$  values for cohesionless soils.

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 62	Embankment Fill	3,000
Below 62	Silty Sand	2,000

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.2.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)



**Table C:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 62.0	Embankment Fill	20	0.4
Below 62.0	Silty Sand	19	0.4

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D: Static Lateral Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.5
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33

#### 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$
<b>Yielding Wall</b>		
$K_{AE}$	0.33	0.40
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36
<b>Non-Yielding Wall</b>		
$K_{AE}$	0.55	0.66
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:



$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth, d (kPa)

d = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

H = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors provided in Table E are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## 6.5 Approach Embankments

The General Plan Drawing indicates that the maximum height of the approach fills was approximately 8.0 m from stream bed to approach fill with an embankment slope of 2H:1V (Horizontal:Vertical). Design drawings indicate that the abutment backfill is to consist of compacted, boulder free fill. The Foundation Design Report states that settlement of the approach fills will be elastic in nature and negligible after the completion of the fills. The embankment foundation is expected to be stable and no long term settlement problems are expected unless the fills are reconfigured.

## 6.6 Erosion Control

The gabions should be maintained to prevent erosion and scouring of the abutment slopes.

## 6.7 Excavations and Backfilling

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native overburden soils reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.



Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

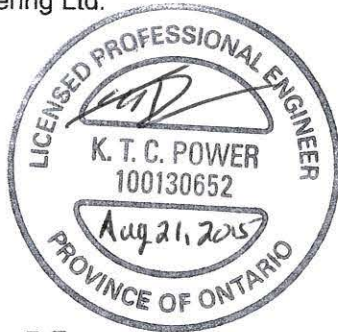
During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one.

Should the proposed works result in excavations to bedrock, samples of the bedrock should be acquired during the investigation and submitted for chemical testing to determine the pyritic heave potential.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.



Kenton C. Power, P.Eng.  
Geotechnical Engineer



Fred Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer

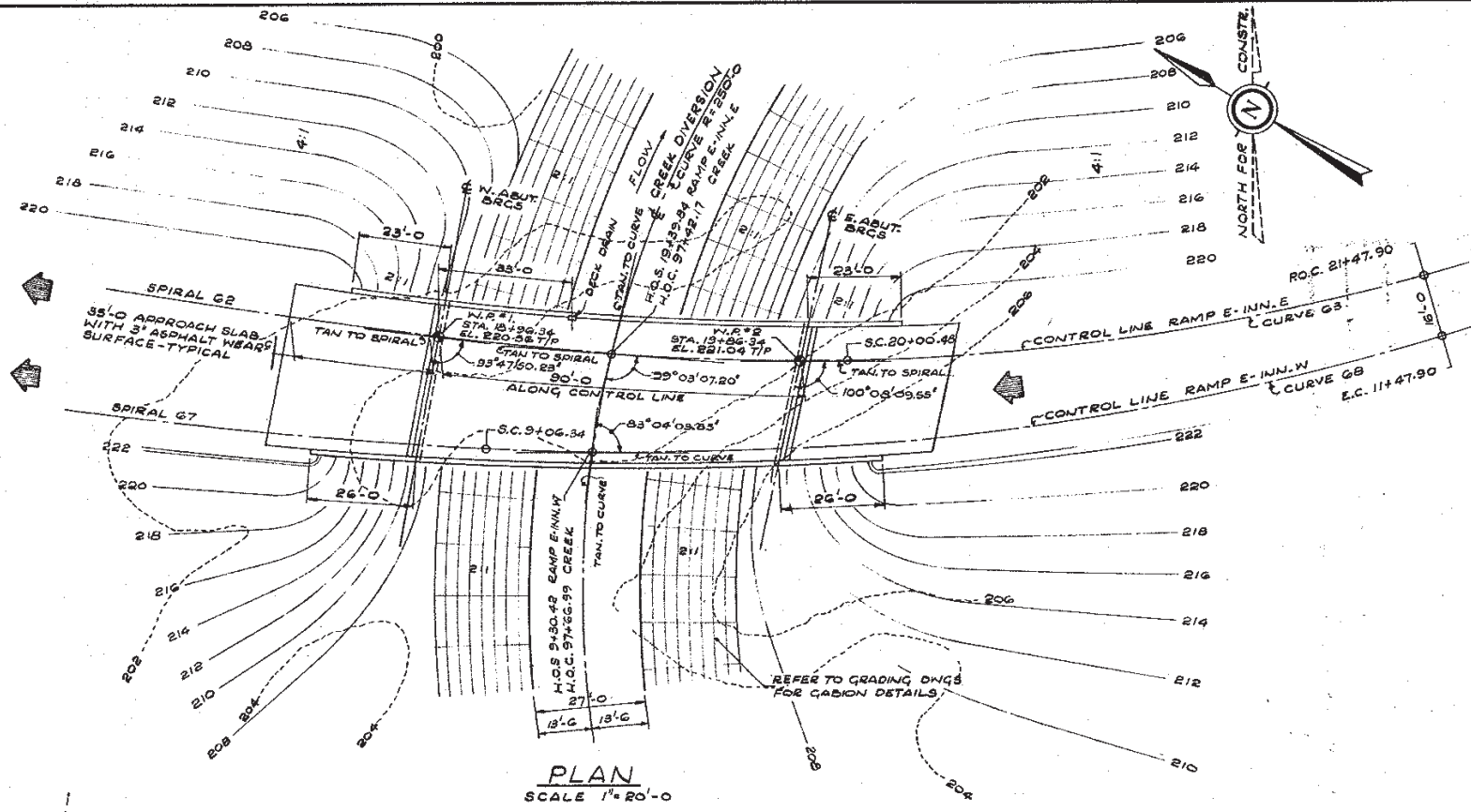


P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

## Attachments

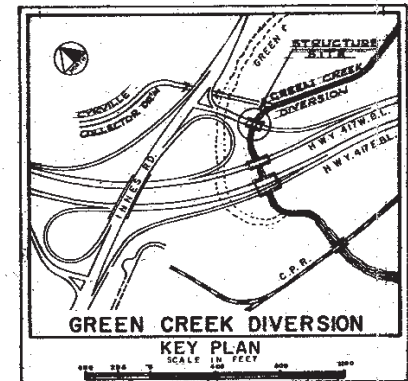
Client: McIntosh Perry  
File No.: 19-3405-3  
E file: 3-310 tel

Date: August 21, 2015  
Page 12



**REFERENCE BENCH MARK**  
BENCH MARK 218.75  
GEODEIC DATUM  
CUT CROSS ON N.E. CORNER OF R/WY BRIDGE  
540.0' RT. OF 202+15 E.B.L.

**CURVE DATA**  
**RAMP E-INN.E**  
CIRCULAR CURVE G3  
 $\Delta = 26^{\circ}00'00.00''$   
 $D = 10^{\circ}00'00.00''$   
 $R = 572.955'$   
 $T = 132.278'$   
 $L = 260.00'$   
 $E = 15.071'$   
**SPIRAL G2**  
 $\theta_s = 10^{\circ}00'00.00''$   
 $L_s = 200.00'$   
 $L_t = 133.547'$   
 $S_t = 66.861'$   
**RAMP E-INN.W**  
CIRCULAR CURVE G8  
 $\Delta = 18^{\circ}07'05.25''$   
 $D = 1^{\circ}30'00.00''$   
 $R = 768.944'$   
 $T = 121.800'$   
 $L = 241.568'$   
 $E = 9.649'$   
**SPIRAL G7**  
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 $L_s = 150.00'$   
 $L_t = 100.051'$   
 $S_t = 50.046'$



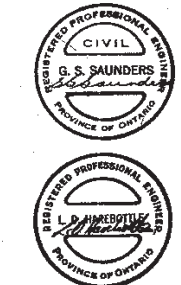
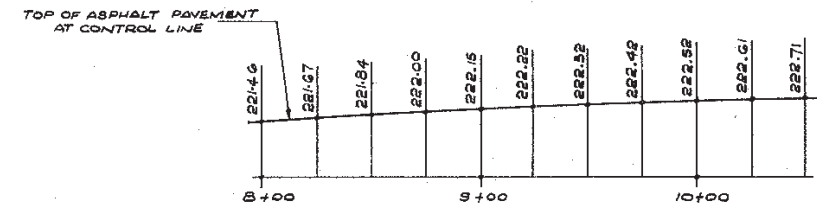
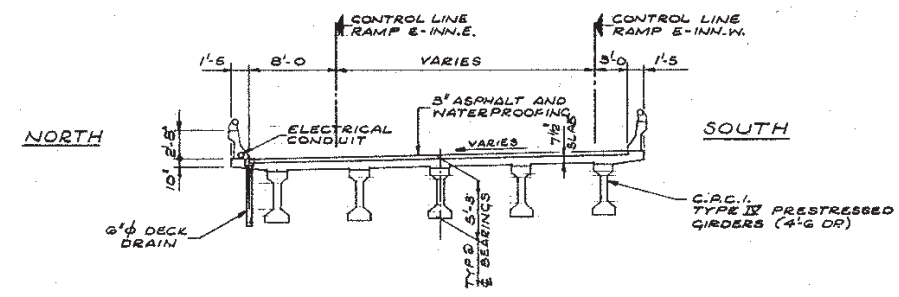
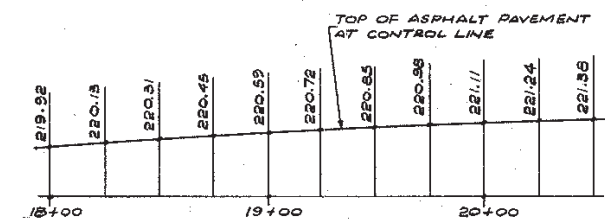
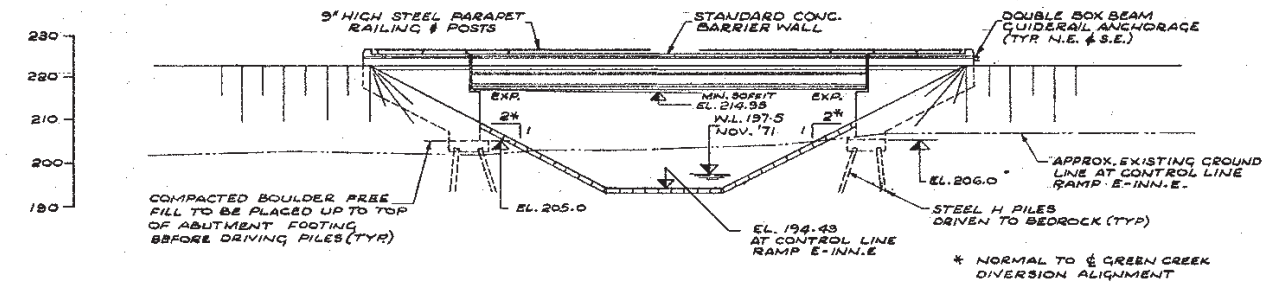
**NOTES:**  
• W.R. DENOTES WORKING POINT.  
• T/P DENOTES TOP OF ASPH. PAVT.

- LIST OF DRAWINGS**
1. GENERAL PLAN
  2. BOREHOLE LOCATIONS & SOIL STRATA
  3. FOUNDATION LAYOUT & DETAILS
  4. WEST ABUTMENT
  5. EAST ABUTMENT
  6. PRESTRESSED GIRDERS & BEARINGS
  7. DECK DETAILS
  8. CONCRETE BARRIER WALL (2'-8" HIGH)
  9. STEEL PARAPET RAILING (SINGLE TUBE)
  10. APPROACH SLABS
  11. STANDARD DETAILS I
  12. STANDARD DETAILS II
  13. PLAN - EMBEDDED DETAILS
  14. EMBEDDED DETAILS

**GENERAL NOTES**  
**CLASS OF CONCRETE**  
PRESTRESSED CONCRETE GIRDERS - 5000 P.S.I.  
APPROACH SLABS - 4000 P.S.I.  
DECK, DIAPHRAGMS, & BARRIER WALLS - 4000 P.S.I.  
REMAINDER - 3000 P.S.I.

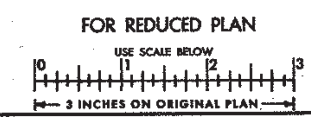
**CLEAR COVER ON REINFORCING STEEL**  
FOOTINGS & ABUTMENTS - 3"  
DECK: TOP - 1 1/2", BOTTOM - 1", DIAPHRAGMS - 1 1/2"  
BARRIER WALLS - 1 1/2"  
APPROACH SLABS - 2"

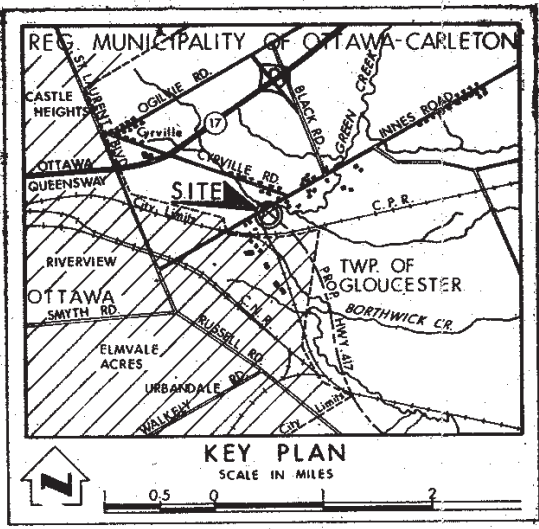
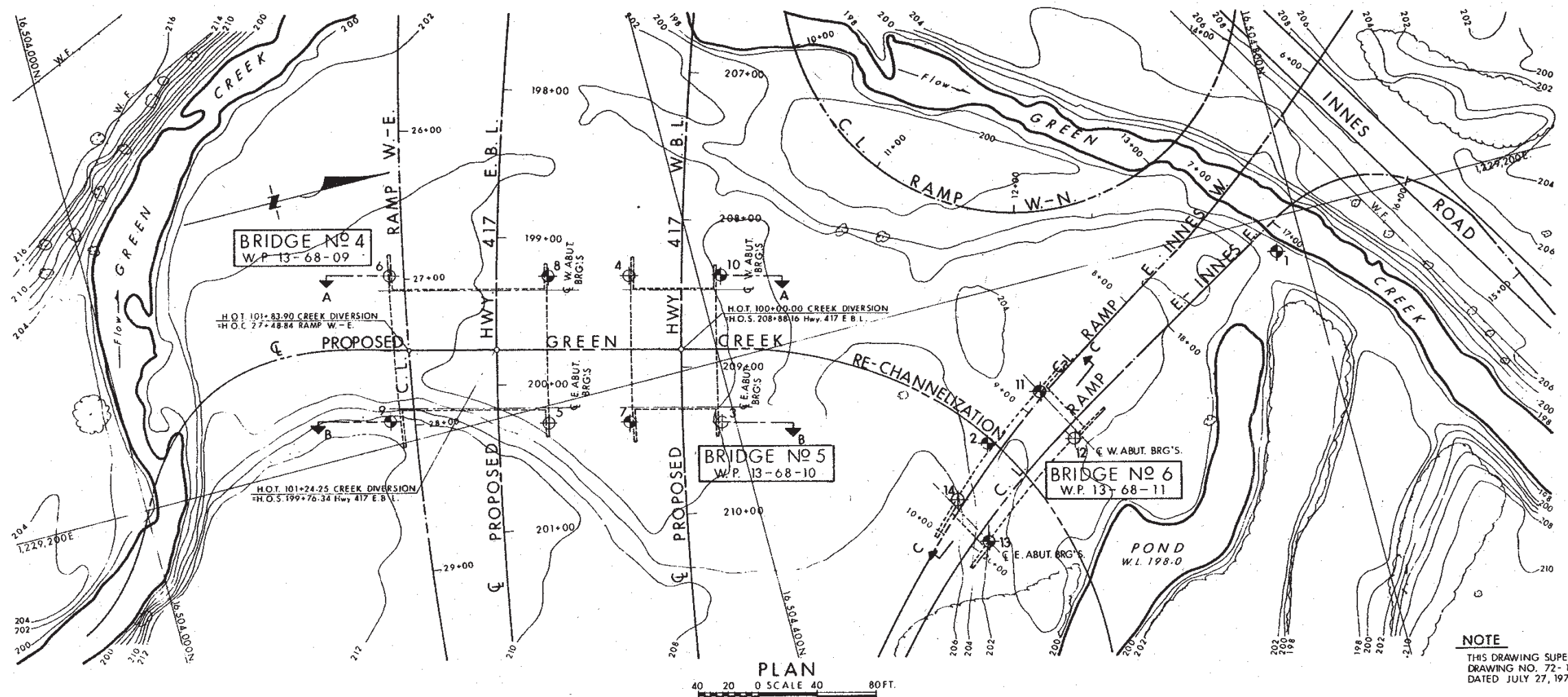
**CONSTRUCTION NOTES**  
THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF  $\pm 1/8"$ . NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.










REVISION	DATE	BY	DESCRIPTION
MAY 74	A.B.		THIS PLAN SUPERCEDES PREVIOUS GENERAL PLAN 3-310-1 ISSUED APRIL 73 DUE TO CHANGE IN GREEN CREEK DIVERSION ALIGNMENT.

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
DeLeuw, Cather ENGINEERS & PLANNERS - OTTAWA	
GREEN CREEK UNDER N.B. RAMP TO INNES ROAD BRIDGE N°6 RELOCATION KING'S HIGHWAY No. 417 CO. REG. MUNICIPALITY OF OTTAWA-CARLETON TWP. CLOUCESTER LOT 23 CON. III	
GENERAL PLAN	
APPROVED R.O. STRUCTURAL ENGINEER	CONTRACT No. 73-191
DESIGN G.S.S. CHECK L.D.H. DRAWING A.B. CHECK R.A.P. DATE MAY 74	W.P. No. 13-68-11 SITE No. 3-310 SHEET 1.





LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation, June 1972 & Apr. 1974		
	Head		
	Artesian Water Levels		
	Encountered		
CO - ORDINATES			
NO.	ELEVATION	CO - ORDINATES	
		NORTH	EAST
1	201.0	16, 504, 781	1,229, 223
2	202.7	16, 504, 561	1,229, 300
3	203.7	16, 504, 391	1,229, 240
4	203.0	16, 504, 355	1,229, 128
5	202.1	16, 504, 277	1,229, 212
6	204.5	16, 504, 280	1,229, 087
7	202.4	16, 504, 331	1,229, 225
8	202.5	16, 504, 302	1,229, 114
9	210.3	16, 504, 175	1,229, 183
10	203.4	16, 504, 415	1,229, 143
11	201.9	16, 504, 604	1,229, 274
12	202.4	16, 504, 619	1,229, 311
13	200.9	16, 504, 545	1,229, 364
14	205.4	16, 504, 531	1,229, 332

NOTE  
THIS DRAWING SUPERCEDES  
DRAWING NO. 72-11067A  
DATED JULY 27, 1972

NOTE  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION
APR 78	S.O.	REVISED	BRIDGE NO. 6 & ADDED BORE HOLES 11, 12, 13 & 14

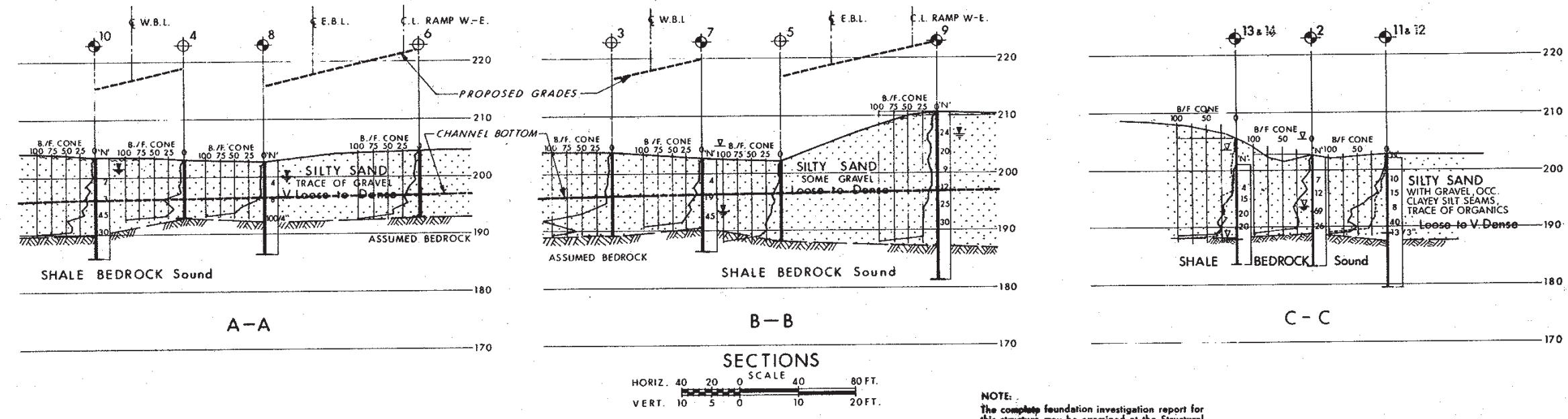
MINISTRY OF TRANSPORTATION & COMMUNICATIONS  
ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE

**GREEN CREEK**

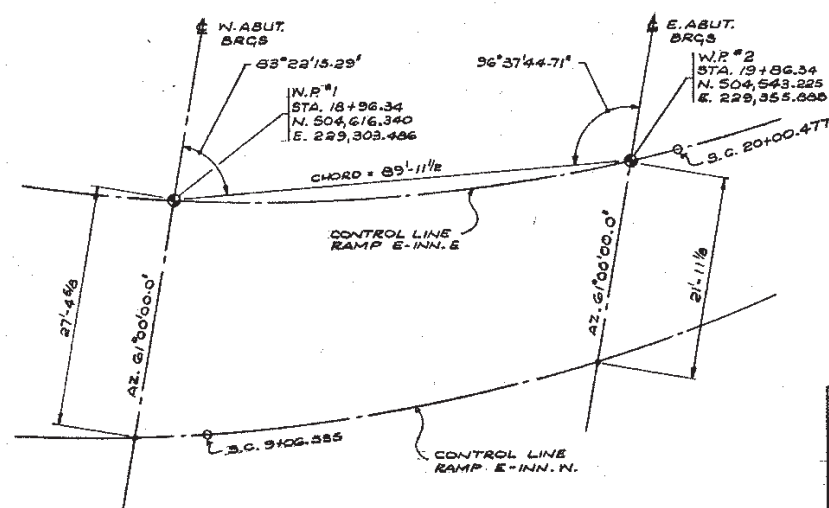
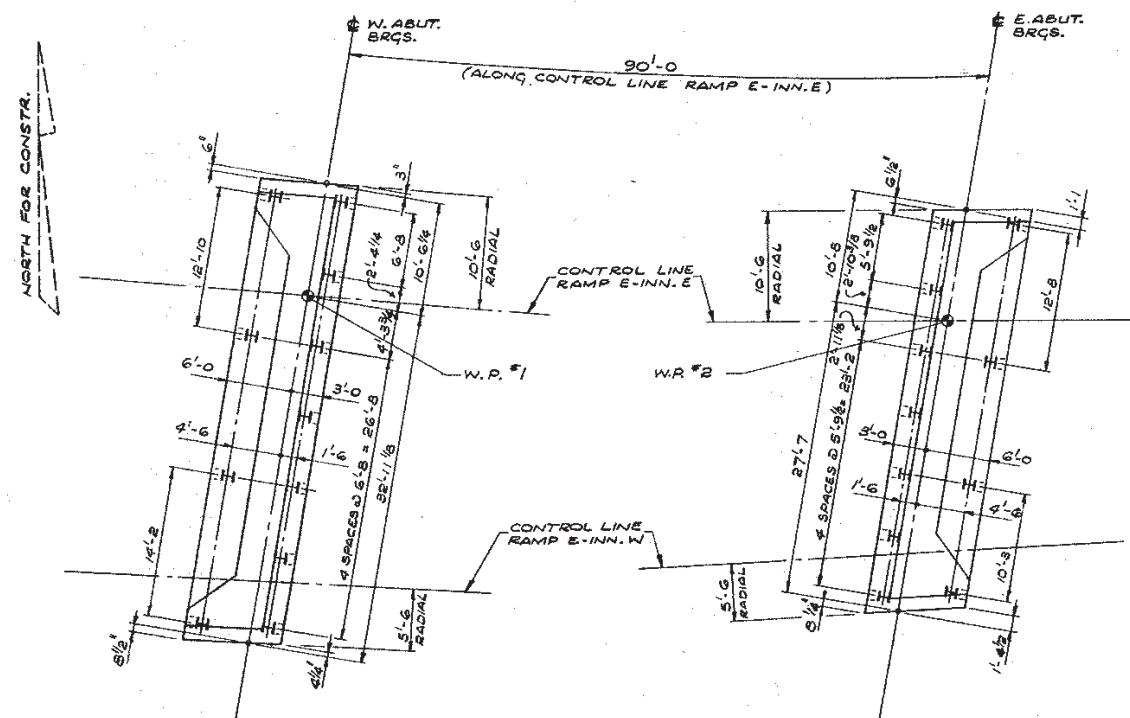
HIGHWAY NO. Prop. 417 DIST. NO. 9  
REG. MUNICIPALITY OF OTTAWA-CARLETON  
TWP. GLOUCESTER LOT CON.

**BORE HOLE LOCATIONS & SOIL STRATA**

SUBMD. S. A. CHECKED	W.P. NO. 13-68-01	DRAWING NO.
DRAWN	JOB NO. 72-11067	<b>72-11067A</b>
DATE APR. 30, 1974	SITE NO. 3-310	BRIDGE DRAWING NO.
APPROVED	CONT. NO. 73-191	<b>3-310-2</b>

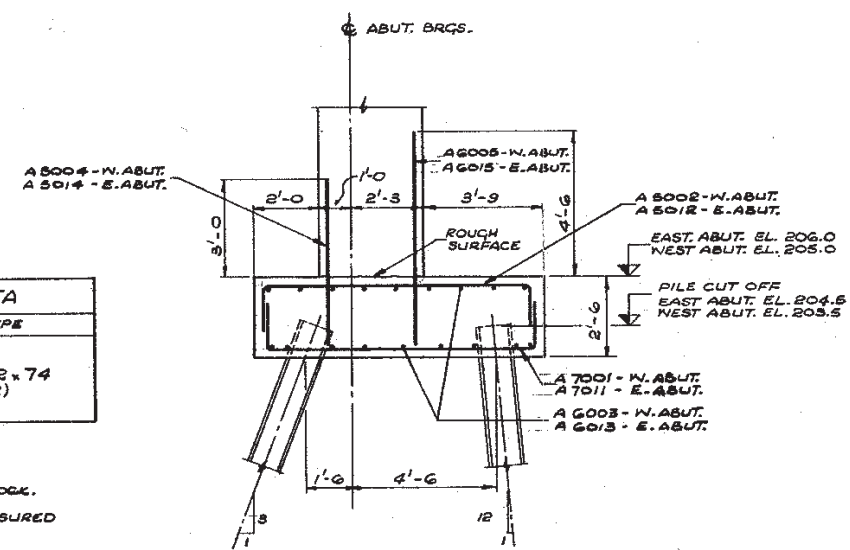
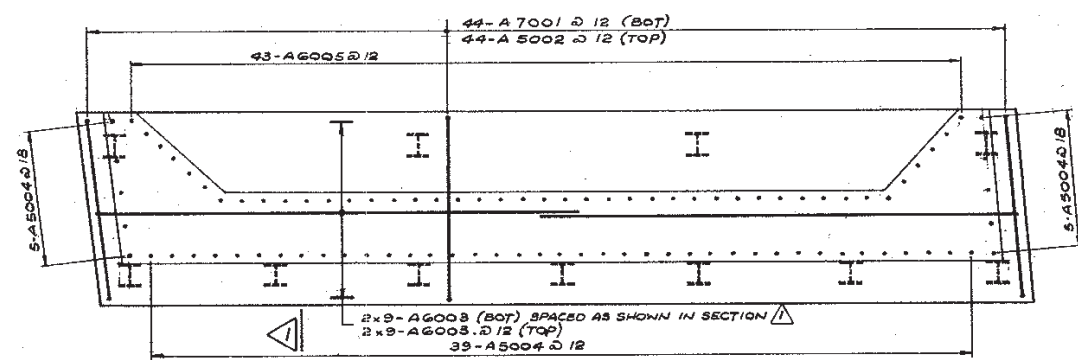
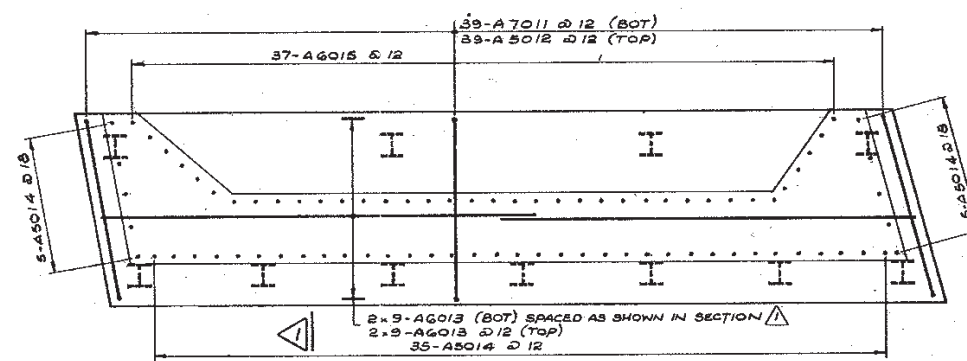


NOTE:  
The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the OTTAWA District Office.




STEEL 'H' PILE DATA			
LOCATION	NO.	LENGTH	TYPE
WEST ABUT	11	18'-0	HP 12 x 74 (TYR)
EAST ABUT	11	20'-0	

- NOTES:
1. PILES TO BE DRIVEN TO BEDROCK.
  2. SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTING.



REVISIONS	MAY '78	A.B.	THIS DNG. SUPERCEDES DNG 3-310-3 ISSUED APRIL '73, DUE TO CHANGE IN GREEN CREEK DIVERSION ALIGNMENT.
	DATE	BY	DESCRIPTION

<b>MINISTRY OF TRANSPORT AND COMMUNICATIONS</b> <b>ONTARIO</b>													
<b>DeLeuw, Cather</b> <b>ENGINEERS &amp; PLANNERS - OTTAWA</b>													
<u><b>GREEN CREEK UNDER N.B. RAMP</b></u> <u><b>TO INNES ROAD</b></u> <u><b>BRIDGE N<sup>o</sup> 6 RELOCATION</b></u>													
<b>KING'S HIGHWAY No. 417</b>	<b>DIST. No. 2</b>												
<b>CO. REG. MUNICIPALITY OF OTTAWA - CARLETON</b>													
<b>TWP. GLOUCESTER</b>	<b>LOT 23</b>												
<b>CON. III</b>													
<u><b>FOUNDATION LAYOUT AND DETAILS</b></u>													
<b>APPROVED</b> 	<b>CONTRACT No. 73-191</b>												
<small>STRUCTURAL ENGINEER</small>													
<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 25%;"><b>DESIGN</b></td> <td style="width: 25%;"><b>L.D.H.</b></td> <td style="width: 25%;"><b>CHECK</b></td> <td style="width: 25%;"><b>C.S.S.</b></td> </tr> <tr> <td><b>DRAWING</b></td> <td><b>A.S.</b></td> <td><b>CHECK</b></td> <td><b>R.A.P.</b></td> </tr> <tr> <td><b>DATE</b></td> <td><b>May 176</b></td> <td><b>LOADING</b></td> <td><b>NO 20-66</b></td> </tr> </table>	<b>DESIGN</b>	<b>L.D.H.</b>	<b>CHECK</b>	<b>C.S.S.</b>	<b>DRAWING</b>	<b>A.S.</b>	<b>CHECK</b>	<b>R.A.P.</b>	<b>DATE</b>	<b>May 176</b>	<b>LOADING</b>	<b>NO 20-66</b>	<b>W.P. No. 13-68-11</b>  <b>SITE No. 3-310 SHEET 3</b>
<b>DESIGN</b>	<b>L.D.H.</b>	<b>CHECK</b>	<b>C.S.S.</b>										
<b>DRAWING</b>	<b>A.S.</b>	<b>CHECK</b>	<b>R.A.P.</b>										
<b>DATE</b>	<b>May 176</b>	<b>LOADING</b>	<b>NO 20-66</b>										



**APPENDIX 9**  
**SITE 3-302/1**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Justin Gray  
(Reviewed by Paul Carnaffan, P.Eng. and  
P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 EASTBOUND CANADIAN PACIFIC RAILWAY OVERPASS (SITE 3-302/1) GWP 4074-11-00 GEOCRES 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing Canadian Pacific Railway (CP) overpass of the eastbound lanes of Highway 417 in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 267-00-01
- Site No. 3-302/1
- GEOCRES No. 31G5-80
- Construction Contract 73-191
- Historic W.P. 13-68-06

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa, in the Township of Gloucester where the eastbound lanes of Highway 417 crosses over the CP railway tracks, approximately 800 m south of the Highway 417 / Innes Road Interchange. Based on the historic General Plan Drawing (copy attached), the bridge is a 20.5 m wide, and 79.1 m long, four-span prestressed, precast concrete girder structure. The main track, an abandoned siding and an allowance for a future track are located between Piers 1 and 2. Another track identified as the east leg of the WYE track is located between Piers 2 and 3. A National Capital Commission access road is located between Pier 3 and the east abutment.

The bridge abutments and wingwalls are supported by steel HP12x74 piles driven to bedrock. The piers are supported by 42 inch diameter steel lined concrete caissons with 36 inch diameter



socket keyed 0.3 m into sound bedrock. The caissons supporting the piers do not have a pile cap as they extend up above the ground surface as columns.

The natural terrain in the vicinity of the bridge is generally flat with elevations ranging from 64 to 65.5 m. The design drawings show that the approach fills were to be constructed by placing as much as 11 m of boulder-free earth fill at a slope of 2H:1V (Horizontal:Vertical).

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRETS Report No. 31G5-80 dated February 1972. The investigation consisted of seven sampled boreholes, six of which were accompanied by dynamic cone penetration tests. Drawing No. 71-11126A (copy attached) illustrates the locations of the bridge, the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by silty sand with some gravel, overlaying shale bedrock.

#### **3.1 Silty Sand**

A silty sand with some gravel deposit was encountered in all boreholes advanced at the site. The surface of this deposit ranged from 64.0 to 64.4 m in elevation, and the layer had a thickness of 10.1 to 11.9 m. The standard penetration test 'N' values varied greatly for this deposit ranging from 7 to greater than 100 blows per 0.3 m of penetration, indicating a loose to very dense condition. Gradation test results on samples of this material indicate a gravel content between 0% and 45%, sand content between 43% and 96% and a fines content (combined silt and clay content) ranging from 2% to 30%. Boulders were noted within this deposit in some of the boreholes.

#### **3.2 Bedrock**

Beneath the silty sand layer, a grey shale bedrock was encountered with surface elevations ranging from 52.5 to 57.3 m. The upper 1.0 to 2.4 m of the shale was generally fractured and weathered. At greater depths the shale was described as being in a sound condition. Geological mapping suggests that this site is near the boundary between the Carlsbad and Billings bedrock formations.

#### **3.3 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 63.1 and 63.5 m.

### **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in March 2012 for Bridge 3-302/1 with the report issued September 2013. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in fair to good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.



The site was inspected by Thurber Engineering Ltd. staff during the week of July 14, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

- No evidence of slope stability issues were noted; the slopes of the approach embankments were well vegetated
- Evidence of erosion was noted beneath both abutments; no erosion protection systems were present beneath either abutment
- A crack in the pavement surface at the end of the approach slab (west side) was observed
- A slight dip in the embankment before the bridge near the east abutment was noted
- No signs of settlement of the piers were observed
- Some ponded water was noted around the middle pier

## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

It is noted that the Foundation Design Report for this structure recommended a minimum of 4 feet (1.2 m) of soil cover for frost protection. The soil cover for the abutments should be reviewed and where insufficient earth cover is provided, polystyrene insulation may be used to enhance existing frost protection measures.

#### **5.2 Seismic Considerations**

This site is classified as a Soil Profile Type I in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC). The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the reported density of the silty sand and the measured groundwater at the site at the time of the investigation, this material is classified as “not susceptible” to liquefaction during the design earthquake event.

#### **5.3 Existing Foundations**

The Foundation Design Report concluded that the loose to compact granular soil conditions present at shallow depth at the site would not provide for an economical spread footing type foundation. In addition, concerns were raised regarding dewatering and basal stability for excavations for footings or pile caps at the pier locations.

As per Foundation Plan Drawing (copy attached) the bridge abutments and retaining walls were designed to be supported on steel HP12x74 piles driven to bedrock. The available construction



drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the piles is 95 tons / HP12x74 pile or approximately 845 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN / HP310x110 pile driven to bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

The piers are supported by 42 inch diameter steel lined concrete caissons with a 36 inch diameter socket extending 1 ft. (0.3 m) into sound bedrock (the total length of the sockets range from 4 to 7 ft. (1.2 to 2.1 m)). The available contract drawings do not indicate the design loads for the caissons, however, the Foundation Design Report indicates that for the caissons socketed at least 1 ft. into sound bedrock, a 30-inch diameter caisson may be designed with an allowable load of 250 tons / caisson or approximately 2200 kN/caisson. For a 36-inch diameter caisson the recommended allowable load increases to 3150 kN/caisson.

## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Deep Foundations**

The design for the replacement or widening of the existing structure will need to consider the potential interaction with the existing structure and compatibility with the existing foundation systems.

#### **6.1.1 Integral Abutment Considerations**

As per the Foundation Plan Drawing the existing abutments and retaining walls are supported by battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

#### **6.1.2 Deep Foundations - Caissons**

For preliminary design purposes, the factored vertical geotechnical resistance at ULS for 36 inch (914 mm) diameter caissons keyed 300 mm into sound shale is 3150 kN/caisson. The SLS condition will not govern for caissons socketed into sound bedrock. The geotechnical resistance could be increased by increasing the socket length within the sound bedrock.



### 6.1.3 Deep Foundations - Driven Steel Piles

Steel piles (Grade 350 W steel) end-bearing on sound shale bedrock at this site may be designed on the basis of the following factored vertical geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

### 6.1.4 Pile Tips

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### 6.1.5 Pile Spacing

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### 6.1.6 Downdrag

In light of the cohesionless makeup of the existing overburden soils at this site, downdrag on existing and new piles is not considered a design issue.

### 6.1.7 Lateral Resistance of Deep Foundations

The lateral resistance of both existing and new driven piles/bored caissons can be assessed based on the method outlined in the CHBDC. For a driven H-pile or bored caisson, the lateral soil resistance may be calculated using the following formula.

$$k_h = \frac{n_h z}{B}$$

where:

$k_h$  = coefficient of horizontal subgrade reaction

$n_h$  = coefficient related to soil density given in Table A

$z$  = depth of pile/caisson embedment (m)

$B$  = pile/caisson width perpendicular to load direction (m)

**Table A:**  $n_h$  values for cohesionless soils.

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 63.5	Embankment Fill	3,000
63.5 to 53.3	Silty Sand with some Gravel	2,000
Below 53.3	Weathered Shale Bedrock	6,000

Note that the weathered shale has been treated as a cohesionless soil for a conservative assessment of the lateral resistance.

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.1.8 Uplift Resistance of Deep Foundations

The unfactored uplift resistance of new or existing piles/caissons can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)



The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)

**Table C:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 63.5	Embankment Fill	20	0.4
63.5 to 53.3	Silty Sand with some Gravel	19	0.4

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

## 6.2 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D:** Static Lateral Earth Pressure Coefficients

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.5
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33

### 6.3 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E:** Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$
<b>Yielding Wall</b>		
$K_{AE}$	0.33	0.40
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36
<b>Non-Yielding Wall</b>		
$K_{AE}$	0.55	0.66
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:



$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth, d (kPa)

d = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

H = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors provided in Table E are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## 6.4 Approach Embankments

Based on General Plan Drawing, the approach embankments should consist compacted, boulder free fill to a maximum height of 11.3 m above the original ground surface constructed with 2H:1V slopes. The Foundation Design Report states that the settlement would be elastic in nature with a predicted total settlement of up to 2 inches, with the majority of the settlement taking place during or immediately after the construction of the embankments. The embankment foundation is expected to be stable and no long term settlement problems are expected unless the fills are reconfigured.

## 6.5 Erosion Control

Active erosion beneath both abutments was noted as no erosion protection systems were present at the site.

The eroded slopes in front of the abutments should be reinstated, erosion protection measures incorporated and the drainage measures enhanced beneath the abutments to prevent further erosion of the embankment material.

## 6.6 Excavations and Backfilling

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native overburden soils reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA.



Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one. In addition, coordination with the railway, railway design code requirements and track protection will need to be considered.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.



Justin Gray, B.Eng., E.I.T.

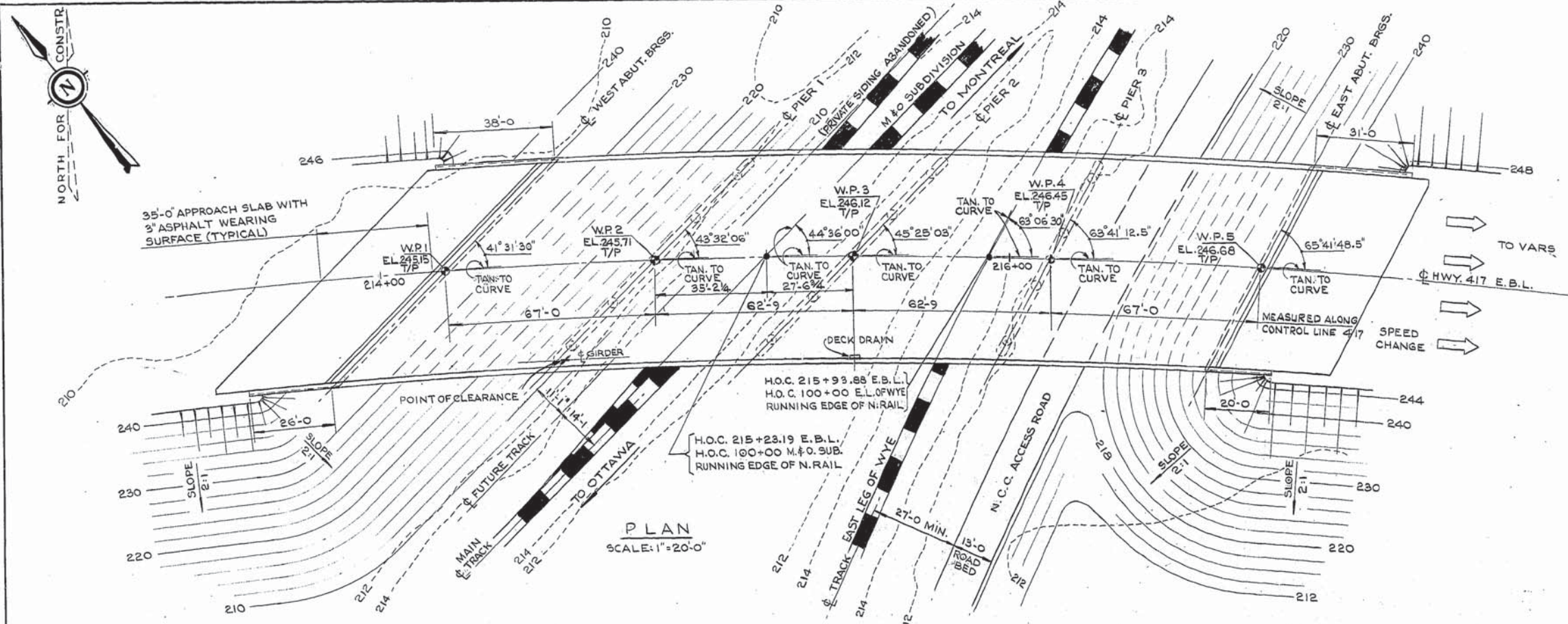
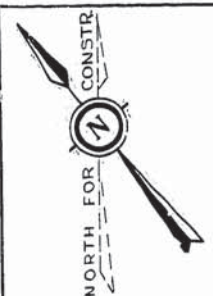


Paul Carnaffan, P.Eng.  
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

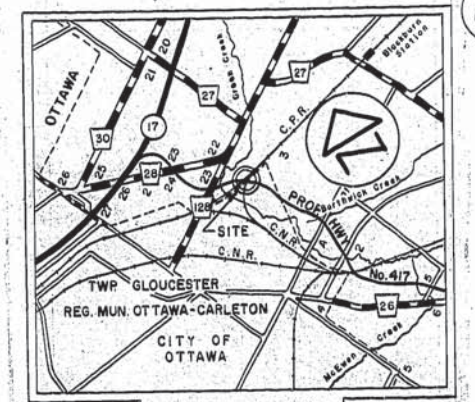
Attachments



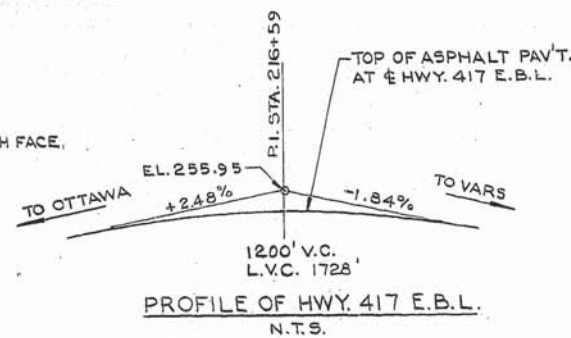
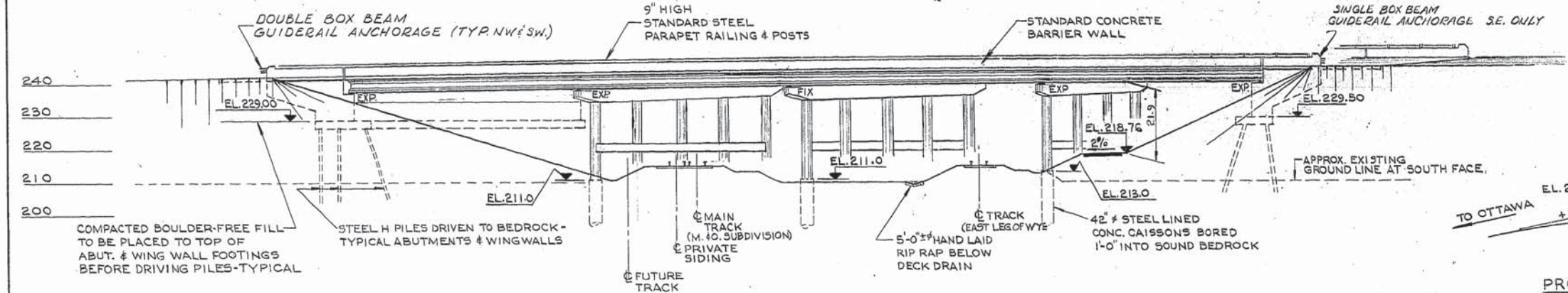
**REFERENCE BENCH MARK**  
B.M. 220.52  
GEODETIC DATUM  
TOP NUT ON THE N.E. CORNER  
OF HYDRO TOWER 350 RIGHT OF  
STA. 212+00 HWY. 417 E.B.L.

**CURVE DATA:**  
HWY. 417 E.B.L.  
 $\Delta = 67^\circ 44' 40''$   
 $\Delta c = 54^\circ 14' 40''$   
 $D = 3^\circ 00' 00''$   
 $R = 1909.86$   
 $Lc = 1808.15$   
 $Ec = 235.96$   
 $Ta = 978.26$   
 $Ls = 450.00$   
 $os = 6^\circ 45' 00''$

**NOTES:**  
W.P. DENOTES WORKING POINT  
T/P DENOTES TOP OF ASPHALT  
PAVEMENT.

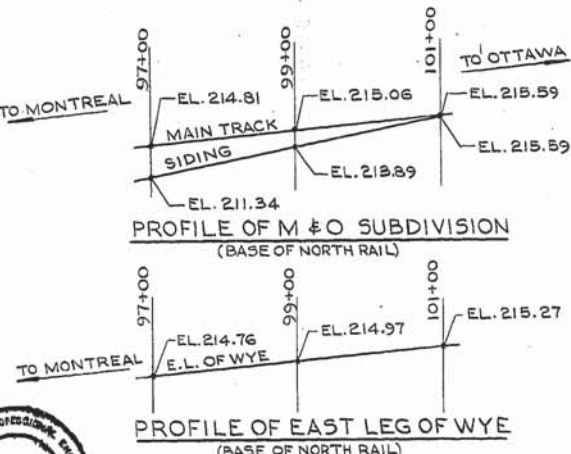
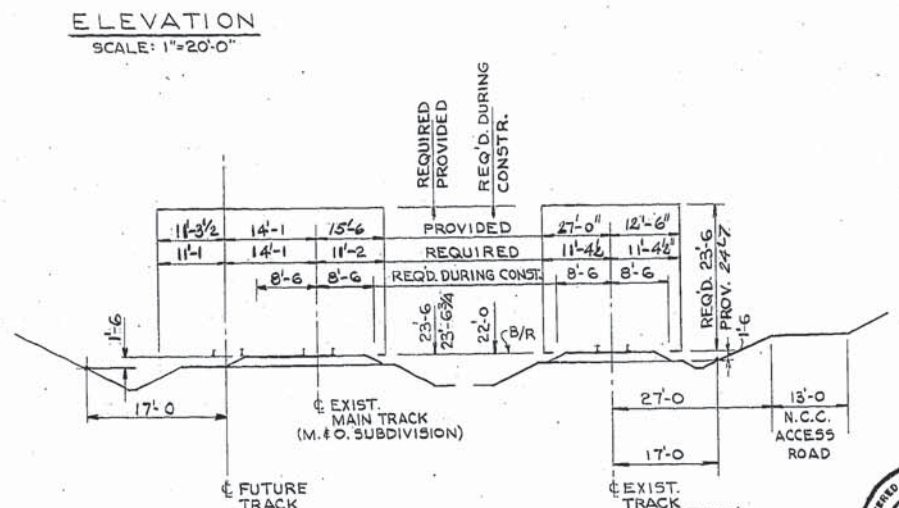
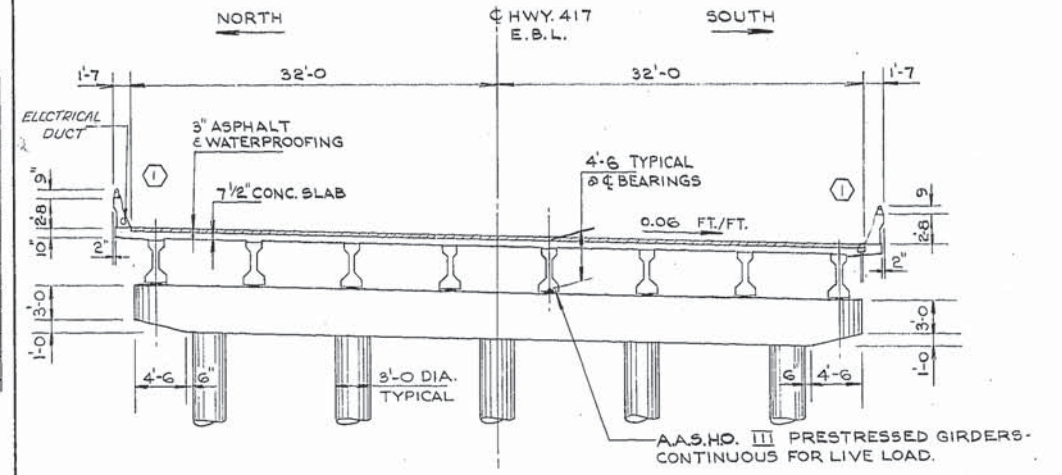


- LIST OF DRAWINGS**
1. GENERAL PLAN.
  2. BORE HOLE LOCATIONS & SOIL STRATA.
  3. FOUNDATION PLAN.
  4. WEST ABUTMENT-DETAILS.
  5. EAST ABUTMENT-DETAILS.
  6. WEST ABUTMENT-REINFORCEMENT.
  7. EAST ABUTMENT-REINFORCEMENT.
  8. PIER DETAILS & REINFORCEMENT I.
  9. PIER DETAILS & REINFORCEMENT II.
  10. PRESTRESSED GIRDERS & BEARINGS.
  11. DECK DETAILS & REINFORCEMENT.
  12. APPROACH SLABS.
  13. CONCRETE BARRIER WALL (2'-8" HIGH).
  14. DETAILS OF 9" HIGH STEEL PARAPET RAILING.
  15. STANDARD DETAILS I.
  16. STANDARD DETAILS II.
  17. PLAN - EMBEDDED DETAILS
  18. EMBEDDED DETAILS



**NOTES:**

1. CLASS OF CONCRETE.  
PRESTRESSED CONCRETE GIRDERS 5,000 P.S.I.  
APPROACH SLABS - 4,000 P.S.I.  
DECK & BARRIER WALLS - 4,000 P.S.I.  
CAISSONS, COLUMNS AND REMAINDER 3,000 P.S.I.
2. CLEAR COVER ON REINFORCING STEEL.  
FOOTINGS 3", ABUTMENTS & PIERS 3",  
DECK: TOP 1 1/2", BOT. 1", DIAPHRAGMS 1 1/2",  
EXCEPT AS NOTED.
3. CONSTRUCTION NOTES.  
THE CONTRACTOR IS RESPONSIBLE FOR FINISHING  
THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED  
ELEVATIONS WITH A TOLERANCE OF  $\pm 1/8"$ .  
NO CONCRETE SHALL BE PLACED ABOVE THE  
ABUTMENT BEARING SEATS, UNTIL THE CONCRETE  
IN THE DECK HAS BEEN PLACED.



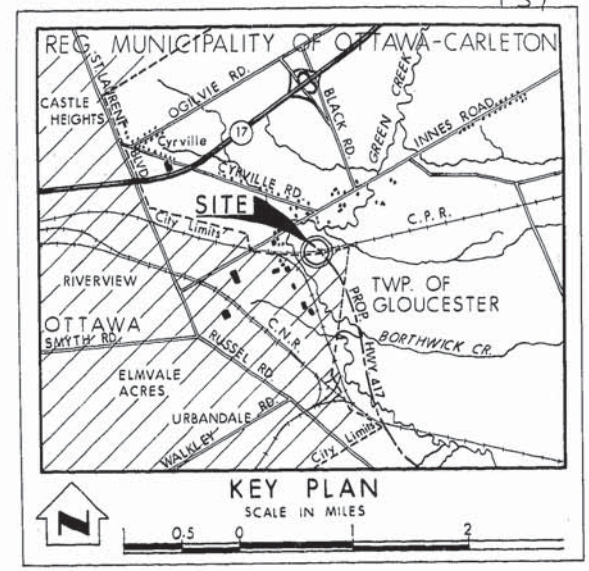
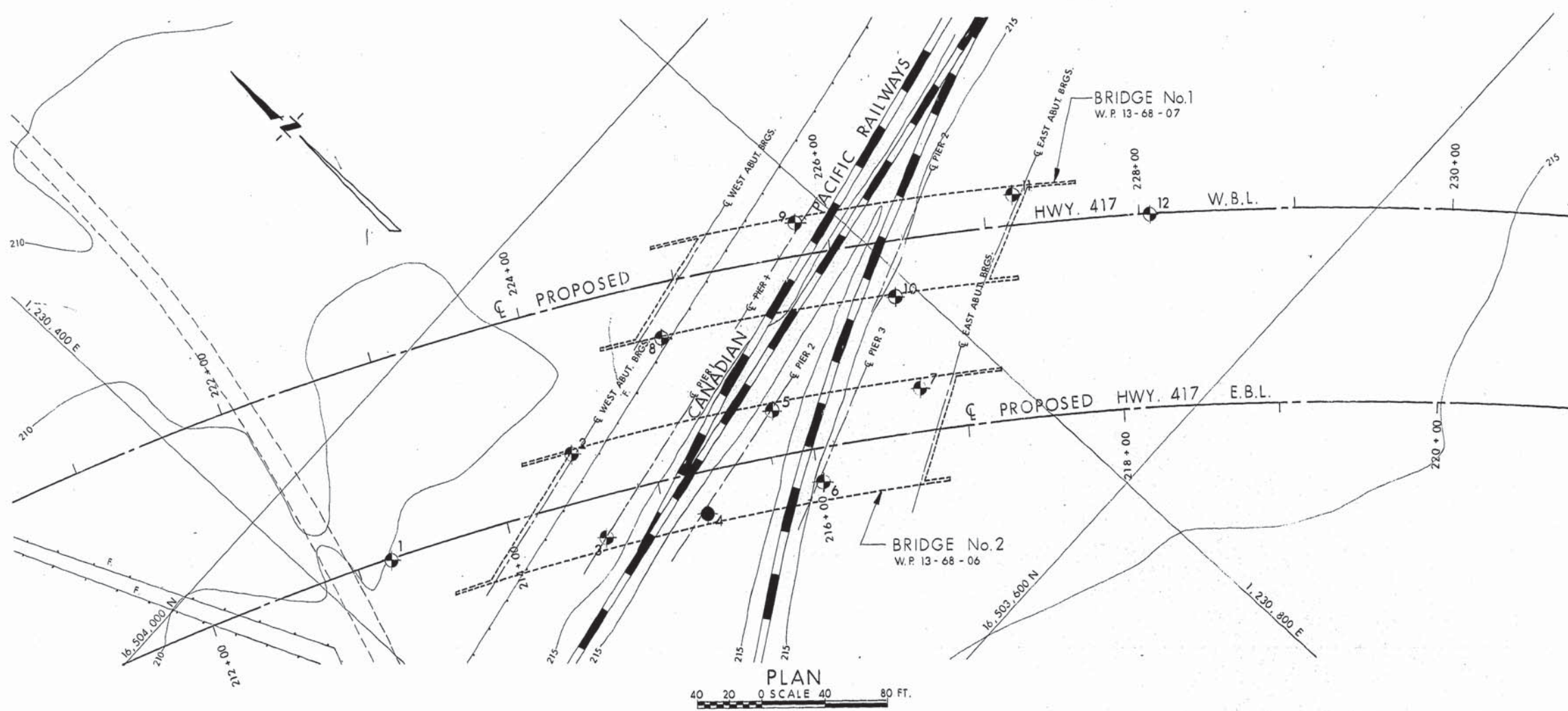
REVISIONS	DATE	BY	DESCRIPTION
1	11/27/74	B.S.	(1) REMOVED DIMENSIONS






DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO  
**DeLeuw, Cather**  
ENGINEERS & PLANNERS - OTTAWA  
C.P.R./C.N.R. OVERHEAD - E.B.L.  
(1.5 MILES SOUTH OF O.Q.W.)  
BRIDGE No. 2  
KING'S HIGHWAY No. 417 DIST. No. 9  
CO. REG. MUN. OF OTTAWA-CARLETON  
TWP. GLOUCESTER LOT 22 CON. 3

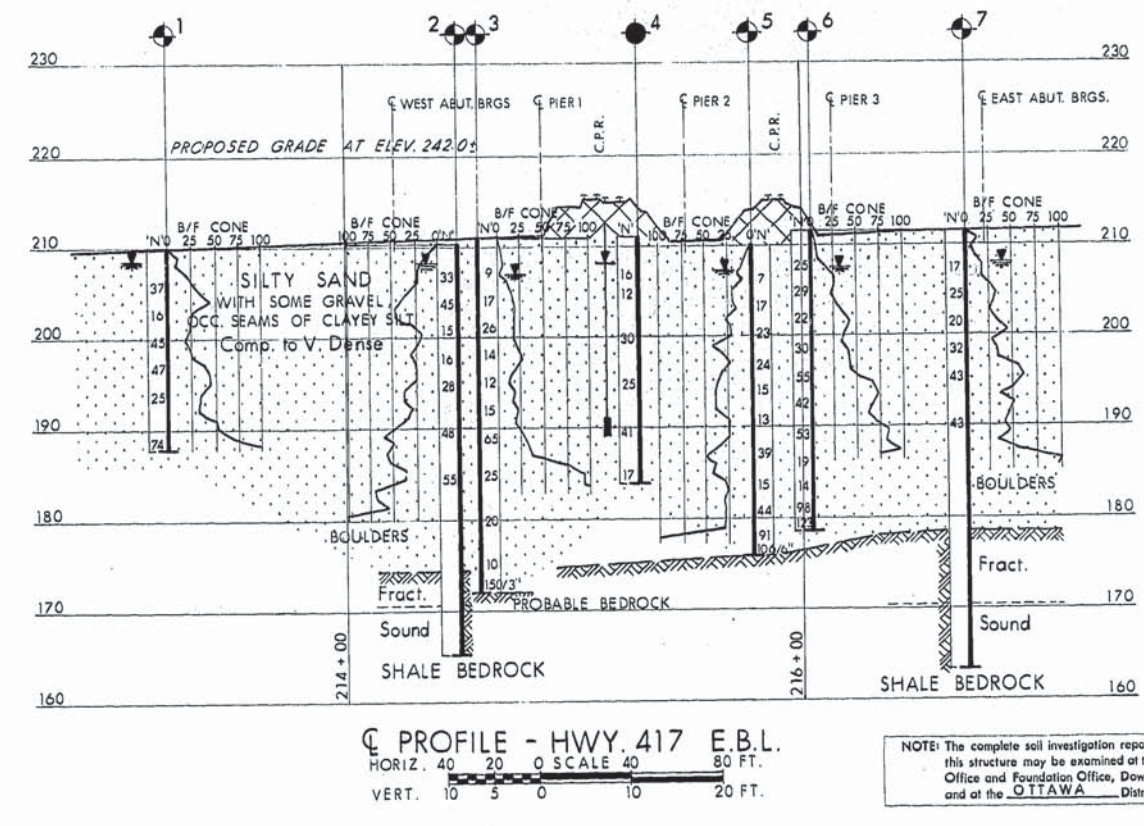
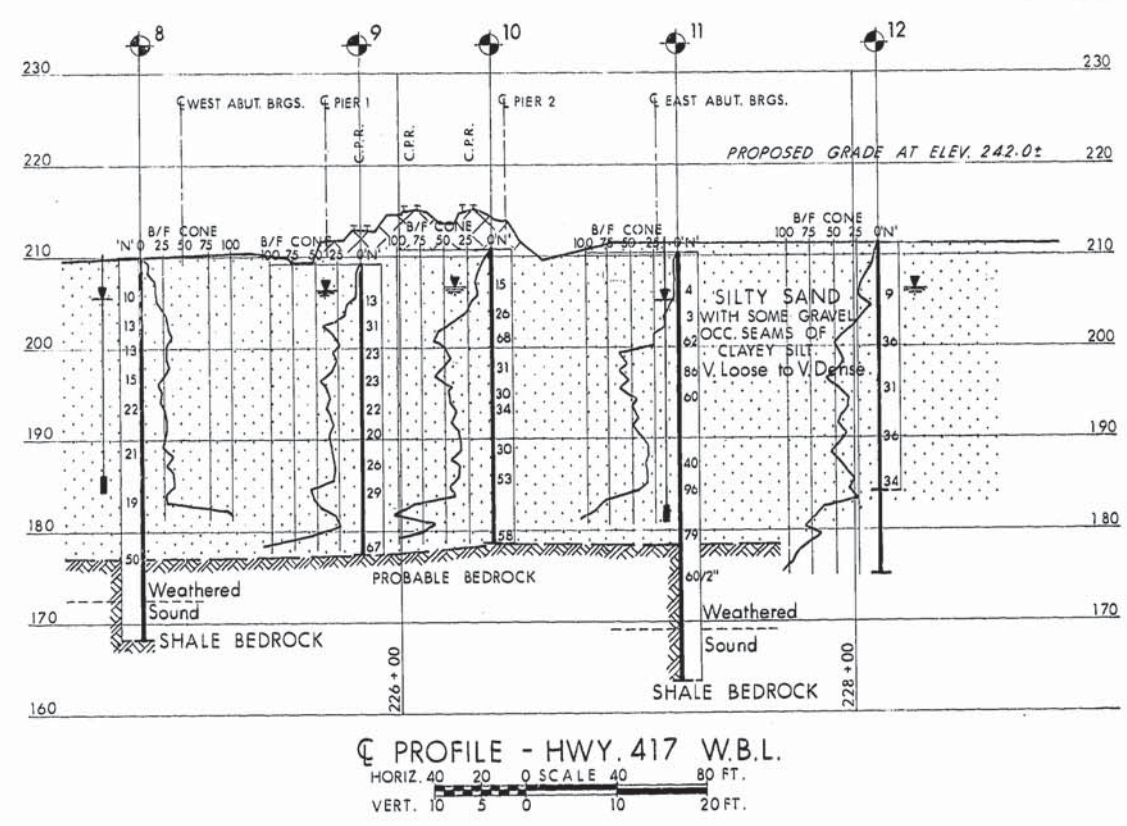
**GENERAL PLAN**

APPROVED	30	SITE No. 3-302A	W.P. No. 13-68-06
DESIGN	S.F.	CHECK	G.S.S.
DRAWING	F.R.G.	CHECK	SF
DATE	FEB/73	LOADING	11/5/20-44
		CONTRACT	No. 73-191
		DRAWING	3-302A-1





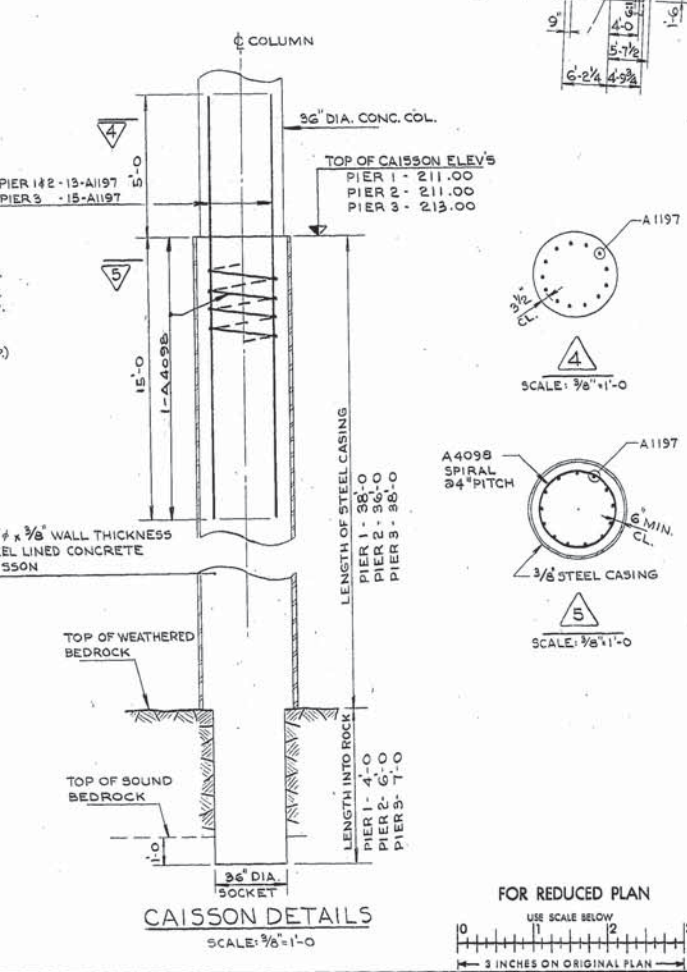
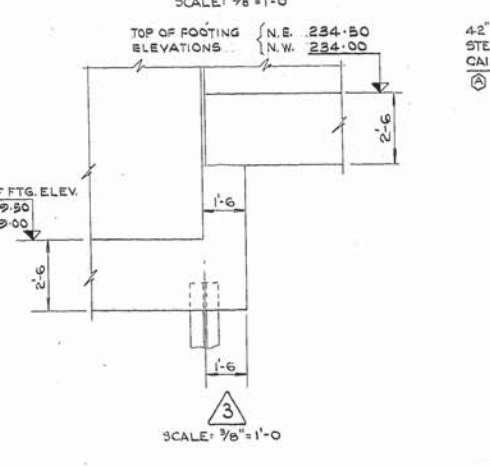
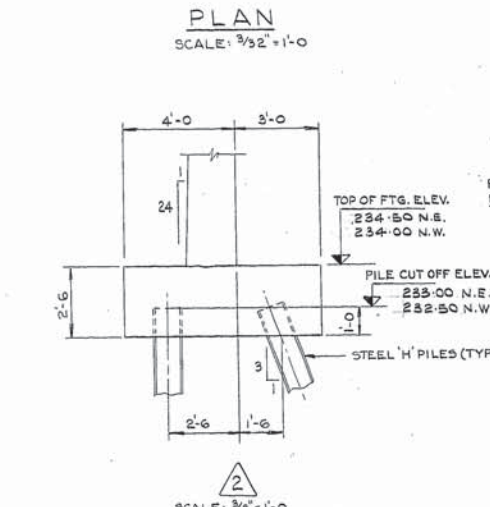
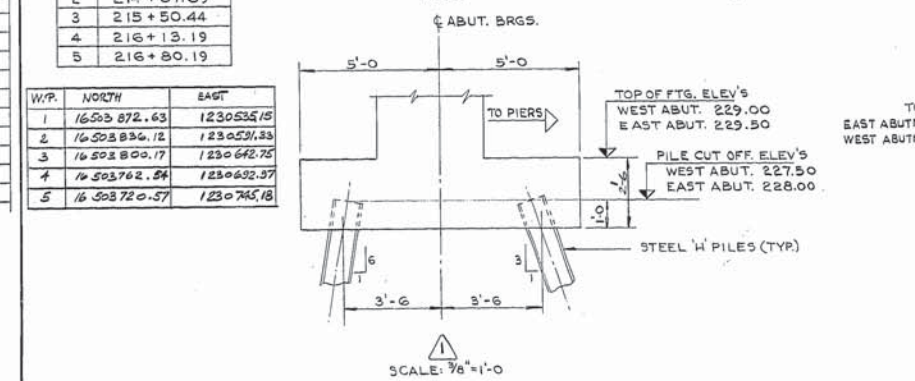
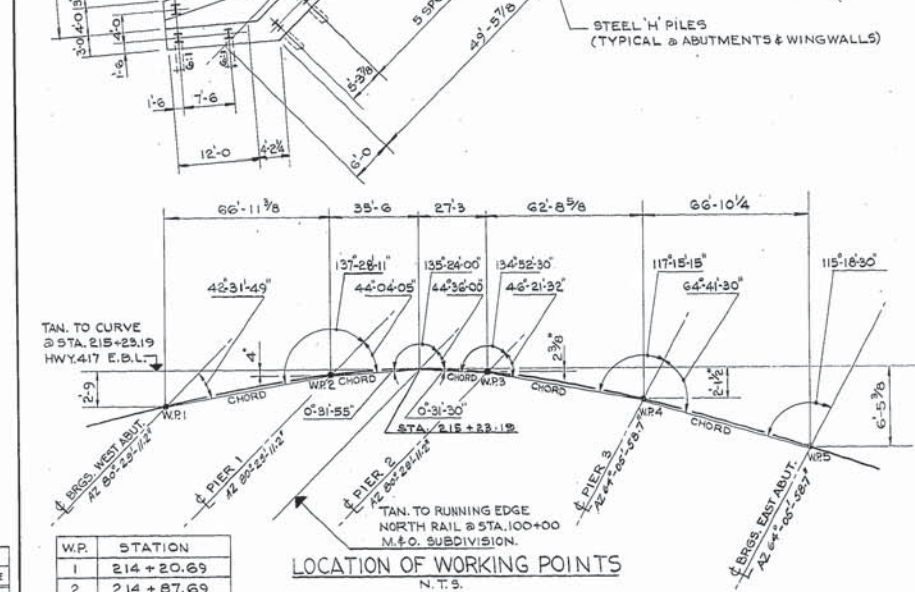
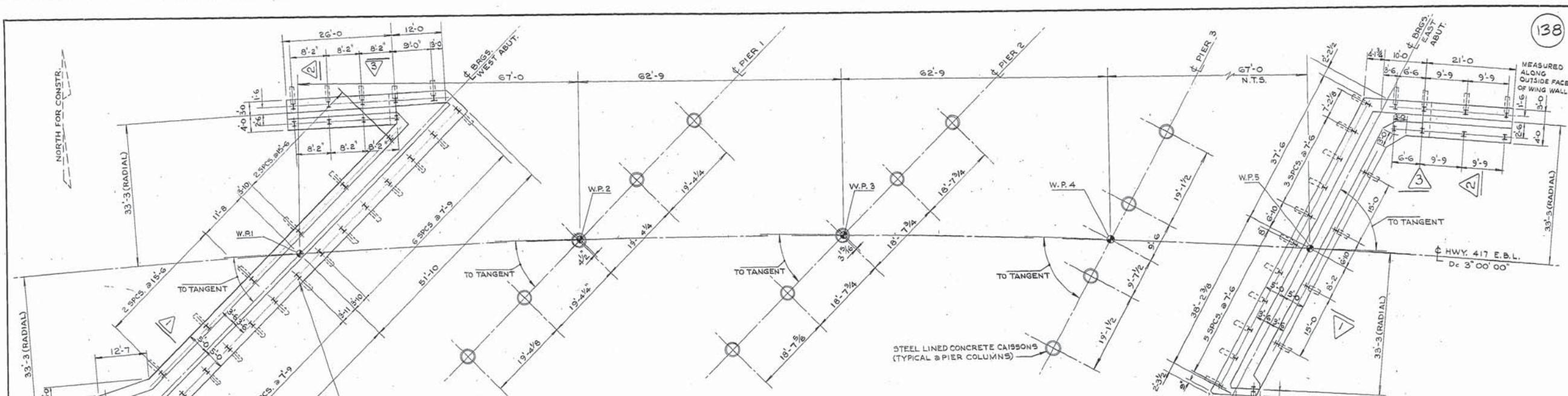
LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation, November 1971.		
	Piezometric Water Levels		
		CO-ORDINATES	
NO.	ELEVATION	NORTH	EAST
1	209.9	16, 503, 918	1, 230, 442
2	210.3	16, 503, 878	1, 230, 571
3	211.1	16, 503, 826	1, 230, 546
4	211.2	16, 503, 788	1, 230, 600
5	210.1	16, 503, 798	1, 230, 680
6	211.6	16, 503, 742	1, 230, 670
7	211.3	16, 503, 738	1, 230, 755
8	210.1	16, 503, 885	1, 230, 570
9	209.2	16, 503, 872	1, 230, 780
10	210.8	16, 503, 790	1, 230, 788
11	210.3	16, 503, 778	1, 230, 886
12	211.2	16, 503, 705	1, 230, 936



**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS DESIGN SERVICES BRANCH—FOUNDATION OFFICE			
BRIDGE No.2 C.P.R. OVERHEAD			
HIGHWAY NO. 417 E.B.L.		DIST. NO. 9	
REG. MUNICIPALITY OF OTTAWA - CARLETON			
TWP. _____		LOT _____ CON. _____	
BORE HOLE LOCATIONS & SOIL STRATA			
SUBMD. W.H. CHECKED <i>JA</i>	W.P. NO. 13-68-06	DRAWING NO. 71-11126 A	
DRAWN S.R. CHECKED <i>JA</i>	JOB NO. 71-11126	BRIDGE DRAWING NO. 3-302A-2	
DATE JANUARY 14, 1972		SITE NO. 3-302A	
APPROVED <i>Alfred</i>		CONT. NO. 73-191	
PRINCIPAL FOUNDATION ENGINEER			



STEEL 'H' PILE DATA			
LOCATION	NO.	LENGTH	TYPE
WEST ABUT.	26	60'-0"	HP 12 x 74 (TYP.)
EAST ABUT.	20	56'-0"	
N.E. RET. WALL	4	62'-0"	
N.W. RET. WALL	6	64'-0"	

- NOTES:
1. PILES TO BE DRIVEN TO BEDROCK.
  2. SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTING.
  3. THE 42" CAISSONS SHALL BE DRIVEN WITH THE MAXIMUM TOLERANCE OF 1/8" FROM THE SHOWN LOCATIONS.



REVISIONS	
DATE	DESCRIPTION
DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO <b>DeLuw, Cather</b> ENGINEERS & PLANNERS - OTTAWA	
C.P.R./C.N.R. OVERHEAD - E.B.L. (1.5 MILES SOUTH OF O.Q.W.) BRIDGE No. 2	
KING'S HIGHWAY No. 417 DIST. No. 9	
CO. REG. MUN. OF OTTAWA-CARLETON	
TWP. GLOUCESTER LOT 22 CON. 3	
FOUNDATION PLAN	
APPROVED	SITE No. 3-302A W.P. No. 13-68-06
DESIGN S.F. CHECK C.S.S.	CONTRACT No. 72-191
DRAWING F.R.G. CHECK R.G./P.S.	DRAWING No. 3-302A-3
DATE FEB/79	LOADING H320-44



57013 TWP#56-302A-3-A (B)



**APPENDIX 10**  
**SITE 3-302/2**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Justin Gray  
(Reviewed by Paul Carnaffan, P.Eng. and  
P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 WESTBOUND CANADIAN PACIFIC RAILWAY OVERPASS (SITE 3-302/2) GWP 4074-11-00 GEOCRETS 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing Canadian Pacific Railway (CP) overpass of the westbound lanes of Highway 417 in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 267-00-02
- Site No. 3-302/2
- GEOCRETS No. 31G5-80
- Construction Contract 73-191
- Historic W.P. 13-68-07

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa where the westbound Highway 417 crosses over the CP railway tracks approximately 800 m south of the Highway 417 / Innes Road Interchange. Based on the historic General Plan Drawing (copy attached), the bridge is a 17.4 m wide, and 64.0 m long, three-span, concrete girder structure. The main track, an abandoned siding and another track identified as the east leg of the WYE track are all located beneath the middle span. A National Capital Commission access road is located between Pier 2 and the east abutment.

The bridge abutments and wingwalls are supported by steel HP12x74 piles driven to bedrock. The piers are supported by 42 inch diameter steel lined concrete caissons with 36 inch diameter



sockets keyed 0.3 m into sound bedrock. The caissons supporting the piers do not have a pile cap as they extend up above the ground surface as columns.

The natural terrain in the vicinity of the bridge is generally flat with elevations ranging from 64.0 to 65.5 m. The design drawings show that the approach fills were to be constructed by placing as much as 11 m of fill at slopes ranging from 2H:1V to 4H:1V (Horizontal:Vertical)

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-80 dated February 1972. The investigation consisted of five sampled boreholes, all of which were accompanied by dynamic cone penetration tests. Drawing No. 71-11126A (copy attached) illustrates the locations of the bridge, the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by silty sand with some gravel, underlain by shale bedrock.

#### **3.1 Silty Sand**

A silty sand with some gravel deposit was encountered in all boreholes advanced at the site. The surface of this deposit ranged from 63.8 to 64.4 m in elevation, and the layer had a thickness of 9.7 to 10.1 m. The standard penetration 'N' values varied greatly for this deposit ranging from 3 to 96 blows per 0.3 m penetration; indicating a very loose to very dense condition. Gradation test results on samples of this material indicate a gravel content between 0% and 32%, sand content between 11% and 97% and a fines content (combined silt and clay content) ranging from 1% to 89%.

#### **3.2 Bedrock**

Beneath the silty sand layer, a grey shale bedrock was encountered with surface elevations ranging from 53.4 to 54.5 m. The upper 1.4 to 2.7 m of the shale was generally fractured and weathered. At greater depths the shale was described as being in a sound condition. Geological mapping suggests that this site is near the boundary between the Carlsbad and Billings bedrock formations.

#### **3.3 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 62.5 and 63.0 m.

### **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in July 2012 for Bridge 3-302/2 with the report issued September 2013. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in fair to good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.



The site was inspected by Thurber Engineering staff during the week of July 14<sup>th</sup>, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

- No evidence of slope stability issues were noted; the slopes on the approach embankments were well vegetated
- Evidence of erosion was noted beneath both abutments; no erosion protection system was present beneath either abutment
- A crack in the pavement surface at the end of the approach slab to the west was observed
- A slight dip in the road surface just before the east approach slab was observed
- No evidence of settlement of the piers were observed
- The top of the pile cap at the east abutment was exposed (see photos)

## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

It is noted that the Foundation Design Report for this structure recommended a minimum of 4 feet (1.2 m) of soil cover for frost protection. The soil cover for the abutments should be reviewed and where insufficient earth cover is provided, polystyrene insulation may be used to enhance existing frost protection measures.

#### **5.2 Seismic Considerations**

This site is classified as a Soil Profile Type I in accordance with the Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC). The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the reported density of the silty sand and the measured groundwater at the site at the time of the investigation, these materials are classified as “not susceptible” to liquefaction during the design earthquake event.

#### **5.3 Existing Foundations**

The Foundations Design Report concluded that the loose to compact granular soil conditions present at shallow depth at the site would not provide for an economical spread footing type foundation. In addition, concerns were raised regarding dewatering and basal stability for excavations for footings or pile caps at the pier locations.

As per Foundation Plan Drawing (copy attached) the bridge abutments and retaining walls were designed to be supported on steel HP12x74 piles driven to bedrock. The available construction



drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the piles is 95 tons / HP12x74 pile or approximately 845 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN / HP310x110 pile driven to bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

The piers are supported by 42 inch diameter steel lined concrete caissons with 36 inch diameter sockets extending 1 foot (0.3 m) into sound bedrock (the total length of the sockets ranges from 6 to 8 feet (1.8 to 2.4 m)). The available contract drawings do not indicate the design loads for the caissons, however, the Foundation Design Report indicates that for the caissons socketed at least 1 ft. into sound bedrock, a 30-inch diameter caisson may be designed with an allowable load of 250 tons/caisson or approximately 2200 kN/caisson. For a 36-inch diameter caisson the recommended allowable load increases to 3150 kN/caisson.

## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Deep Foundations**

The design for replacement or widening of the existing structure will need to consider the potential interaction with the existing structure and compatibility with the existing foundation systems.

#### **6.1.1 Integral Abutment Considerations**

As per the Foundation Plan Drawing the existing abutments and retaining walls are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

#### **6.1.2 Deep Foundations - Caissons**

For preliminary design purposes, the factored vertical geotechnical resistance for 36 inch (914 mm) diameter caissons keyed 300 mm into sound shale is 3150 kN/caisson. The SLS condition will not govern for caissons socketed into sound bedrock. The geotechnical resistance could be increased by increasing the socket length within the sound bedrock.



### 6.1.3 Deep Foundations - Driven Steel Piles

Steel piles (Grade 350 W steel) end-bearing on sound bedrock at this site may be designed on the basis of the following factored, vertical, geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

### 6.1.4 Pile Tips

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### 6.1.5 Pile Spacing

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### 6.1.6 Downdrag

In light of the cohesionless makeup of the existing overburden soils at this site, downdrag on existing and new piles is not considered a design issue.

### 6.1.7 Lateral Resistance of Deep Foundations

The lateral resistance of both existing and new driven piles/bored caissons can be assessed based on the method outlined in the CHBDC. For a driven H-pile or bored caisson, the lateral soil resistance may be calculated using the following formula.

$$k_h = \frac{n_h z}{B}$$

where:

$k_h$  = coefficient of horizontal subgrade reaction

$n_h$  = coefficient related to soil density given in Table A

$z$  = depth of pile/caisson embedment (m)

$B$  = pile/caisson width perpendicular to load direction (m)

**Table A:**  $n_h$  values for cohesionless soils.

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )
Above 64	Embankment Fill	3,000
Between 64 to 54.3	Silty Sand with some Gravel	2,000
Below 54.3	Weathered Shale Bedrock	6,000



Note that the weathered shale has been treated as a cohesionless soil for a conservative assessment of the lateral resistance.

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.1.8 Uplift Resistance of Deep Foundations

The unfactored uplift resistance of new or existing piles/caissons can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)



**Table C:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 64	Embankment Fill	20	0.4
Between 64 to 54.3	Silty Sand with some Gravel	19	0.4

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

## 6.2 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D: Static Lateral Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	35°	30°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.50
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33

### 6.3 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I / Silty Sand
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20 / 19
Angle of Internal Friction, $\phi$	35°	30°
<b>Yielding Wall</b>		
$K_{AE}$	0.33	0.40
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36
<b>Non-Yielding Wall</b>		
$K_{AE}$	0.55	0.66
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:



$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth, d (kPa)

d = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

H = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors provided in Table E are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## 6.4 Approach Embankments

Based on General Plan Drawing, the approach embankments should consist of compacted, boulder free fill to a maximum height of 11.3 m above the original ground surface constructed with slopes ranging from 2H:1V to 4H:1V. The Foundation Design Report states that the settlement would be elastic in nature with a predicted total settlement of up to 2 inches, with the majority of the settlement taking place during or immediately after the construction of the embankments. The embankment foundation is expected to be stable and no long term settlement problems are expected unless the fills are reconfigured.

## 6.5 Erosion Control

Active erosion beneath both abutments was noted as no erosion protection systems were present at the site.

The eroded slopes in front of the abutments should be reinstated, erosion protection measures incorporated and the drainage measures enhanced beneath the abutments to prevent further erosion of the embankment material.

## 6.6 Excavations and Backfill

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native overburden soils reported at this site should be classified as Type 2 above and Type 4 below the groundwater table in accordance with OHSA.



Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that monitoring wells be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

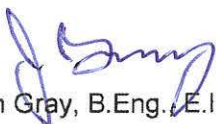
During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one. In addition, coordination with the railway, railway design code requirements and track protection will need to be considered.



## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.

  
Justin Gray, B.Eng., E.I.T.

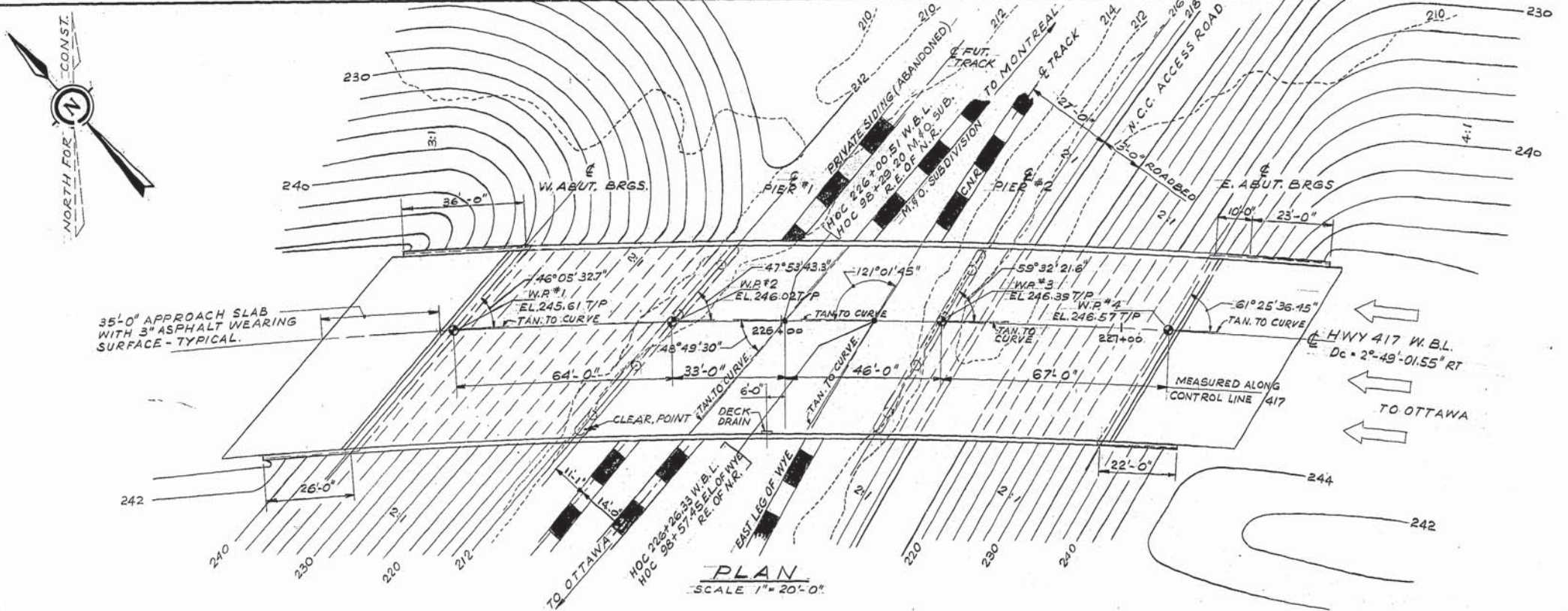


Paul Carnaffan, P.Eng.  
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

Attachments



REFERENCE BENCH MARK

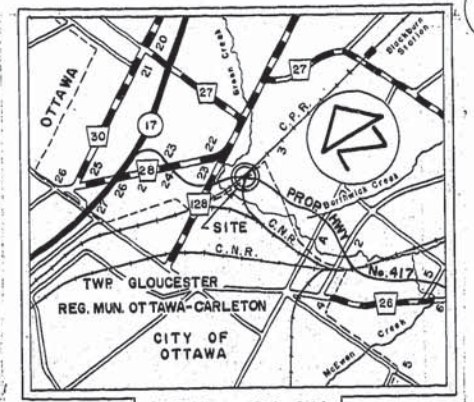
B.M. 220.52 GEODETIC DATUM  
TOP NUT ON N.E. CORNER OF HYDRO  
TOWER 350 FT OF STA. 212+00 HWY 417 E.B.L.

CURVE DATA

HWY 417 W.B.L.  
A = 67°44'40"  
Ac = 55°04'03.03"  
D = 2°49'01.55"  
R = 2033.86'  
Lc = 1954.76'  
Es = 259.78'  
Ts = 1060.28'  
Ls = 450'  
Bs = 6°20'18.49"

NOTES

- W.P. DENOTES WORKING POINT  
- T/P DENOTES TOP OF ASPHALT  
PAVEMENT.



KEY PLAN  
SCALE 1 IN = 1 MI.

LIST OF DRAWINGS

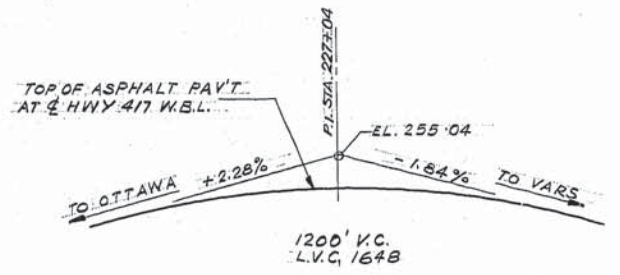
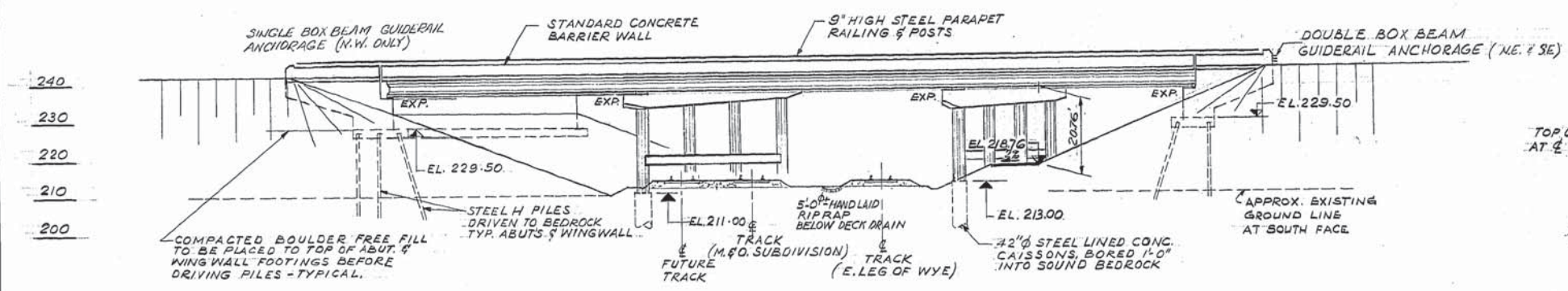
1. GENERAL PLAN.
2. BORE HOLE LOCATIONS & SOIL STRATA.
3. FOUNDATION PLAN.
4. WEST ABUTMENT - DETAILS.
5. EAST ABUTMENT - DETAILS.
6. WEST ABUTMENT - REINFORCEMENT.
7. EAST ABUTMENT - REINFORCEMENT.
8. PIER DETAILS & REINFORCEMENT.
9. PRESTRESSED GIRDERS & BEARINGS.
10. DECK DETAILS & REINFORCEMENT.
11. APPROACH SLABS.
12. CONCRETE BARRIER WALL (2'-8" HIGH).
13. DETAILS OF 9" HIGH STEEL PARAPET RAILING.
14. STANDARD DETAILS I
15. STANDARD DETAILS II
16. PLAN - EMBEDDED DETAILS
17. EMBEDDED DETAILS

NOTES

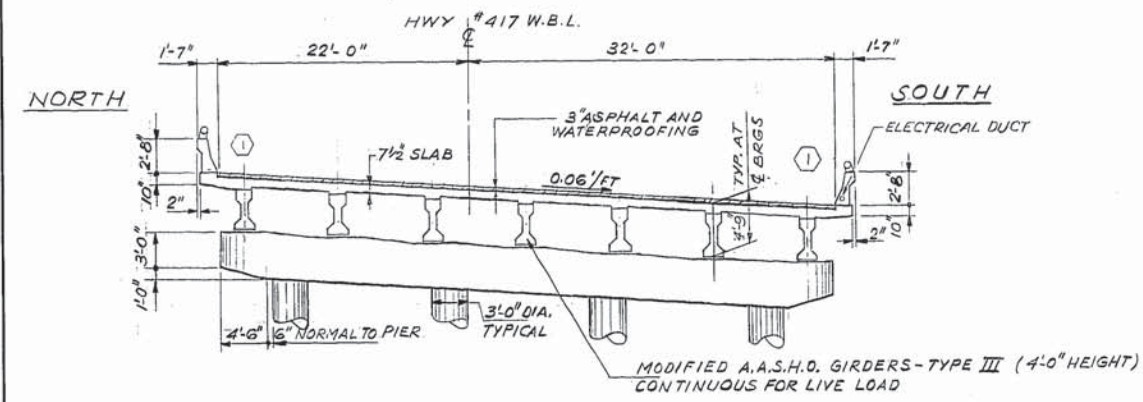
CLASS OF CONCRETE  
PRESTRESSED CONCRETE GIRDER - 5000 P.S.I.  
APPROACH SLABS - 4000 P.S.I.  
DECK & BARRIER WALLS - 4000 P.S.I.  
CAISSONS, COLUMNS AND REMAINDER - 3000 P.S.I.

2. CLEAR COVER ON REINFORCING STEEL  
FOOTINGS 3" ABUTMENTS & PIERS 3"  
DECK: TOP 1 1/2", BOT. 1", DIAPHRAGMS 1 1/2"  
EXCEPT AS NOTED.

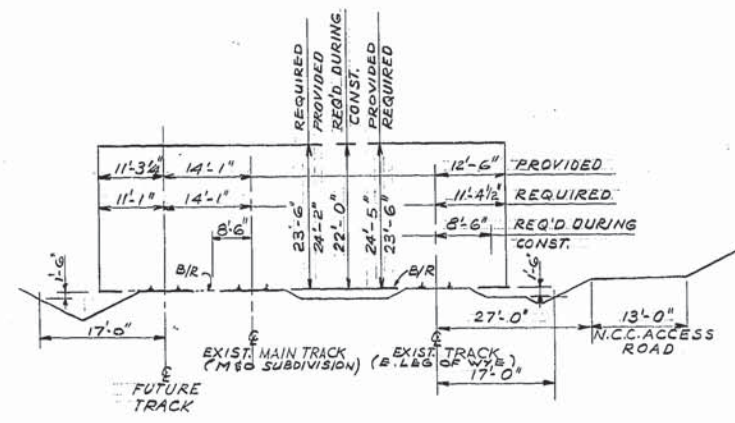
3. CONSTRUCTION NOTES  
THE CONTRACTOR IS RESPONSIBLE FOR FINISHING  
THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED  
ELEVATIONS WITH A TOLERANCE OF ± 1/8"  
NO CONCRETE SHALL BE PLACED ABOVE THE  
ABUTMENT BEARING SEATS, UNTIL THE CONCRETE  
IN THE DECK HAS BEEN PLACED.



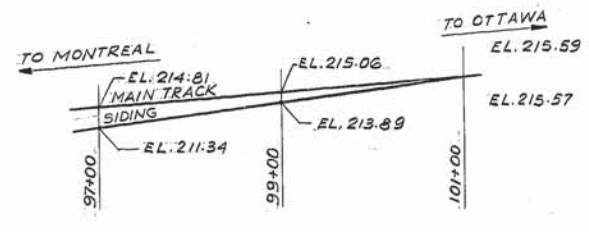
PROFILE HWY 417 W.B.L.  
N.T.S.



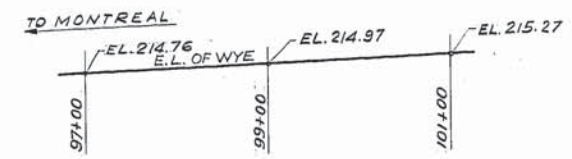
TYPICAL DECK SECTION  
SCALE 1/8"=1'-0"



RAILWAY CLEARANCE DIAGRAM  
N.T.S.



PROFILE OF M. & O. SUBDIVISION  
(BASE OF NORTH RAIL)

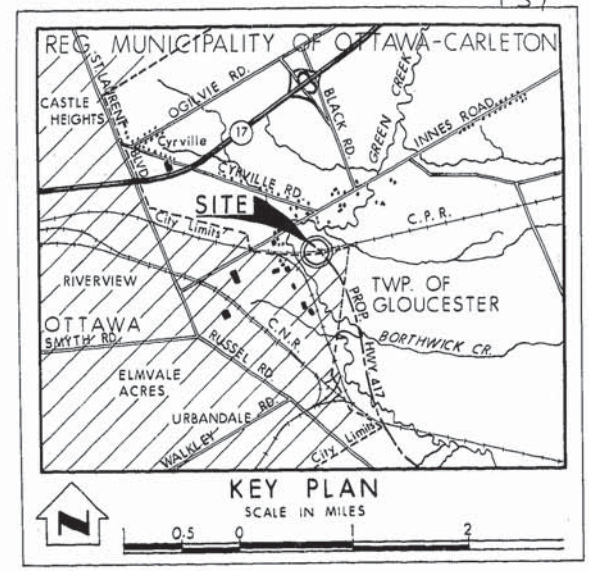
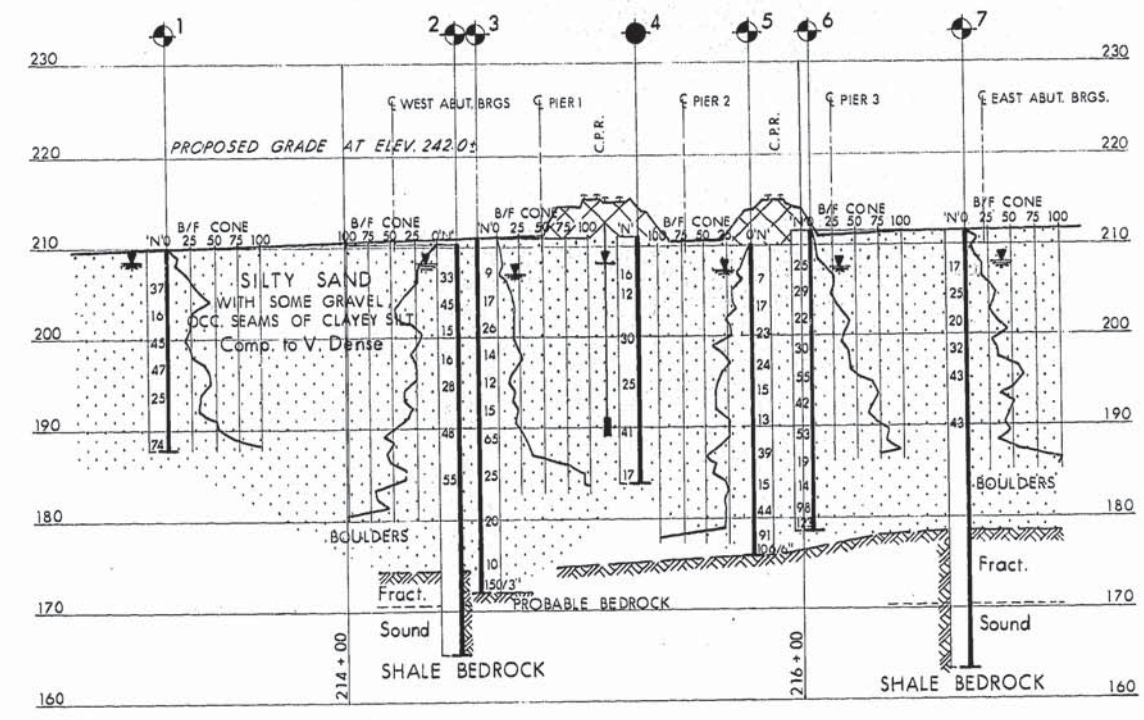
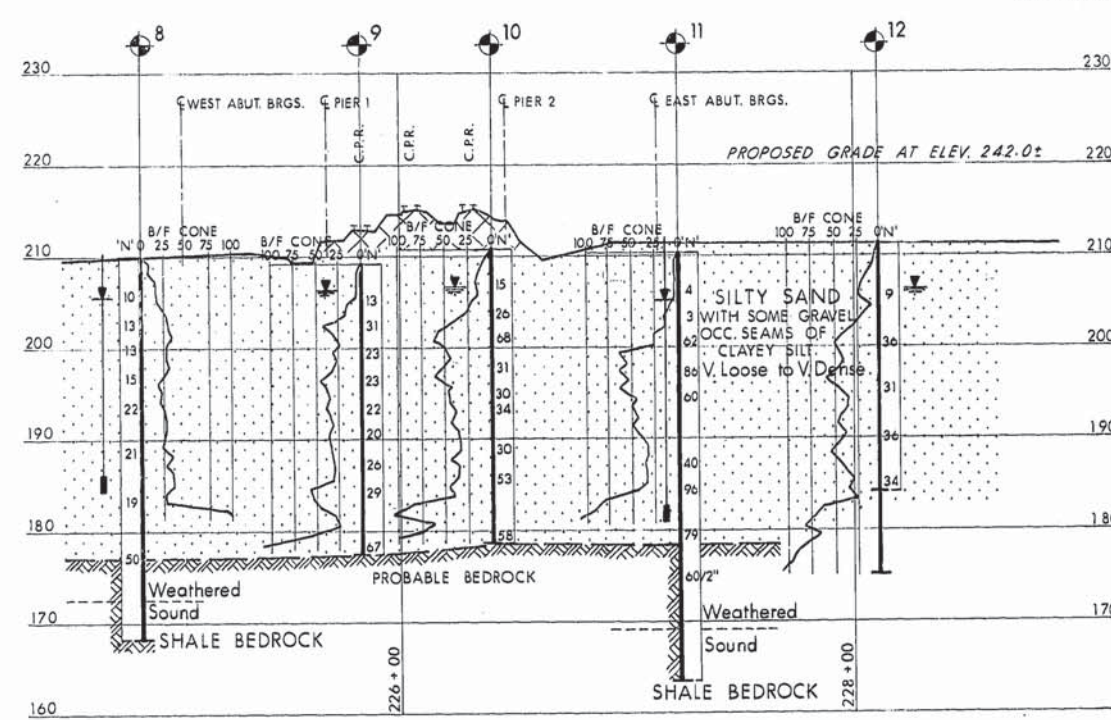
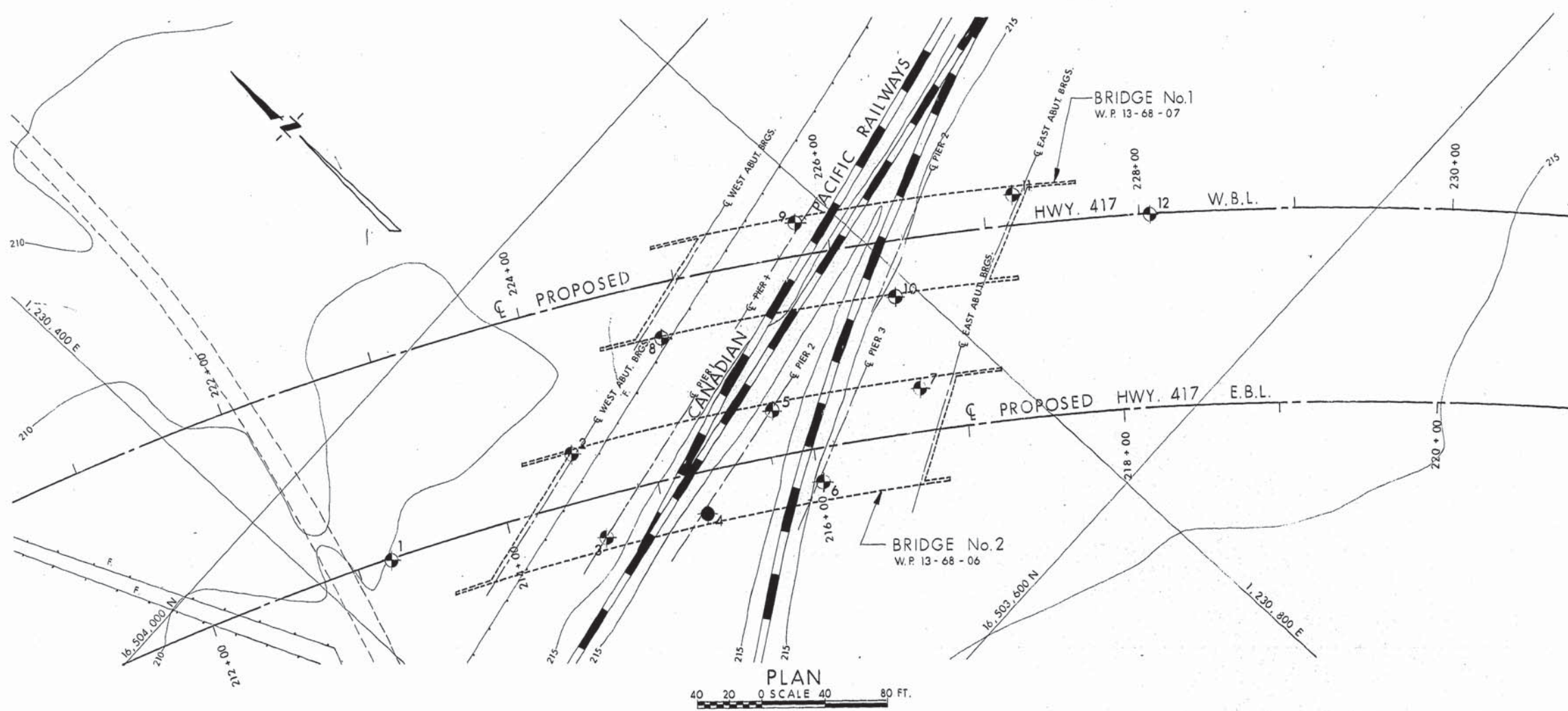


PROFILE OF EAST LEG OF WYE  
(BASE OF NORTH RAIL)

FOR REDUCED PLAN  
USE SCALE BELOW



REVISIONS	
DATE	DESCRIPTION
HAR/74 S.S. (1) REMOVED DIMENSION	
DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
DeLeuw, Cather ENGINEERS & PLANNERS - OTTAWA	
C.P.R./C.N.R. OVERHEAD - W.B.L. (1.5 MILES SOUTH OF O.G.W.) BRIDGE No. 1	
KING'S HIGHWAY No. 417 DIST. No. 9	
CO. REG. MUN. OF OTTAWA-CARLETON	
TWP. GLOUCESTER LOT 22 CON. 3	
GENERAL PLAN	
APPROVED	SITE No. 3-302B W.P. No. 73-68-07
DESIGN S.F. CHECK G.S.S.	CONTRACT No. 73-191
DRAWING R.G. CHECK S.F.	DRAWING No. 3-302B-1
DATE FEB. 73	LOADING AS20-44



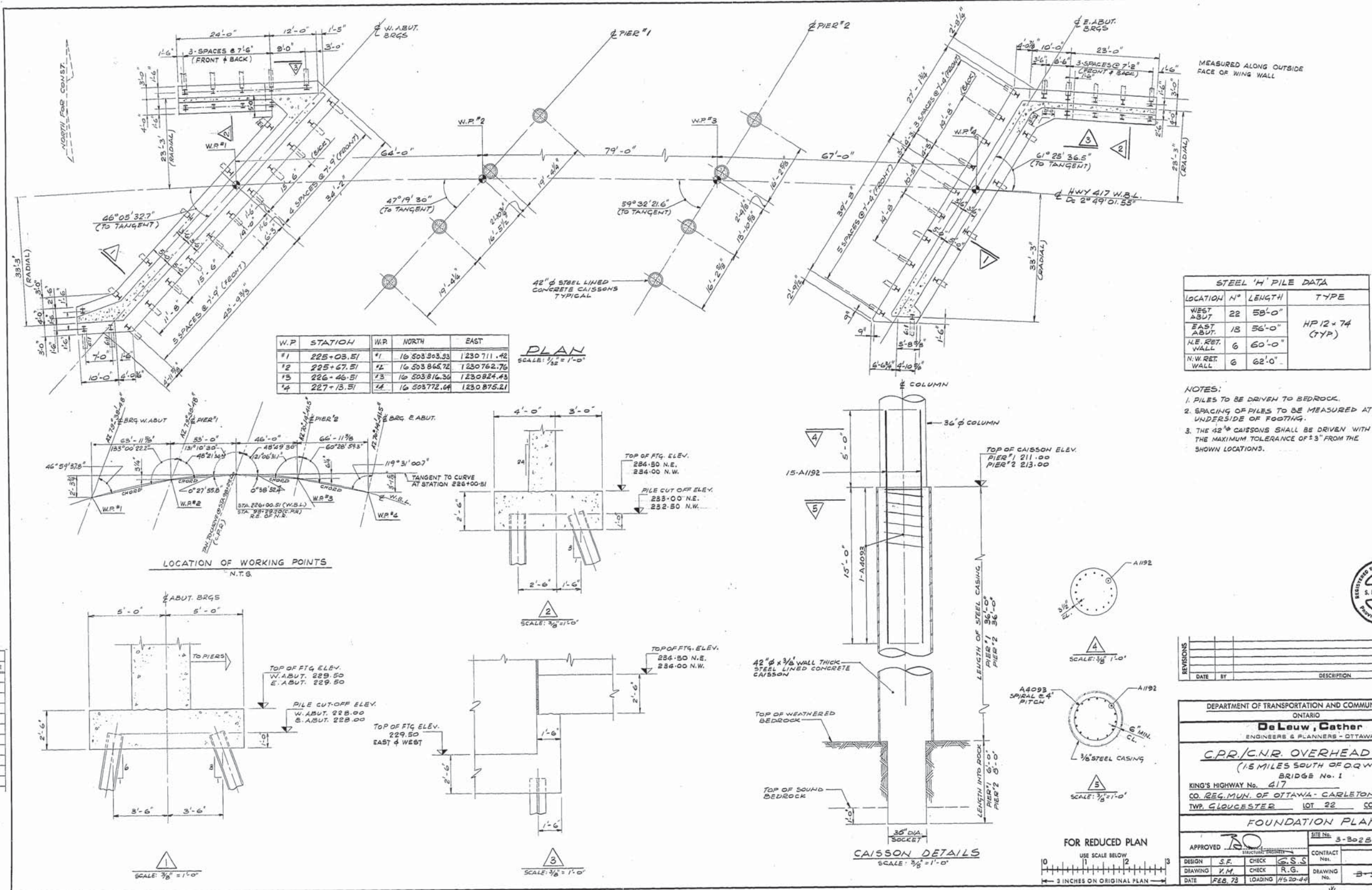
LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation, November 1971.		
	Piezometric Water Levels		
NO.	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	209.9	16,503,918	1,230,442
2	210.3	16,503,878	1,230,571
3	211.1	16,503,826	1,230,546
4	211.2	16,503,788	1,230,600
5	210.1	16,503,798	1,230,680
6	211.6	16,503,742	1,230,670
7	211.3	16,503,738	1,230,755
8	210.1	16,503,885	1,230,570
9	209.2	16,503,872	1,230,780
10	210.8	16,503,790	1,230,788
11	210.3	16,503,778	1,230,886
12	211.2	16,503,705	1,230,936

**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS		
DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS DESIGN SERVICES BRANCH—FOUNDATION OFFICE			
BRIDGE No. 2 C.P.R. OVERHEAD			
HIGHWAY NO. 417 E.B.L.		DIST. NO. 9	
REG. MUNICIPALITY OF OTTAWA - CARLETON			
TWP. _____		LOT _____ CON. _____	
BORE HOLE LOCATIONS & SOIL STRATA			
SUBMD. W.H. CHECKED <i>JA</i>	W.P. NO. 13-68-06	DRAWING NO. 71-11126 A	
DRAWN S.R. CHECKED <i>JA</i>	JOB NO. 71-11126	BRIDGE DRAWING NO. 3-302A-2	
DATE JANUARY 14, 1972		SITE NO. 3-302A	
APPROVED <i>Alfred</i>		CONT. NO. 73-191	
PRINCIPAL FOUNDATION ENGINEER			

NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the OTTAWA District Office.



REVISIONS	
DATE	DESCRIPTION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO  
**DeLeuw, Cather**  
ENGINEERS & PLANNERS - OTTAWA

**C.P.R./C.N.R. OVERHEAD - W.B.L.**  
(1.5 MILES SOUTH OF OROUW)  
BRIDGE No. 1

KING'S HIGHWAY No. 417 DIST. No. 9  
CO. REG. MUN. OF OTTAWA - CARLETON  
TWP. GLOUCESTER LOT 22 CON. 3

**FOUNDATION PLAN**

APPROVED: *[Signature]* SITE No. 5-302B W.P. No. 75-68-07  
DESIGN: S.F. CHECK: G.S.S. CONTRACT No. 73-191  
DRAWING: K.M. CHECK: R.G. DRAWING No. 5-302B-3  
DATE: FEB. 73 LOADING: 7520-44

57030 TWP# 56-302B-3-A (2)



**APPENDIX 11**  
**SITE 3-306**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Kenton Power, P.Eng.  
(Reviewed by Fred Griffiths, P.Eng. and  
P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 WALKLEY ROAD UNDERPASS (SITE 3-306) GWP 4074-11-00 GEOCRETS 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing Highway 417 Underpass of Walkley Road in Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4116-01-01
- Site No. 3-306
- GEOCRETS No. 31G5-113
- Construction Contract 73-190
- Historic W.P. 10-69-08

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa, in the Township of Gloucester approximately 600 m east of the intersection of Sheffield Road and Walkley Road. The two-span structure carries both the Walkley Road east and westbound lanes (four lanes in total plus a concrete median and sidewalks) and the on and off ramps for the Highway 417 / Walkley Road Interchange over both the east and west bound lanes of Highway 417. The terrain in the vicinity of the bridge is generally flat and is brush and grass covered. Site photos showing the general site conditions are attached.

Based on the historic General Layout Drawing (copy attached) the bridge is approximately 31 m wide, 83 m long with a cast-in-place concrete deck. The bridge abutments are supported by



steel HP12x74 piles driven to bedrock. The center pier includes four columns, each supported by steel HP12x89 piles also driven to bedrock.

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-113 dated May 1972. The original investigation was conducted in December 1971 and consisted of five sampled boreholes designated Boreholes 1, 2, 4, 5 and 7; all of which were accompanied by dynamic cone penetration tests (DCPT). Two additional DCPT designated Boreholes 3 and 6 were also carried out opposite both Boreholes 4 and 5. Additional fieldwork was undertaken in April 1972 and consisted of four sampled boreholes designated 1A to 4A; all of which were accompanied by DCPT tests. The results of the additional field work were combined with the original works and a single report was issued.

Drawing No. 71-11125A (copy attached) illustrates the locations of the bridge and the investigation boreholes, as well as the soil strata plot for both investigations. The stratigraphy in the area of the bridge is generally characterized by a silty clay stratum, overlaying a glacial till material consisting mainly of silt and sand, underlain by shale bedrock.

#### **3.1 Silty Clay**

A clay stratum was encountered near or at the ground surface in all sampled boreholes. The surface of this deposit ranged from 65.0 m to 65.7 m in elevation, and the layer had a thickness of 3.5 m to 6.4 m. Borehole 1 was terminated in this stratum. The standard penetration test (SPT) 'N' values ranged from 9 to 19 blows per 0.3 m of penetration. The estimated undrained shear strength based on in-situ field vane tests ranged from 42 kPa to greater than 110 kPa while laboratory shear strength tests results ranged from 38 kPa to greater than 121 kPa. Results of shear strength testing indicate a very stiff consistency in the upper portion of the deposit that decreases to firm with depth.

The results of a grain size analysis tests including hydrometer testing completed on seven samples of this material indicated a gravel content of 0% for all samples tested, sand content from 1% to 11%, silt content from 48% to 85%, and clay content from 14% to 50%. Atterberg Limits test results of 21 samples of the silty clay material indicate a clay of low to high plasticity.

Consolidation characteristics determined for the silty clay from four oedometer tests indicate an initial void ratio from 1.0 to 1.53 and compression index from 0.16 to 0.98. The pre-consolidation pressure was found to be in excess of 210 kPa.

The moisture content of the samples tested ranged from 16% to 57%.

#### **3.2 Glacial Till**

A glacial till stratum was encountered beneath the silty clay stratum in all sampled boreholes except Borehole 1. The till was described as a heterogeneous mixture of silt, sand and gravel with trace clay. A boulder zone was reported in the lower part of the till in Boreholes 1A and 5. Coring was required to penetrate this layer. The surface of this deposit ranged from 59.3 m and



61.2 m in elevation, and the layer had a thickness of 2.8 m to 5.2 m. SPT 'N' values ranged from 5 to more than 100 blows per 0.3 m of penetration, indicating a loose to very dense condition but typically dense.

The results of a grain size analysis tests including hydrometer testing completed on seven samples of this material indicated a gravel content from 6% to 41%, sand content from 18% to 39%, silt content from 22% to 54%, and clay content from 5% to 22%. Atterberg Limits test results of three samples of the fines portion of this material indicate a silty clay material.

The moisture content of the samples tested ranged from 7% to 28%.

### **3.3 Bedrock**

A grey shale bedrock was encountered beneath the till in Boreholes 2, 4, 5, 1A and 2A as proven by BX size coring; and DCPT refusal on probable bedrock in the remaining. The bedrock surface ranged in elevation from 55.3 m to 57.2 m.

Bedrock core recovery ranged between 10% and 100%. The bedrock was described to be in sound condition. Geological mapping suggests the bedrock at this site is shale of the Carlsbad Formation.

### **3.4 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 64.9 m to 65.7 m.

## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in July 2012 for Bridge 3-306 with the report issued September 2013. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering Ltd. staff during the week of July 14, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

East and West Abutments:

- Concrete slope paving was installed for erosion control of the abutment slopes
- Settlement of the slope paving was noted in several sections on both abutment slopes
- Abutment slopes were measured at approximately 26° or approximately 2H:1V (Horizontal:Vertical)
- Bare patches with minor erosion where embankment vegetation had not taken hold was noted on the west embankment
- In general the embankment slopes were noted to be good condition



- Rust staining and spalling of concrete were noted on the underside of the bridge deck and on the abutment foundations

Pier:

- Could not be safely accessed and as such the pier was visually inspected from the west abutment
- No obvious signs of foundation settlement were observed
- Ditchline was vegetated
- Slight rust staining of the pier columns was noted but no concrete spalling was observed

Bridge and Road Surface:

- Rust staining and concrete spalling of the barriers, sidewalks and on the sides of bridge deck were noted
- Frequent longitudinal and transverse cracking of the asphalt surface was noted
- Some potholes, patching and crack repair of the asphalt surface were noted

## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

Where insufficient earth cover is provided, polystyrene insulation may be used to enhance frost protection.

#### **5.2 Seismic Considerations**

This site is best classified as a Soil Profile Type I in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC). The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the density of the till and the plasticity of the clay, these materials are classified as “not susceptible” to liquefaction during the design earthquake event.



### **5.3 Existing Foundations**

As per the Footing Layout and Reinforcement Drawing (copy attached), the bridge abutments were designed to be supported on steel HP12x74 piles driven to bedrock; while the four center pier columns were to be supported by steel HP12x89 piles also driven to bedrock.

The available construction drawings do not indicate the design loads or grade of steel used for the piles. However, the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended design allowable load for the pier foundations is 95 tons / 14BP74 pile or approximately 845 kN/pile. Due to the effects of negative skin friction the Foundation Design Report recommended that abutment piles be designed with a reduced allowable design load of 80 tons / 14BP74 pile or approximately 700 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles; while HP12x89 piles are nominally equivalent to HP310x132 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS for piles driven to bedrock of 2000 kN / HP310x110 pile, and 2400 kN / HP310x132 pile. The SLS condition will not govern for piles end-bearing in or on bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Shallow Foundations**

The very stiff to firm silty clay deposit may not provide sufficient geotechnical resistance for footings to carry a bridge structure. The depth of the top of the till deposit was observed to range from 3.5 m to 6.4 m below original grade which is considered relatively deep for spread footing foundations. As such spread footings within the overburden are not recommended and deep foundations are preferred at this site.

### **6.2 Deep Foundation – Piles**

Based on the review of existing subsurface stratigraphy at the site driven piles are considered to be suitable for the support of abutment and pier foundations. It should be noted that the bedrock surface elevation ranges from 55.3 m to 57.2 m.

#### **6.2.1 Axial Resistance**

Steel piles (Grade 350 W steel) end-bearing on sound shale bedrock at this site may be designed on the basis of the following factored vertical geotechnical resistances at ULS:



- 2,000 kN per HP310x110 pile; and
- 2,400 kN per HP 310x132 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

### **6.2.2 Pile Tips**

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### **6.2.3 Integral Abutment Considerations**

As per the Footing Layout and Reinforcement Drawing the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### **6.2.4 Pile Spacing**

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### **6.2.5 Downdrag**

Downdrag forces need to be considered for piles in areas where grades have increased from original. This includes the piles supporting the existing abutments. It is anticipated that the design of new piles required to support a widened foundation at the abutments will also need to include downdrag loads due to approach fill widening. Downdrag loads for piles at the center pier are not anticipated unless grades are modified.

The following pile SLS dead loads were provided by the structural design team in order to determine the downdrag forces acting on Structure 3-306:

- Abutment pile loading = 330 kN / pile
- Pier pile loading = 441 kN / pile

The value of downdrag on existing and new piles should be considered for this site as follows:

Existing piles:

- The abutment piles are being subject to unfactored downdrag loads of approximately 125 kN/pile due to the original placement of as much as 7.0 m of fill at the approaches.



Placement of additional fill for embankment widening or grade raises will not add further downdrag load to these piles.

#### New Piles:

Should modifications be required to the approach fills, assuming the same SLS dead loads listed above and that any additional approach fills will match existing grades; new piles at Bridge Structure 3-306 would be subject to the following downdrag loads:

- New abutment piles will be subject to unfactored downdrag loads of approximately 125 kN/pile due to the placement of fill to address widening and/or grade raises at the approaches.
- New pier piles will not be subjected to downdrag load unless there are modifications to the grades at the pier location.

The downdrag load should be multiplied by a load factor of 1.25 as per the CHBDC Commentary Clause C6.8.4 to obtain a factored downdrag load. In accordance with Section 6.8.4 of the CHBDC and Clause C6.8.4 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag loads should not exceed the factored structural resistance of a pile. In geotechnical analysis of downdrag, live load effects should not be considered.

#### 6.2.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formulae:

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

For cohesive soils:

$$k_h = \frac{67S_u}{B}$$

where:

- $k_h$  = coefficient of horizontal subgrade reaction
- $n_h$  = coefficient related to soil density given in Table A
- $z$  = depth of pile embedment (m)
- $B$  = pile width perpendicular to load direction (m)
- $S_u$  = undrained shear strength given in Table A

**Table A:**  $n_h$  Values for Cohesionless Soils and  $S_u$  Values for Cohesive Soils

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )	$S_u$ (kPa)
Above 65	Embankment Fill	3000	-
Between 65 and 60	Native Clay	-	100 to 50
Below 60	Native Glacial Till	2000	-

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

## 6.2.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)



The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)

**Table C:**  $\beta$  Values for Driven Piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 65	Embankment Fill	20	0.4
Between 65 and 60	Native Clay	17	0.25
Below 60	Native Glacial Till	19	0.4

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D: Static Lateral Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Silty Clay
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	17
Angle of Internal Friction, $\phi$	35°	30°	27°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.50	0.55
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33	0.38

#### 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of 250A (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Silty Clay
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	17
Angle of Internal Friction, $\phi$	35°	30°	27°
<b>Yielding Wall</b>			
$K_{AE}$	0.33	0.40	0.45
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36
<b>Non-Yielding Wall</b>			
$K_{AE}$	0.55	0.66	0.74
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43



The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors provided in Table E are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## 6.5 Approach Embankments

Based on the original Foundation Design Report, the maximum height of the fill embankments would extend 7.0 m above the grades existing at the time of the investigation. Side slopes of 2H:1V were considered to be stable. Settlement of up to 125 mm was predicted with 50% of the consolidation occurring within 12 months of fill placement.

It is anticipated that settlement of the native clay will occur should the embankments be widened or if additional lanes are added to Highway 417. Further settlement of the existing embankment and roadway may also occur due to the increase in stress caused by the project works.

## 6.6 Erosion Control

Active erosion below the slope paving was noted at both abutment location at this site. Bare patches with minor erosion where the embankment vegetation had not taken hold was also noted on the west embankment

The slope paving should be repaired where required and maintained and drainage measures should be enhanced beneath the abutments to prevent erosion below the slope paving. Bare patches within the embankment vegetation should be re-vegetated as part of construction in general accordance with OPSS 804.



## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native very stiff to firm silty clay soils reported at this site should be classified as Type 3 in accordance with the OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that piezometers be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one.

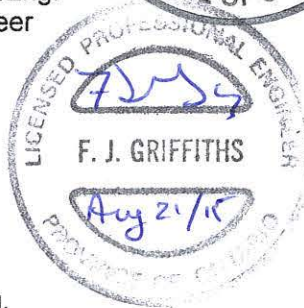
## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.



Kenton C. Power, P.Eng.  
Geotechnical Engineer

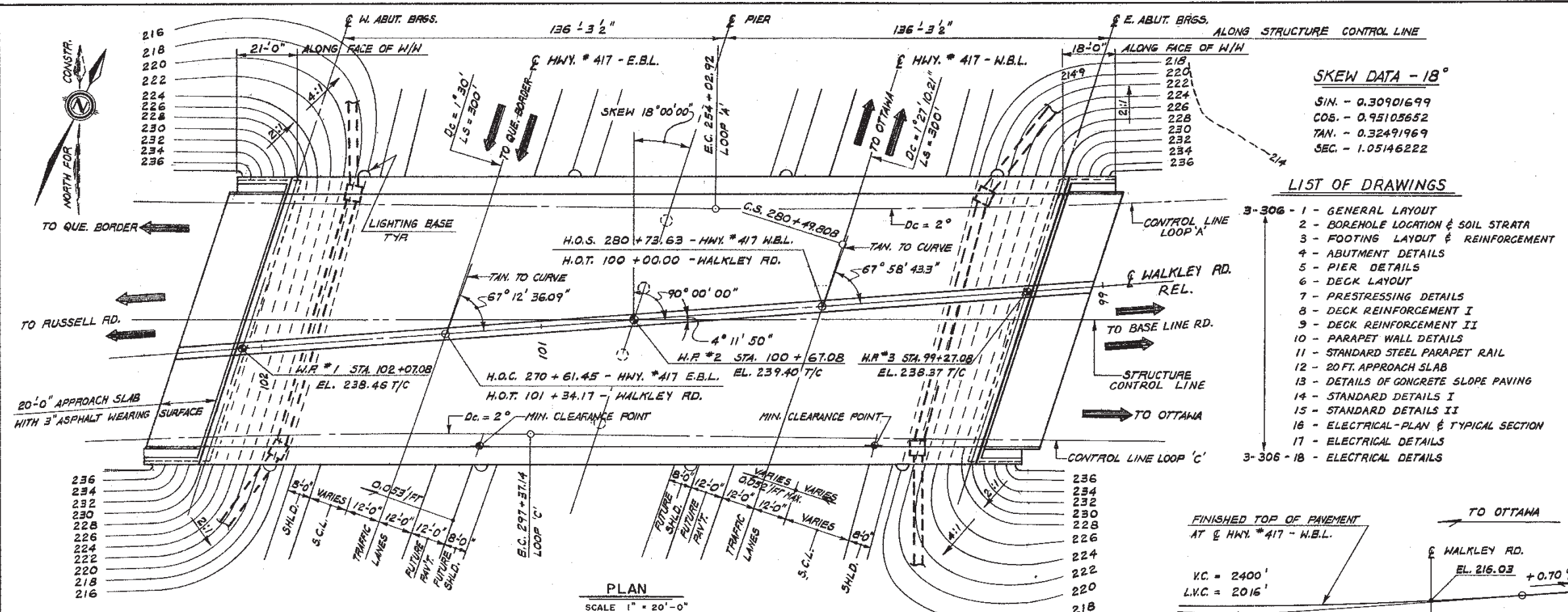


Fred Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

Attachments



**SKREW DATA - 18°**  
SIN. - 0.30901699  
COS. - 0.95105652  
TAN. - 0.32491969  
SEC. - 1.05146222

- LIST OF DRAWINGS**
- 3-306-1 - GENERAL LAYOUT
  - 2 - BOREHOLE LOCATION & SOIL STRATA
  - 3 - FOOTING LAYOUT & REINFORCEMENT
  - 4 - ABUTMENT DETAILS
  - 5 - PIER DETAILS
  - 6 - DECK LAYOUT
  - 7 - PRESTRESSING DETAILS
  - 8 - DECK REINFORCEMENT I
  - 9 - DECK REINFORCEMENT II
  - 10 - PARAPET WALL DETAILS
  - 11 - STANDARD STEEL PARAPET RAIL
  - 12 - 20 FT. APPROACH SLAB
  - 13 - DETAILS OF CONCRETE SLOPE PAVING
  - 14 - STANDARD DETAILS I
  - 15 - STANDARD DETAILS II
  - 16 - ELECTRICAL-PLAN & TYPICAL SECTION
  - 17 - ELECTRICAL DETAILS
  - 3-306-18 - ELECTRICAL DETAILS

**REFERENCE BENCH MARK**

BM 215.12  
GEODETIC DATUM  
TOP OF H.E.P.C. CONC. MONUMENT  
323.0' RT. OF 269 + 27 E.B.L.

**NOTES**

- W.P. DENOTES WORKING POINT
- T/C DENOTES TOP OF CONCRETE MEDIAN

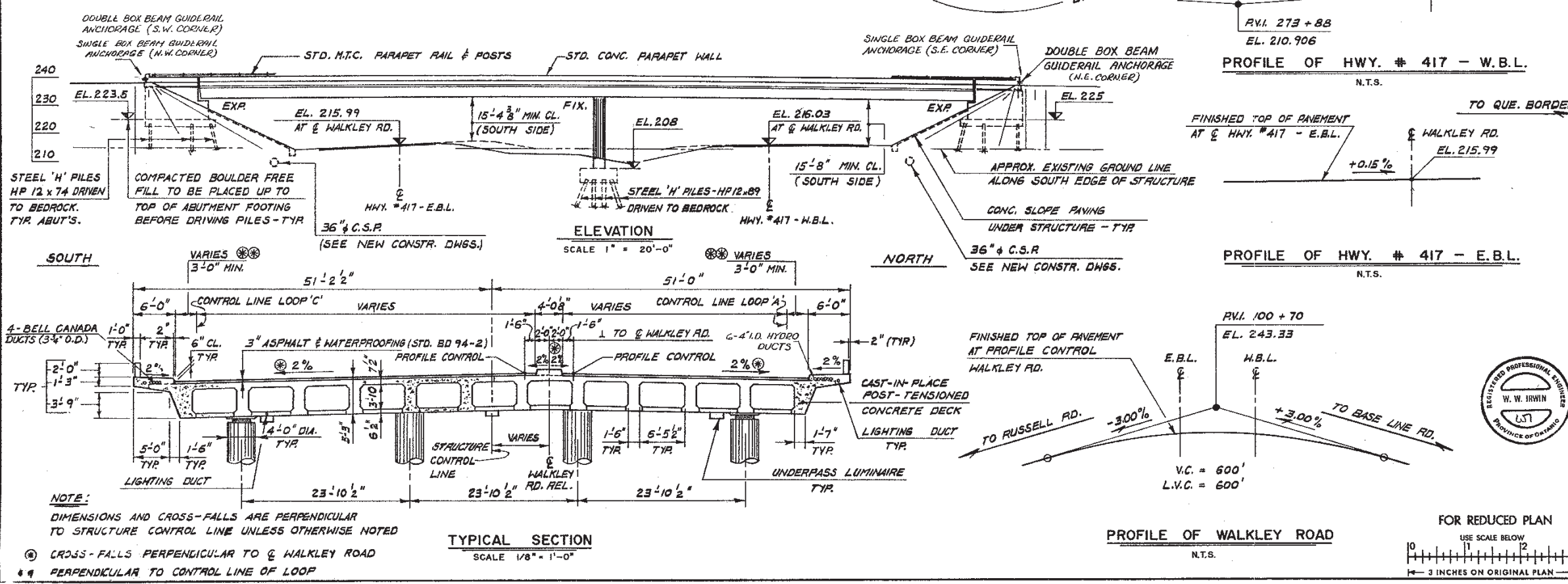
**CLASS OF CONCRETE**

DECK, SIDEWALKS, PARAPET WALLS - 5000 P.S.I.  
PIER COLUMNS & MEDIAN - 4000 P.S.I.  
PIER FOOTINGS - 4000 P.S.I.  
REMAINDER - 3000 P.S.I.  
**CLEAR COVER ON REINF. STEEL**

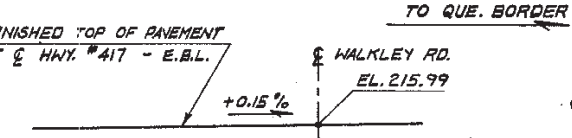
FOOTINGS & ABUTMENTS - 3'  
TOP OF DECK, OUTSIDE WEBS & PARAPET WALLS - 1 1/2'  
REMAINDER OF DECK - 1'  
PIER, SIDEWALKS & MEDIAN - 2'

**CONSTRUCTION NOTES**

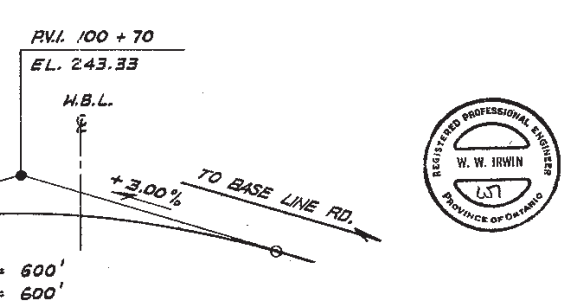
- THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 8".
- NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED, STRESSED & GROUTED.



**PROFILE OF HWY. # 417 - W.B.L.**  
N.T.S.



**PROFILE OF HWY. # 417 - E.B.L.**  
N.T.S.

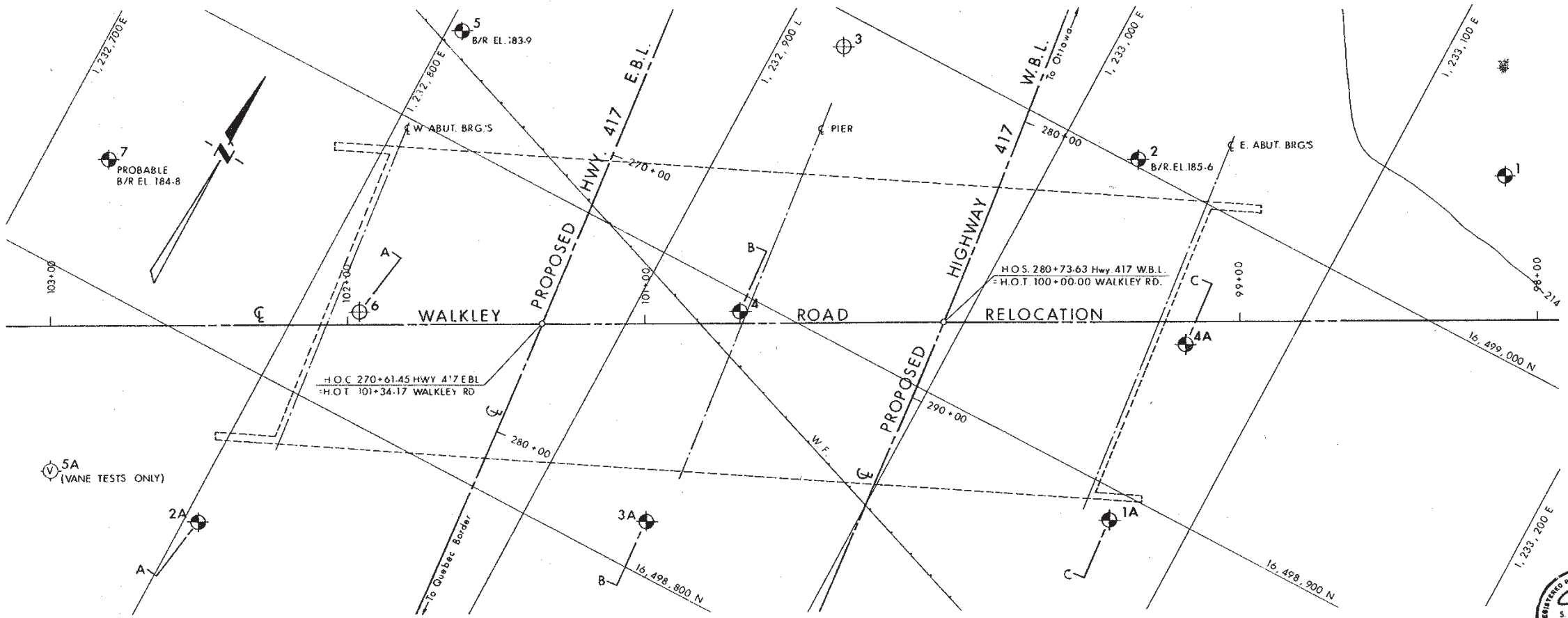


**PROFILE OF WALKLEY ROAD**  
N.T.S.

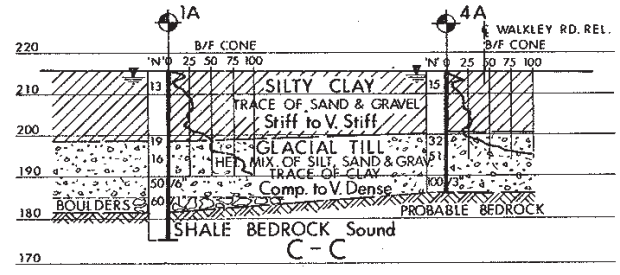
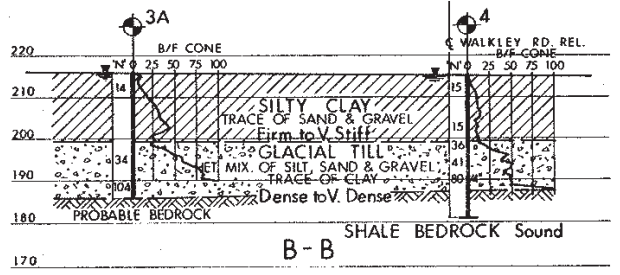
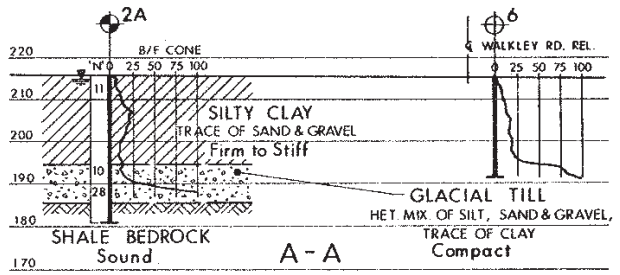
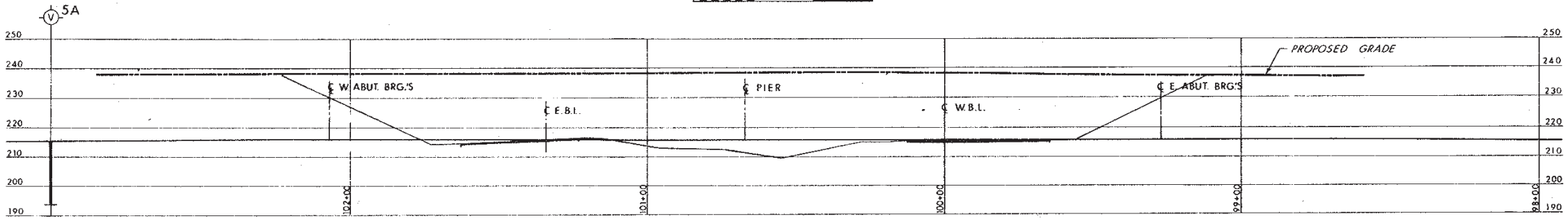


REVISIONS	
DATE	DESCRIPTION

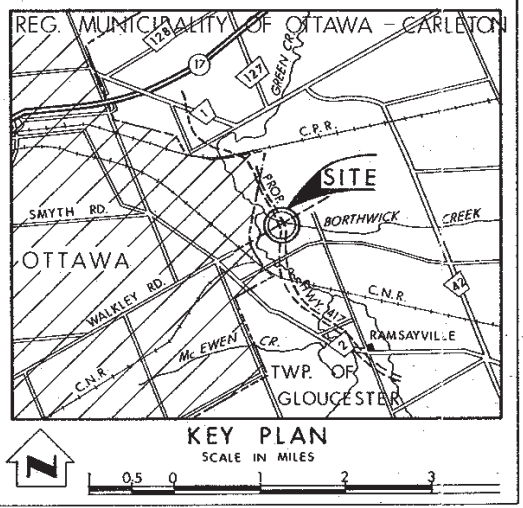
DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
Consulting Engineers & Planners	
<b>WALKLEY ROAD INTERCHANGE</b> 1.5 MILES SOUTH OF INNES RD.	
KING'S HIGHWAY No. 417	DIST. No. 9
CO. REG. MUN. OTTAWA - CARLETON	
TWP. GLOUCESTER	LOT A CON. V.I.
<b>GENERAL LAYOUT</b>	
APPROVED: [Signature]	SITE No. 3-306 W.P. No. 10-69-08
DESIGN: [Signature]	CONTRACT No.
DRAWING: C.M.B. CHECK: [Signature]	DRAWING No. 3-306-1
DATE: DEC/72	LOADING: HS20-44



PLAN  
20 10 0 SCALE 20 40 FT.



PROFILE & SECTIONS  
20 10 0 SCALE 20 40 FT.



**LEGEND**

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation. November, December 1971 & April 1972

NO.	ELEVATION	CO - ORDINATES	
		NORTH	EAST
1	213.2	16,499,055	1,233,136
1A	215.6	16,498,890	1,233,074
2	215.1	16,499,002	1,233,025
2A	215.6	16,498,746	1,232,806
3	215.0	16,498,990	1,232,920
3A	215.5	16,498,817	1,232,938
4	215.1	16,498,894	1,232,932
4A	215.7	16,498,955	1,233,069
5	214.9	16,498,934	1,232,805
5A	215.0	16,498,738	1,232,754
6	215.2	16,498,834	1,232,820
7	214.9	16,498,840	1,232,721

**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS	DATE	BY	DESCRIPTION
FEB 73	G.P.		REVISE WALKLEY RD. RELOCATION ON PLAN

MINISTRY OF TRANSPORTATION & COMMUNICATIONS  
DESIGN SERVICES BRANCH - FOUNDATIONS OFFICE

**WALKLEY ROAD RELOCATION**

HIGHWAY NO. 417 DIST. NO. 9  
REG. MUNICIPALITY OF OTTAWA - CARLETON  
TWP. GLOUCESTER LOT A CON. VI

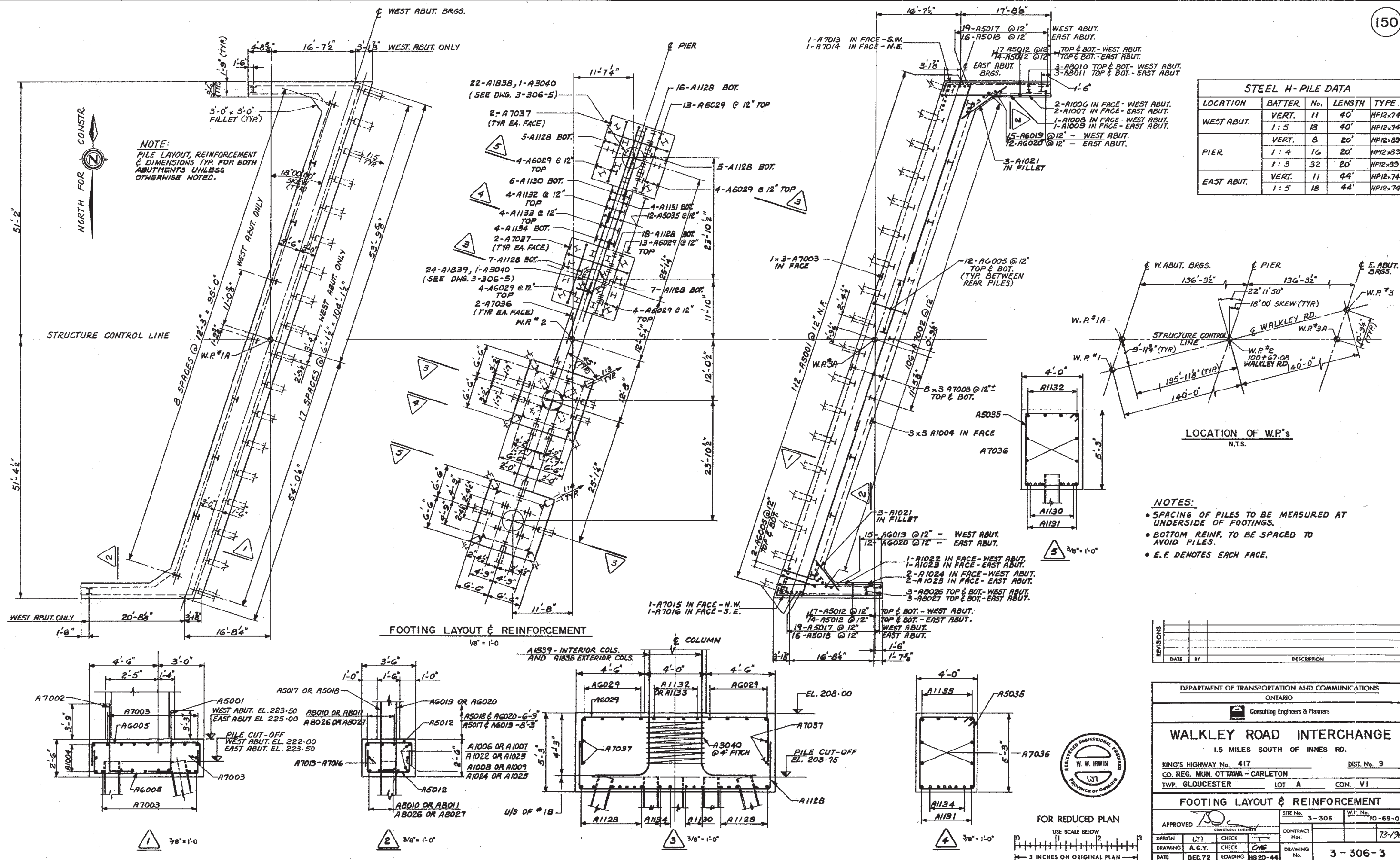
**BORE HOLE LOCATIONS & SOIL STRATA**

SUBMD. S.A.	CHECKED	W.P. NO. 10 - 69 - 08	DRAWING NO.
DRAWN S.R.	CHECKED	JOB NO. 71 - 11125	71 - 11125A
DATE MAY 12, 1972		SITE NO. 3-306	BRIDGE DRAWING NO.
APPROVED		CONT. NO. 73-190	3-306-2

PRINCIPAL FOUNDATION ENGINEER

NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the Ottawa District Office.

REF. No: E - 5228 - 1





**APPENDIX 12**  
**SITE 3-301/1**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Fred J. Griffiths, Ph.D., P.Eng.  
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 EASTBOUND CANADIAN NATIONAL RAILWAY OVERPASS (SITE 3-301/1) GWP 4074-11-00 GEOCRES 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing eastbound Highway 417 Overpass of the Canadian National Railway (CN) east of Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives are not yet defined however, it is likely that the structure will need to be widened by 3.2 m to the east to accommodate the addition of another lane. This will necessitate the widening of the piers and abutments. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 266-00-01
- Site No. 3-301.1
- GEOCRES No. 31G5-79
- Construction Contract 73-190
- Historic W.P. 10-69-03

### **2 SITE DESCRIPTION**

The site is located in the Township of Gloucester, where the eastbound lanes of Highway 417 cross the CN railway tracks; approximately 850 m south of Walkley Road. For clarity and site orientation in this report the four pier foundations have been designated Pier 1 to Pier 4 with the northerly pier being designated Pier 1 and the numbers increasing going south.

Based on the historic General Layout Drawing (copy attached), the bridge is a 14 m wide, and 73 m long, five-span prestressed concrete overpass of the CN line. The railway line is between Piers 2 and 3 while a National Capital Commission access road is present between Piers 3 and 4. The two bridge abutments and the four piers are supported by steel HP12x74 piles driven to bedrock.



The natural terrain in the vicinity of the bridge is generally flat with elevations ranging from 64 m to 67 m. The design drawings show that the approach fills were to be constructed by placing as much as 10 m of fill at a 2H:1V (Horizontal:Vertical) slope with 3.0 m to 3.4 m wide mid-height berms.

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-79 dated February 1972. The investigation consisted of eight sampled boreholes, five of which were accompanied by dynamic cone penetration tests. Drawing No. 71-11124B (copy attached) illustrates the locations of the bridge, the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy along the centre of the bridge is generally characterized by a silty clay to clay layer, overlaying glacial till, underlain by shale bedrock.

#### **3.1 Silty Clay**

The top of the silty clay to clay layer ranged from 66.1 to 65.8 m in elevation, and the layer had a thickness of 5.2 to 5.8 m. Atterberg Limits test results indicate a liquid limit from 38% to 72%, and a plastic limit from 21% to 32%. The moisture content of the samples tested ranged from 32% to 62%. The estimated undrained shear strength based on in-situ field vane tests ranged from 100 kPa decreasing with depth to 35 kPa. Results of the shear strength testing indicate a very stiff consistency decreasing to firm with depth with a sensitivity between 6 and 20.

Consolidation testing was carried out on five samples and indicated that the deposit is slightly over-consolidated.

#### **3.2 Glacial Till**

Underlying the silty clay layer is a glacial till deposit. The surface of this deposit ranged from 60.3 to 60.9 m in elevation, and the layer had a thickness of 1.7 to 4.8 m. The standard penetration test 'N' values varied greatly for this deposit ranged from 2 to 163 blows per 0.3 m of penetration, indicating a very loose to very dense condition. The results of a grain size analysis tests including hydrometer testing completed on six samples of this material indicated a gravel content from 17% to 47%, sand content from 20% to 42%, silt content from 18% to 34%, and clay content from 8% to 17%.

#### **3.3 Bedrock**

Beneath the glacial till layer shale bedrock was encountered with surface elevations ranging from 55.4 to 58.8 m. The bedrock was described to be in sound condition. Geological mapping suggests the bedrock at this site is of the Carlsbad Formation.

#### **3.4 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 65.1 m and 65.9 m.



## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in October 2012 for Bridge 3-301/1 with the report issued September 2013. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in fair to good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering staff during the week of July 14<sup>th</sup>, 2014. Several photographs of the site are attached. At the time of the inspection, the following observations were made:

- No evidence of slope stability issues was noted on the well vegetated side slopes of the approach fills and the mid-height berm appeared to be intact.
- No erosion protection system was present beneath either abutment.
- Evidence of erosion was noted beneath both abutments. The mid-height berm was fully eroded beneath the north abutment which could potentially lead to slope instability.
- A pile and the pile cap were both visible beneath the north abutment due to the extensive erosion.
- The embankment slope at the lower edge of the road beneath the south abutment was very steep.
- No evidence of settlement issues was observed. The ride across the transition from deck to approaches was relatively smooth.

## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

The bridge foundations have been performing satisfactorily and frost protection should be reinstated for any exposed pile caps. The soil cover for the abutments should be reviewed and where insufficient earth cover is provided, polystyrene insulation may be used to enhance existing frost protection measures.

#### **5.2 Seismic Considerations**

This site is best classified as a Soil Profile Type III in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC). The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the plasticity of the clay at this site, it is classified as “not susceptible” to liquefaction during an earthquake event.



### **5.3 Existing Foundations**

As per the Foundation Layout and Reinforcement Drawing (copy attached) the bridge piers and abutments were designed to be supported on HP12x74 piles driven to bedrock. The available construction drawings do not indicate the design loads or grade of steel used for the piles; however the Foundation Design Report indicates that for steel 14HP74 end-bearing piles driven to bedrock, the recommended design allowable load for the pier foundations is 95 tons / 14HP74 pile or approximately 845 kN/pile. Due to the effects of negative skin friction the Foundation Design Report recommended that abutment piles be designed at a reduced allowable design load of 80 tons / 14HP74 pile or approximately 700 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN / HP310x110 pile driven to bedrock. The SLS condition will not govern for piles end-bearing in or on the bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined however it is noted that a 3.2 m widening of the existing structure may be required. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Shallow Foundations**

The very stiff to firm silty clay to clay deposit is not considered suitable to carry a bridge structure. The depth of the till deposit was observed to range from 4.0 m to 5.8 m below original grade which is considered relatively deep for spread footing foundations. As such spread footings within the overburden are not recommended and deep foundations are preferred at this site.

### **6.2 Deep Foundations – Piles**

Based on the review of existing subsurface stratigraphy at the site driven piles are considered to be suitable for the support of abutment and pier foundations. It should be noted that the bedrock surface elevation ranges from 55.4 m to 58.8 m.

#### **6.2.1 Axial Resistance**

Steel piles bearing on sound bedrock at this site may be designed on the basis of the following factored, vertical, geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.



### **6.2.2 Pile Tips**

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### **6.2.3 Integral Abutment Considerations**

As per the Foundation Layout and Reinforcement Drawing the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### **6.2.4 Pile Spacing**

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### **6.2.5 Downdrag**

Downdrag forces need to be considered for piles in areas where grades have increased from original. This includes the piles supporting the existing abutments and Piers 1 and 4. It is anticipated that the design of new piles required to support a widened foundation at both the abutments and Piers 1 and 4 will also need to include downdrag loads due to approach fill widening. Downdrag loads for piles at Piers 2 and 3 are not anticipated unless grades are modified at these pier locations.

The following pile SLS dead loads were provided by the structural design team in order to determine the downdrag forces acting on Structure 3-301/1:

- Abutment pile loading = 140 kN / pile
- Pier pile loading = 502 kN / pile

The value of downdrag on existing and new piles should be considered for this site as follows:

Existing piles:

- The abutment piles are being subject to unfactored downdrag loads of approximately 300 kN/pile due to the original placement of as much as 10.0 m of fill at the approaches. Placement of additional fill for embankment widening or grade raises will not add further downdrag load to these piles.
- The piles supporting Piers 1 and 4 are being subject to unfactored downdrag loads of approximately 50 kN/pile due to the original placement of as much as 10.0 m of fill at the approaches.



- Placement of additional fill for embankment widening or grade raises will not add further downdrag load to these piles.

#### New Piles:

If there are no modifications to the width or height of the approach fills or in the areas of the piers, new piles at the site will not be subject to downdrag loads. Should modifications be required, assuming the same SLS dead loads listed above and that any additional approach fills will match existing grades; new piles at Bridge Structure 3-301/1 would be subject to the following downdrag loads:

- New abutment piles will be subject to unfactored downdrag loads of approximately 300 kN/pile due to the placement of fill to address widening and/or grade raises at the approaches.
- New piles at Piers 1 and 4 will be subject to unfactored downdrag loads of approximately 50 kN / pile due to the placement of fill to address widening and/or grade raises at the approaches.

The downdrag load should be multiplied by a load factor of 1.25 as per the CHBDC Commentary Clause C6.8.4 to obtain a factored downdrag load. In accordance with Section 6.8.4 of the CHBDC and clause C6.8.4 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag loads should not exceed the factored structural resistance of a pile. In geotechnical analysis of downdrag, live load effects should not be considered.

### 6.2.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formulas for cohesionless and cohesive soils:

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

For cohesive soils:

$$k_h = \frac{67S_u}{B}$$

where:

- $k_h$  = coefficient of horizontal subgrade reaction
- $n_h$  = coefficient related to soil density given in Table A
- $z$  = depth of pile embedment (m)
- $B$  = pile width perpendicular to load direction (m)
- $S_u$  = undrained shear strength given in Table A

**Table A:**  $n_h$  values for cohesionless soils and  $S_u$  values for cohesive soils

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )	$S_u$ (kPa)
Above 66	Embankment Fill	3000	-
Between 66 and 61	Native Clay	-	100 to 35
Below 61	Glacial Till	2000	-

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

## 6.2.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

$C$  = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)

**Table C:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 66	Embankment Fill	20	0.4
Between 66 and 61	Native Clay	17	0.25
Below 61	Glacial Till	19	0.4

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D: Static Lateral Earth Pressure Coefficients**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Clay
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	17
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$	$27^\circ$
<b>Horizontal Back-Slope</b>			
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.50	0.55
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33	0.38

#### 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E: Lateral Earth Pressure (Under Seismic Loads)**

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Clay
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	17
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$	$27^\circ$
<b>Yielding Wall</b>			
$K_{AE}$	0.33	0.4	0.45
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36
<b>Non-Yielding Wall</b>			
$K_{AE}$	0.55	0.66	0.74
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43



The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors provided in Table E are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## 6.5 Approach Embankments

Based on General Plan Drawing, the approach embankments should consist of compacted, boulder free fill to a maximum height of 10 m above the original ground surface. An embankment slope of 2H:1V was used and it was recommended that berms be incorporated in the slopes where earth fill embankments are higher than 8 m. Settlement of up to 100 mm was predicted with 50% of the consolidation occurring within 12 months of fill placement.

The embankment fills are expected to be stable and no further consolidation settlement is expected unless the fills are reconfigured.

Embankment widening of as much as 3.2 m is anticipated at this site. For preliminary purposes it is recommended that slopes be designed at 2H:1V with a mid-height bench of at least 3.0 m in width. Settlement is anticipated due to the consolidation of the underlying clay soils. For preliminary purposes it is estimated that settlement would be in the range of 100 mm to 150 mm. It is likely that most of this settlement will occur quickly, within several months, nonetheless the consequences of this movement to the existing foundations and approach fills will need to be evaluated during detailed design.

## 6.6 Erosion Control

Active erosion beneath both abutments was noted as no erosion protection systems were present at the site.



The eroded slopes in front of the abutments should be reinstated, erosion protection measures incorporated and the drainage measures enhanced beneath the abutments to prevent further erosion of the embankment material.

## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native very stiff to firm silty clay soils reported at this site should be classified as Type 3 in accordance with the OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation for pile caps must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that piezometers be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

Should the preferred alternative include widening of the existing structure, shoring may be required to allow excavation to the underside of the pile cap at Piers 2 and 3 due to their proximity to the railway line. Shoring may also be needed at Piers 1 and 4 to support the approach fills during pile cap widening. Consideration should also be given to drilling foundation boreholes at these locations should the widening alternative be selected for this site.

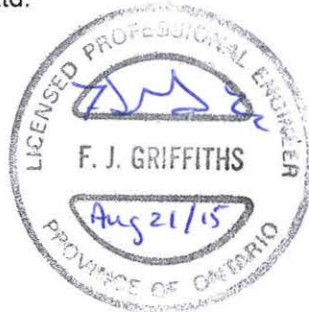
During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or the construction of a new structure adjacent to the existing one. Similarly mitigation programs may be needed to be defined in the event of widening or substructure strengthening (seismic), given the close proximity to the rail tracks. In addition, coordination with the railway, railway design code requirements and track protection will need to be considered.



## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.

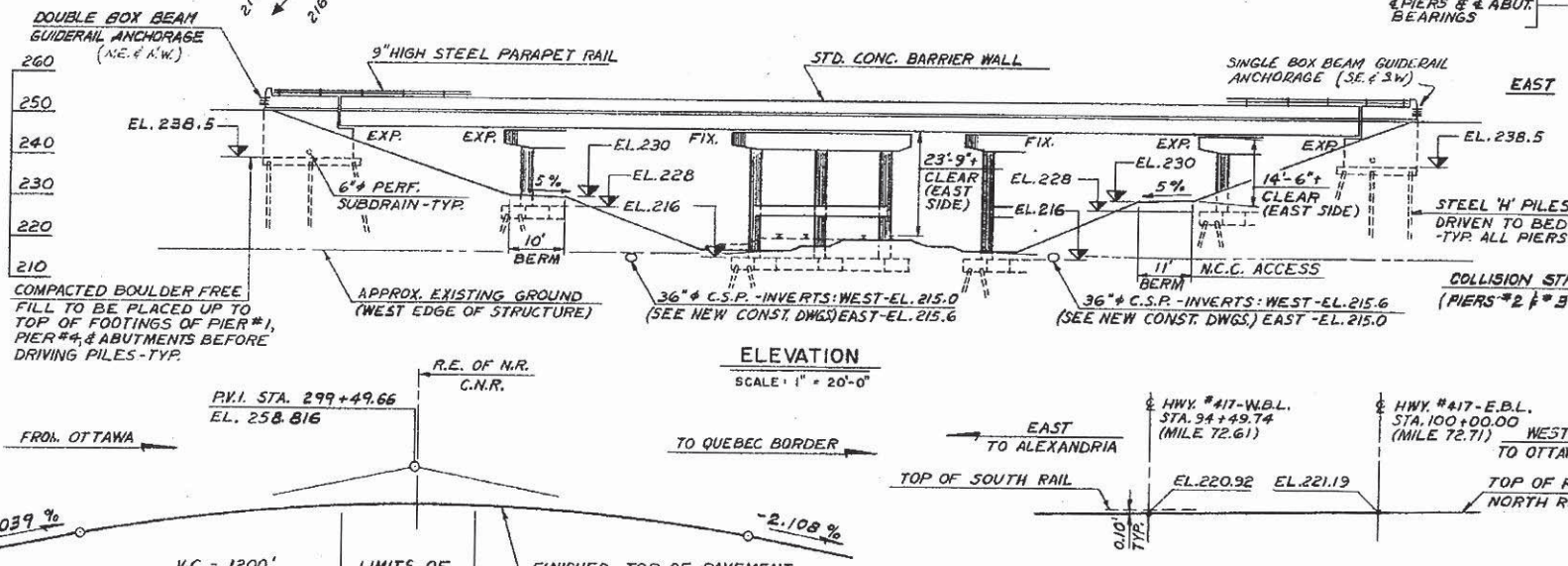
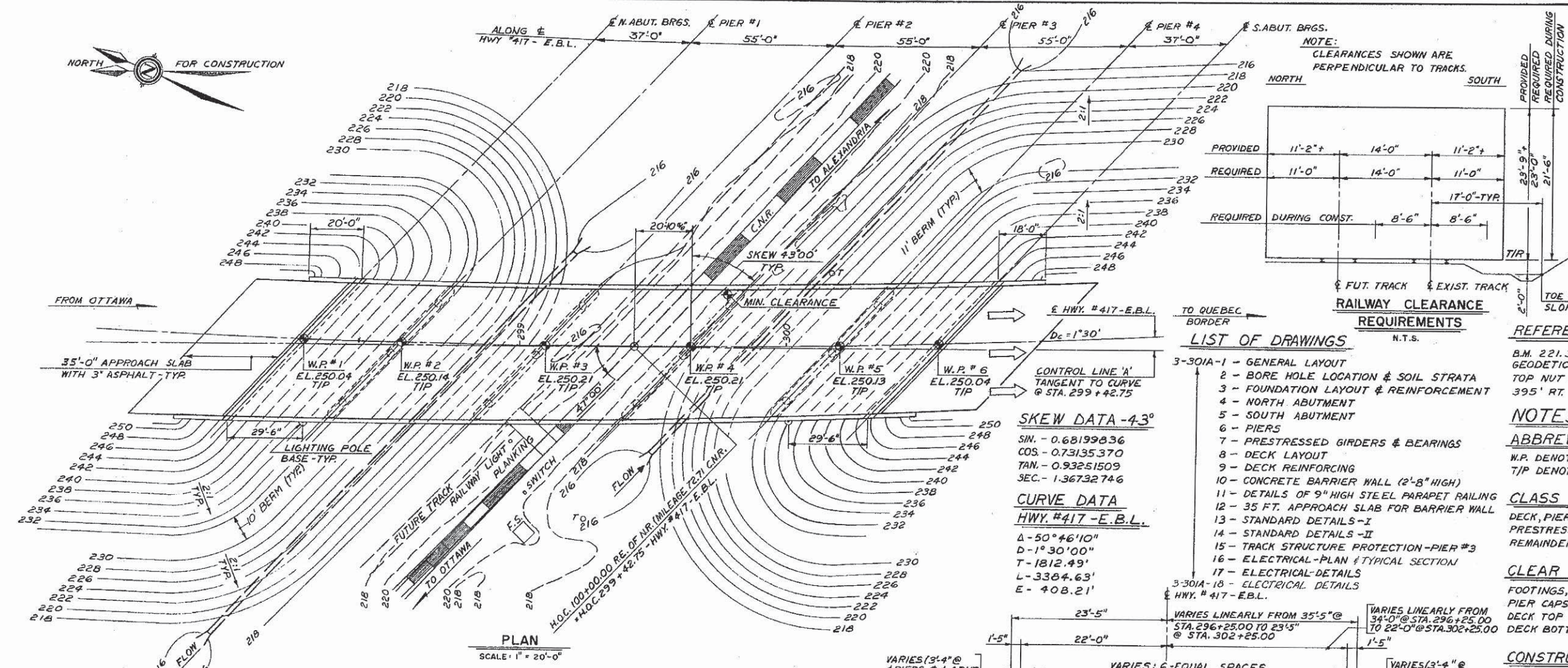


Fred J. Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer

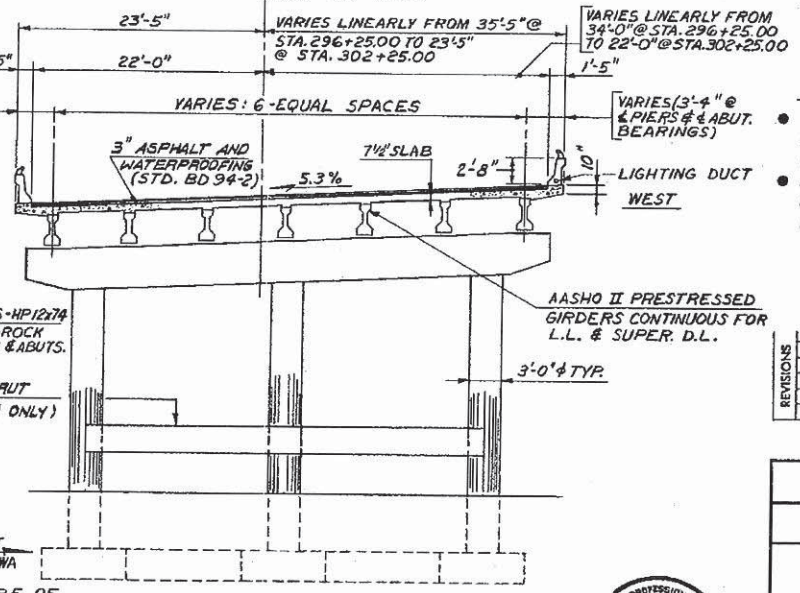


P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

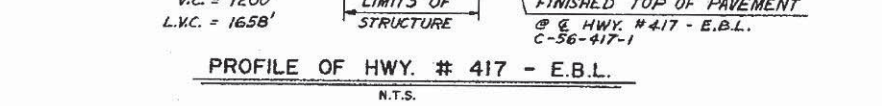
Attachments



PROFILE OF C.N.R. ALEXANDRIA SUBDIVISION  
N.T.S.



TYPICAL SECTION  
SCALE: 1/8" = 1'-0"



NOTE:  
CLEARANCES SHOWN ARE  
PERPENDICULAR TO TRACKS.

	PROVIDED	REQUIRED	REQUIRED DURING CONST.
11'-2"	11'-2"	11'-0"	8'-6"
14'-0"	14'-0"	14'-0"	8'-6"
11'-2"	11'-2"	11'-0"	8'-6"
17'-0"-TYP.			

RAILWAY CLEARANCE  
REQUIREMENTS  
N.T.S.

LIST OF DRAWINGS

- 3-301A-1 - GENERAL LAYOUT
- 2 - BORE HOLE LOCATION & SOIL STRATA
- 3 - FOUNDATION LAYOUT & REINFORCEMENT
- 4 - NORTH ABUTMENT
- 5 - SOUTH ABUTMENT
- 6 - PIERS
- 7 - PRESTRESSED GIRDERS & BEARINGS
- 8 - DECK LAYOUT
- 9 - DECK REINFORCING
- 10 - CONCRETE BARRIER WALL (2'-8" HIGH)
- 11 - DETAILS OF 9" HIGH STEEL PARAPET RAILING
- 12 - 35 FT. APPROACH SLAB FOR BARRIER WALL
- 13 - STANDARD DETAILS-I
- 14 - STANDARD DETAILS-II
- 15 - TRACK STRUCTURE PROTECTION-PIER #3
- 16 - ELECTRICAL-PLAN & TYPICAL SECTION
- 17 - ELECTRICAL-DETAILS
- 3-301A-18 - ELECTRICAL DETAILS

SKREW DATA-43°

SIN. - 0.68199836  
COS. - 0.73135370  
TAN. - 0.93251509  
SEC. - 1.36732746

CURVE DATA

HWY. #417 - E.B.L.  
Δ - 50° 46' 10"  
D - 1° 30' 00"  
T - 1812.49'  
L - 3384.63'  
E - 408.21'

REFERENCE BENCH MARK

B.M. 221.39  
GEODETIC DATUM  
TOP NUT ON S.E. CORNER OF HYDRO TOWER  
395' RT. OF 299+77 - E.B.L.

NOTES

ABBREVIATIONS

W.P. DENOTES WORKING POINT  
T/P DENOTES TOP OF PAVEMENT

CLASS OF CONCRETE

DECK, PIERS, BARRIER WALLS & APPROACH SLABS - 4,000 P.S.I.  
PRESTRESSED GIRDERS - 5,000 P.S.I.  
REMAINDER - 3,000 P.S.I.

CLEAR COVER ON REINF. STEEL

FOOTINGS, ABUTMENTS & PIER COLUMNS - 3"  
PIER CAPS & COLLISION STRUT - 2"  
DECK TOP & BARRIER WALLS - 1 1/2"  
DECK BOTTOM - 1"

CONSTRUCTION NOTES

- THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8"
- NO CONCRETE IS TO BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO

Consulting Engineers & Planners

**C.N.R. OVERHEAD**  
(EASTBOUND LANES)  
2.2 MILES EAST OF INNES ROAD  
KING'S HIGHWAY No. 417  
CO. REG. MUN. OTTAWA-CARLETON  
TWP. GLOUCESTER LOT 2 CON. VI

DIS. No. 9

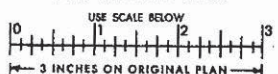
GENERAL LAYOUT

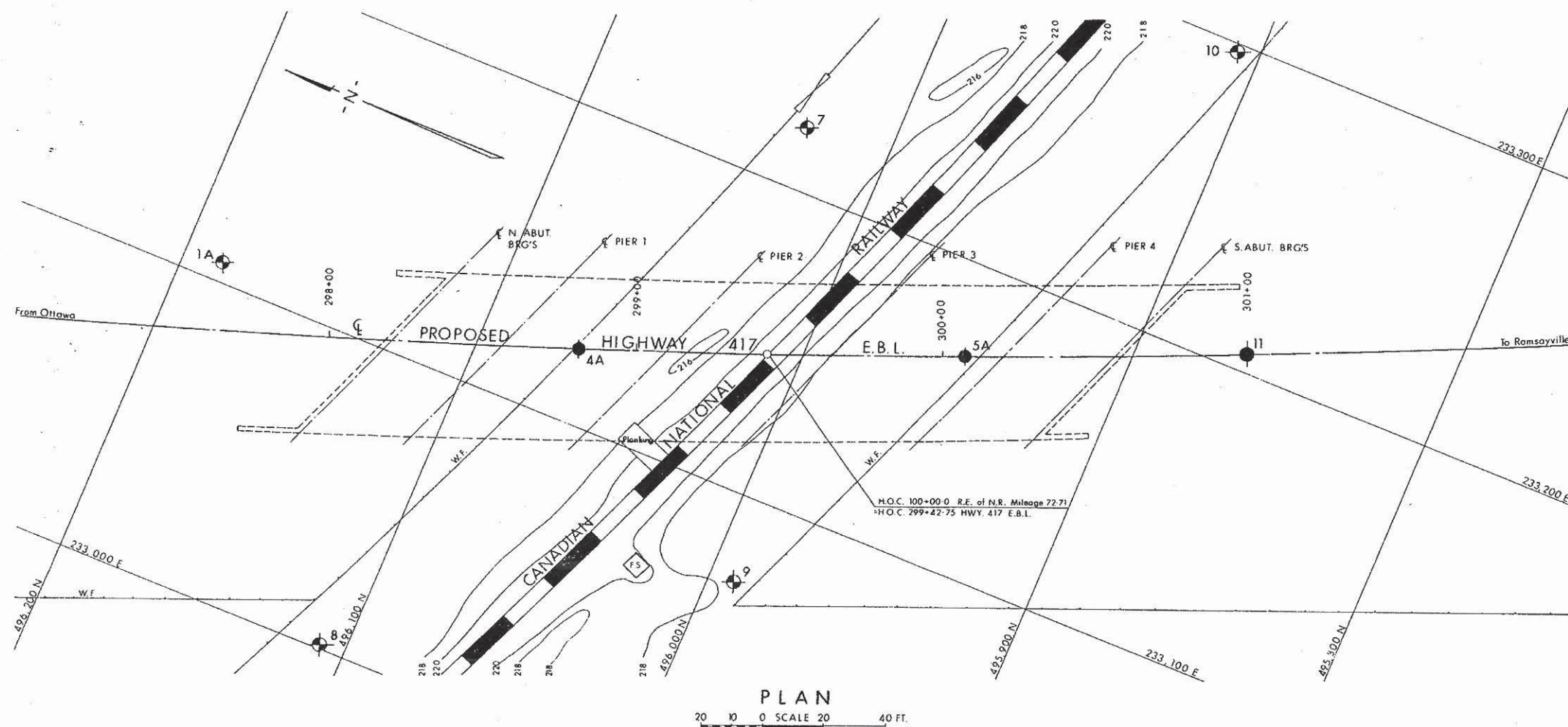
APPROVED: [Signature]  
DESIGN: [Signature] CHECK: [Signature]  
DRAWING: [Signature] CHECK: [Signature]  
DATE: DEC. 72 LOADING: HS 20-44

SITE No. 3-301A W.P. No. 10-69-03  
CONTRACT No. 73-170  
DRAWING No. 3-301A-1



FOR REDUCED PLAN  
USE SCALE BELOW





SEE DRAWING NO. 71-11124A

KEY PLAN  
SCALE IN MILES

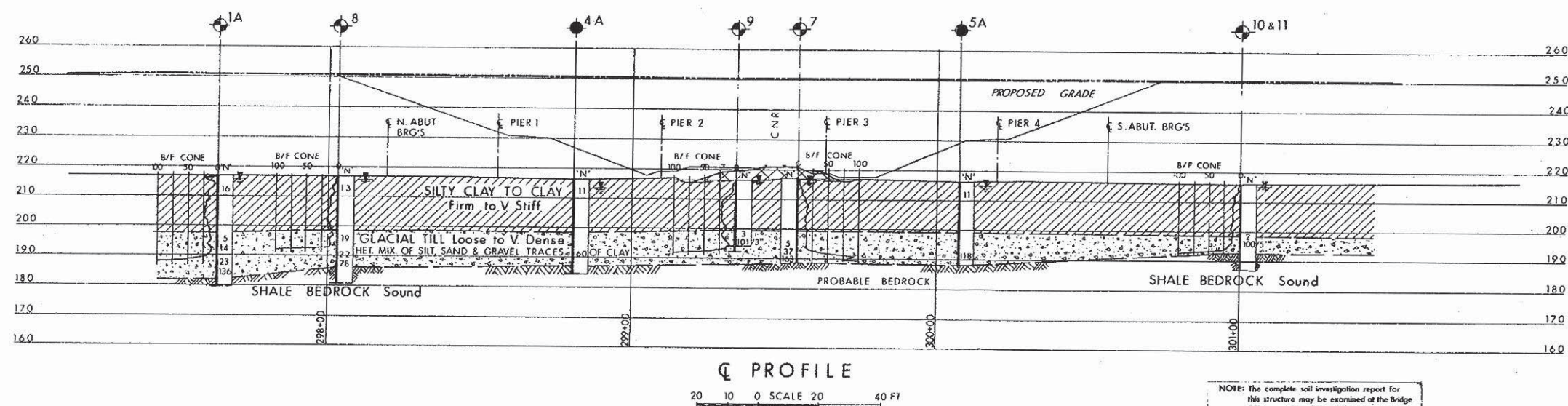
## LEGEND

- Bore Hole
- ⊕ Cone Penetration Test
- ⊗ Bore Hole & Cone Test
- ≡ Water Levels established at time of field investigation NOV & DEC. 71 APRIL 72 & JUNE 1972

NO.	ELEVATION	CO-ORDINATES	
		NORTH	EAST
7	216.6	496,028	233,220
8	216.7	496,108	233,001
9	216.7	495,992	233,072
10	216.0	495,907	233,297
11	216.1	495,866	233,206
1A	216.7	496,188	233,106
4A	216.6	496,069	233,124
5A	216.3	495,951	233,170

## NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.



NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the Ottawa District Office.

REF. NO. E-5225-1

REVISIONS	DATE	BY	DESCRIPTION
1	July 1972	G.P.	ADDED BORE HOLES NO. 4A & 5A ON PLAN & PROFILE
2	June 1972	S.R.	ADDED BORE HOLE NO. 1A

MINISTRY OF TRANSPORTATION & COMMUNICATIONS  
DESIGN SERVICES BRANCH - FOUNDATIONS OFFICE

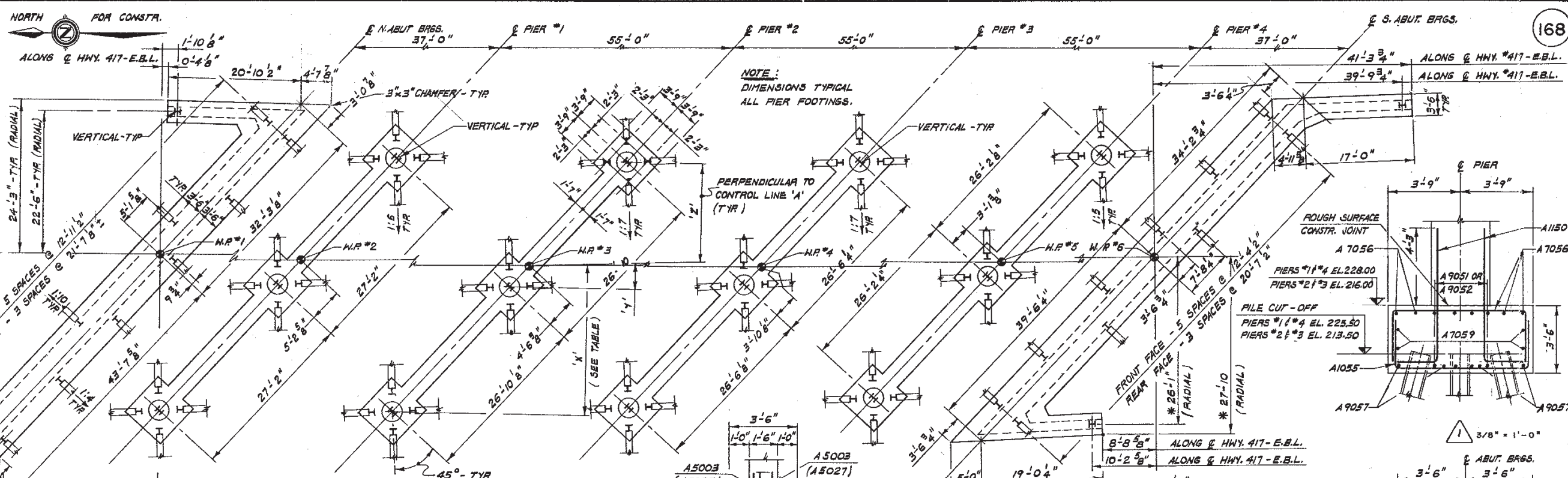
CANADIAN NATIONAL RAILWAY  
(APPROX. 1.3 MILES N. OF RAMSAYVILLE)

HIGHWAY NO. 417 E.B.L. DIST. NO. 9  
CO. REG. MUN. OTTAWA-CARLETON  
TWP. GLOUCESTER LOT 2 CON. 6

BORE HOLE LOCATIONS & SOIL STRATA

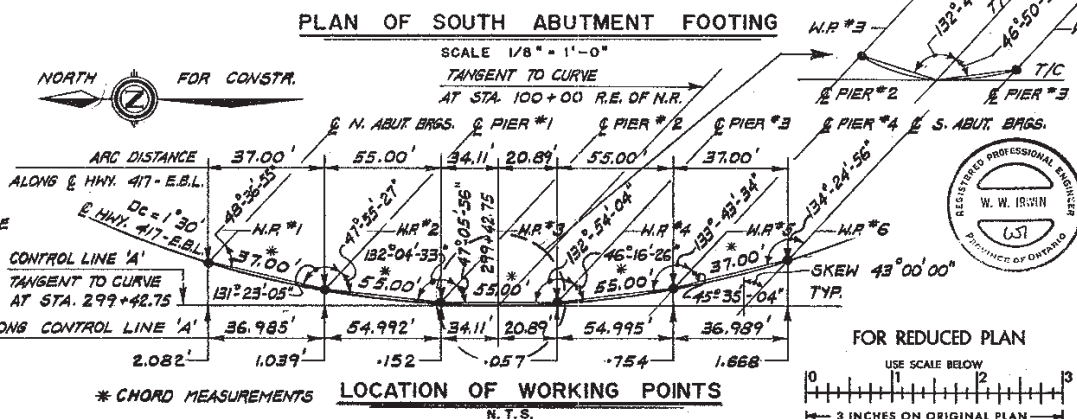
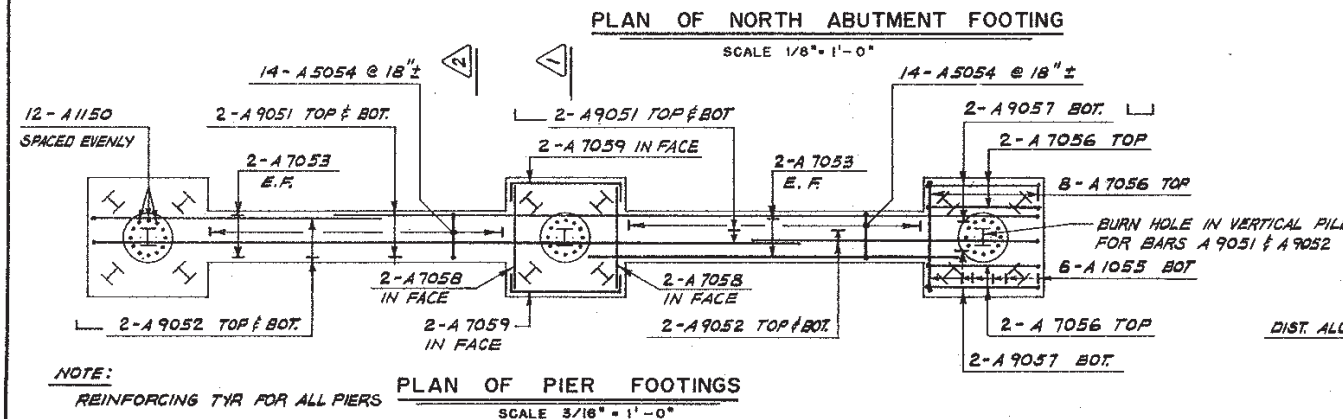
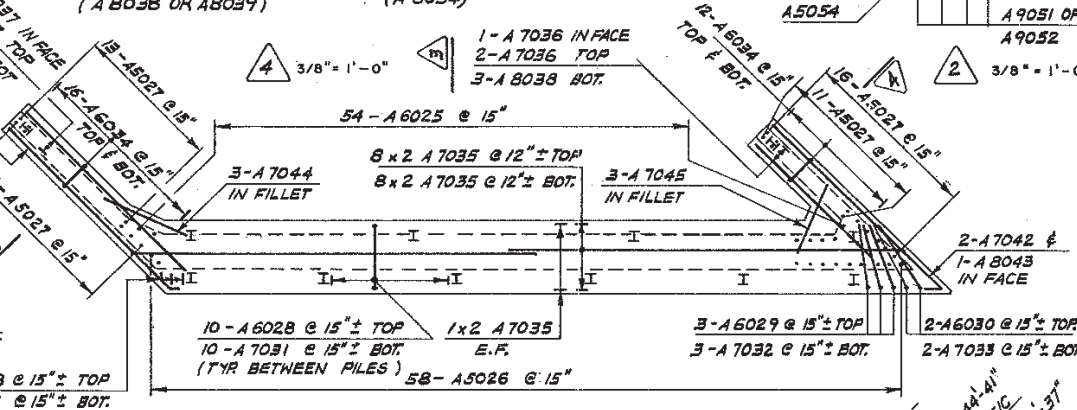
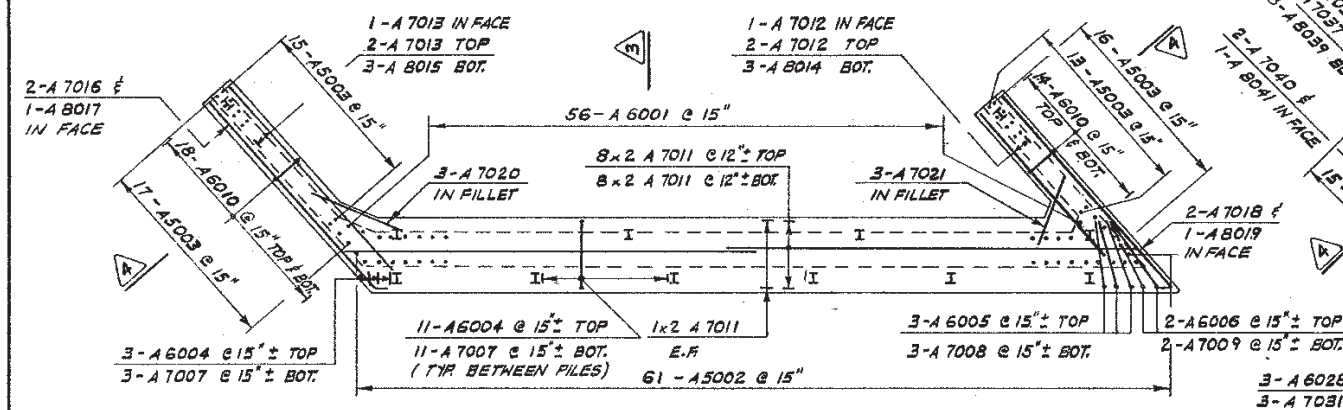
SUBMD. S.A.	CHECKED: [Signature]	W.P. NO. 10-69-03	DRAWING NO. 71-11124 B
DRAWN S.O.	CHECKED: [Signature]	JOB NO. 71-11124	BRIDGE DRAWING NO. 3-301A-2
DATE 28 JAN. 1972		SITE NO. 3-301A	
APPROVED: [Signature]		CONT. NO. 75-1-10	

STEEL 'H' PILE DATA			
LOCATION	BATTER	NO.	LENGTH
N. ABUT.	VERT.	2	54'
	1:10	6	56'
PIER #1	VERT.	3	42'
	1:10	4	56'
PIER #2	VERT.	3	42'
	1:10	4	56'
PIER #3	VERT.	3	42'
	1:10	4	56'
PIER #4	VERT.	3	42'
	1:10	4	56'
S. ABUT.	VERT.	2	54'
	1:10	6	56'



LOCATION	'X'	'Y'	'Z'
PIER #1	23'-8"	3'-9"	16'-0"
PIER #2	22'-11"	3'-3"	16'-3"
PIER #3	22'-2"	2'-9"	16'-7"
PIER #4	21'-5"	2'-3"	16'-10"

**FOOTING LAYOUT**  
SCALE 1/8" = 1'-0"



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
Consulting Engineers & Planners	
<b>C.N.R. OVERHEAD</b> (EASTBOUND LANES) 2.2 MILES EAST OF INNES RD. KING'S HIGHWAY No. 417 DIST. No. 9 CO. REG. MUN. OTTAWA - CARLETON TWP. GLOUCESTER LOT 2 CON. V1	
<b>FOUNDATION LAYOUT &amp; REINFORCEMENT</b>	
APPROVED	SITE No. 3-301A W.P. No. 10-69-03
DESIGN	CONTRACT No.
DRAWING	CHECK
DATE	LOADING
3-301A-3	



**APPENDIX 13**  
**SITE 3-301/2**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Fred J. Griffiths, Ph.D., P.Eng.  
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 WESTBOUND CANADIAN NATIONAL RAILWAY OVERPASS (SITE 3-301/2) GWP 4074-11-00 GEOCRES 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing westbound Highway 417 Overpass of Canadian National Railway (CN) east of Ottawa, Ontario. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives are not yet defined, however, MTO's Transportation Environmental Assessment Report from 2007 indicates that the structure be will need to be widened by 4.6 m to the west to accommodate an additional lane. This will necessitate the widening of the piers and abutments. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 266-00-02
- Site No. 3-301.2
- GEOCRES No. 31G5-79
- Construction Contract 73-190
- Historic W.P. 10-69-04

### **2 SITE DESCRIPTION**

The site is located in the Township of Gloucester, where the westbound lanes of Highway 417 cross the CN railway tracks; approximately 850 m south of Walkley Road. For clarity and site orientation in this report the four pier foundations have been designated Pier 1 to Pier 4 with the northerly pier being designated Pier 1 and the numbers increasing going south.

Based on the historic General Layout Drawing (copy attached) the bridge is a 13 m wide, and 71 m long, five-span prestressed concrete overpass of the CN line. The railway is located between



Piers 2 and 3. A National Capital Commission access road is present between Piers 3 and 4. The two bridge abutments and four piers are supported by steel HP12x74 piles driven to bedrock.

The natural terrain in the vicinity of the bridge is generally flat with elevations ranging from 64 m to 67 m. The design drawings show that the approach fills were to be constructed by placing fill to elevation of 75.9 m at a 2H:1V (Horizontal:Vertical) slope with 3.0 m to 3.4 m wide mid-height benches.

A 1.8 m diameter corrugated steel culvert was to be included between Piers 3 and 4 at shallow depth to facilitate drainage beneath the structure.

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-79; dated February 1972. The investigation consisted of one cone penetration hole and seven sampled boreholes, six of which were accompanied by dynamic cone penetration tests. Drawing No. 71-11124A (copy attached) illustrates the locations of the bridge, the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy along the centre of the bridge is generally characterized by a silty clay to clay layer, overlaying a glacial till, underlain by shale bedrock.

#### **3.1 Silty Clay**

The top of the silty clay to clay layer ranged from 66.1 to 65.5 m in elevation, and the layer had a thickness of 4.0 to 5.2 m. Atterberg Limits test results indicate a liquid limit from 62% to 72%, and a plastic limit from 25% to 26%. The moisture content of the samples tested ranged from 28% to 58%. The estimated undrained shear strength based on the in-situ field vane tests ranged from 100 kPa decreasing with depth to 35. Results of the shear strength testing indicate a very stiff consistency decreasing to firm with depth with a sensitivity between 6 and 20.

Consolidation testing was carried out on three samples of this material and indicated that the deposit is over-consolidated with 170 kPa between existing effective stress and the preconsolidation pressure.

#### **3.2 Glacial Till**

Underlying the silty clay layer is a glacial till deposit. The surface of this deposit ranged from 60.5 to 61.8 m in elevation, and the layer had a thickness of 1.5 to 3.5 m. The standard penetration 'N' values for this deposit ranged from 1 to 18 blows per 0.3 m of penetration indicating, a very loose to compact condition. The results of a grain size analysis tests including hydrometer testing completed on seven samples of this material indicated a gravel content from 17% to 35%, sand content from 20% to 42%, silt content from 18% to 34%, and clay content from 8% to 17%.



### **3.3 Bedrock**

Beneath the glacial till layer a grey shale bedrock was encountered with surface elevations ranging from 57.6 to 59.5 m. The bedrock was described to be in sound condition. Geological mapping suggests the bedrock at this site is of the Carlsbad Formation.

### **3.4 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 64.6 m and 66.1 m.

## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in July 2012 for Bridge 3-301/2 with the report issued September 2013. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in fair to good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering staff during the week of July 14<sup>th</sup>, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

- No evidence of slope stability issues were noted on the well vegetated side slopes of the approach fills and the mid-height bench appeared to be intact
- No erosion protection system was present beneath either abutment
- Evidence of erosion was noted beneath both abutments. The mid-height bench was fully eroded beneath the north abutment and the top of the pile cap was visible beneath both abutments due to the erosion
- The embankment slope at the lower edge of the road beneath the south abutment was noted to be very steep
- No evidence of settlement issues of the roadway were observed. The ride across the transition from deck to approaches was relatively smooth

## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

The bridge foundations have been performing satisfactorily and frost protection should be reinstated for any exposed pile caps. The soil cover for the abutments should be reviewed and where insufficient earth cover is provided, polystyrene insulation may be used to enhance existing frost protection measures.



## **5.2 Seismic Considerations**

This site is best classified as a Soil Profile Type III in accordance with Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC). The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the plasticity of the clay at this site, it is classified as “not susceptible” to liquefaction during an earthquake event.

## **5.3 Existing Foundations**

As per Foundation Layout and Reinforcement Drawing the bridge abutments and pier foundations were designed to be supported on steel HP12x74 piles driven to bedrock. The available construction drawings do not indicate the design loads or grade of steel used for the piles. However, the Foundation Design Report indicates that for 14HP74 end-bearing piles driven to bedrock the recommended design allowable load for the pier foundations is 95 tons / 14HP74 pile or approximately 845 kN/pile. Due to the effects of negative skin friction the Foundation Design Report recommended that abutment piles be designed at a reduced allowable design load of 80 tons / 14HP74 pile or approximately 700 kN/pile.

HP12x74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN / HP310x110 pile driven to bedrock. The SLS condition will not govern for piles end-bearing in or on the bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

# **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined however it is noted that a 4.6 m widening of the existing structure may be required. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

## **6.1 Shallow Foundations**

The very stiff to firm silty clay to clay deposit is not considered suitable to carry a bridge structure. The depth of the till deposit was observed to range from 4.0 m to 5.8 m below original grade which is considered relatively deep for spread footing foundations. As such spread footings within the overburden are not recommended and deep foundations are preferred at this site.



## **6.2 Deep Foundations**

Based on the review of existing subsurface stratigraphy at the site driven piles are considered to be suitable for the support of abutment and pier foundations. It should be noted that the bedrock surface elevation ranges from 57.6 m to 59.5 m.

### **6.2.1 Axial Resistance**

Steel piles (Grade 350 W steel) end-bearing on sound shale bedrock at this site may be designed on the basis of the following factored, vertical, geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

### **6.2.2 Pile Tips**

Where new piles are driven to bedrock, the pile tips should be protected from damage during driving.

### **6.2.3 Integral Abutment Considerations**

As per the Foundation Layout and Reinforcement Drawing (copy attached) the existing abutments are supported by pile groups that includes battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### **6.2.4 Pile Spacing**

Since new piles at this site will be end bearing on bedrock, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### **6.2.5 Downdrag**

Downdrag forces need to be considered for piles in areas where grades have increased from original. This includes the piles supporting the existing abutments and Piers 1 and 4. It is anticipated that the design of new piles required to support a widened foundation these locations will also need to include downdrag loads due to approach fill widening.

Unless grades are modified in the vicinity of Piers 2 and 3, downdrag loads for the piles at these pier locations are not anticipated.



The following pile SLS dead loads were provided by the structural design team in order to determine the downdrag forces acting on Structure 3-301/2:

- Abutment pile loading = 150 kN / pile
- Pier pile loading = 477 kN / pile

The value of downdrag on existing and new piles should be considered for this site as follows:

Existing piles:

- The abutment piles are being subject to unfactored downdrag loads of approximately 125 kN/pile due to the original placement of as much as 9.0 m of fill at the approaches. Placement of additional fill for embankment widening or grade raises will not add further downdrag load to these piles.
- The piles supporting Piers 1 and 4 are being subject to unfactored downdrag loads of approximately 125 kN/pile due to the original placement of as much as 9.0 m of fill at the approaches.
- Placement of additional fill for embankment widening or grade raises will not add further downdrag load to these piles.

New Piles:

If there are no modifications to the width or height of the approach fills or in the areas of the piers, new piles at the site will not be subject to downdrag loads. Should modifications be required, assuming the same SLS dead loads listed above and that any additional approach fills will match existing grades; new piles at Bridge Structure 3-301/2 would be subject to the following downdrag loads:

- New abutment piles will be subject to downdrag loads of approximately unfactored 125 kN/pile due to the placement of fill to address widening and/or grade raises at the approaches.
- New piles at Piers 1 and 4 will be subject to downdrag loads of approximately unfactored 125 kN/pile due to the placement of fill to address widening and/or grade raises at the approaches.

The downdrag load should be multiplied by a load factor of 1.25 as per the CHBDC Commentary Clause C6.8.4 to obtain a factored downdrag load. In accordance with Section 6.8.4 of the CHBDC and clause C6.8.4 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag loads should not exceed the factored structural resistance of a pile. In geotechnical analysis of downdrag, live load effects should not be considered.

### 6.2.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formulae:

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

For cohesive soils:

$$k_h = \frac{67S_u}{B}$$

where:

- $k_h$  = coefficient of horizontal subgrade reaction
- $n_h$  = coefficient related to soil density given in Table A
- $z$  = depth of pile embedment (m)
- $B$  = pile width perpendicular to load direction (m)
- $S_u$  = undrained shear strength given in Table A

**Table A:**  $n_h$  values for cohesionless soils and  $S_u$  values for cohesive soils.

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )	$S_u$ (kPa)
Above 66	Embankment Fill	3000	-
Between 66 and 61	Native Clay	-	100 to 35
Below 61	Glacial Till	2000	-

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table B below. Intermediate values may be obtained by linear interpolation.

**Table B:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

## 6.2.7 Uplift Resistance

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table C

$\sigma'_v$  = effective vertical stress (kPa)

**Table C:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 66	Embankment Fill	20	0.4
Between 66 and 61	Native Clay	17	0.25
Below 61	Glacial Till	19	0.4

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.



### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table D.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table D are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table D:** Static Lateral Earth Pressure Coefficients

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Clay
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	21	20	17
Angle of Internal Friction, $\phi$	35°	30°	27°
<b>Horizontal Back-Slope</b>			
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.50	0.55
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33	0.38

## 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

**Table E:** Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Clay
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21	20	17
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$	$27^\circ$
<b>Yielding Wall</b>			
$K_{AE}$	0.33	0.4	0.45
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36
<b>Non-Yielding Wall</b>			
$K_{AE}$	0.55	0.66	0.74
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil ( $\text{kN/m}^3$ )

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth



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

The factors provided in Table E are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## **6.5 Approach Embankments**

The Foundation Design Reports states that the embankment soils should consist of compact to dense deposits of non-cohesive soil to a maximum height of 10 m above the original ground surface. An embankment slope of 2H:1V was used and it was recommended that berms be incorporated in the slopes where earth fill embankments are higher than 8 m. Settlement of up to 100 mm was predicted with 50% of the consolidation occurring within 12 months of fill placement.

The embankment fills are expected to be stable and no further consolidation settlement is expected unless the fills are reconfigured.

Embankment widening of as much as 4.6 m is anticipated at this site. For preliminary purposes it is recommended that slopes be designed to match existing, i.e. sloped at 2H:1V with a mid-height bench of at least 3.0 m in width. Settlement is anticipated due to the consolidation of the underlying clay soils. For preliminary purposes it is estimated that settlement would be in the range of 120 mm. It is likely that most of this settlement will occur quickly, within several months, nonetheless the consequences of this movement on the existing foundation and paved surfaces will need to be evaluation during detailed design.

## **6.6 Erosion Control**

Active erosion beneath both abutments was noted as no erosion protection systems were present at the site.

The eroded slopes in front of the abutments should be reinstated, erosion protection measures incorporated and the drainage measures enhanced beneath the abutments to prevent further erosion of the embankment material.

## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The native very stiff to firm silty clay soils reported at this site should be classified as Type 3 in accordance with the OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.



Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation for pile caps must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that piezometers be installed to better define the groundwater level as well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

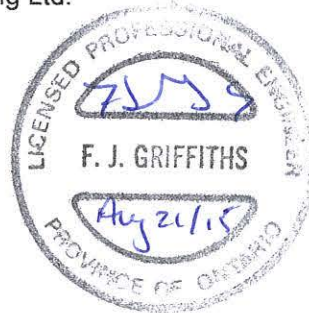
Should the preferred alternative include widening of the existing structure, shoring may be required to allow excavation to the underside of the pile cap at Piers 2 and 3 due to their proximity to the railway line. Shoring may also be needed at Piers 1 and 4 to support the approach fills during pile cap widening. Consideration should also be given to drilling foundation boreholes at these locations should the widening alternative be selected for this site.

During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or a new structure adjacent to existing. Similarly mitigation programs may be needed to be defined in the event of widening or substructure strengthening (seismic), given the close proximity to the rail tracks. Also, coordination with the railway, railway design code requirements and track protection will need to be considered.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.

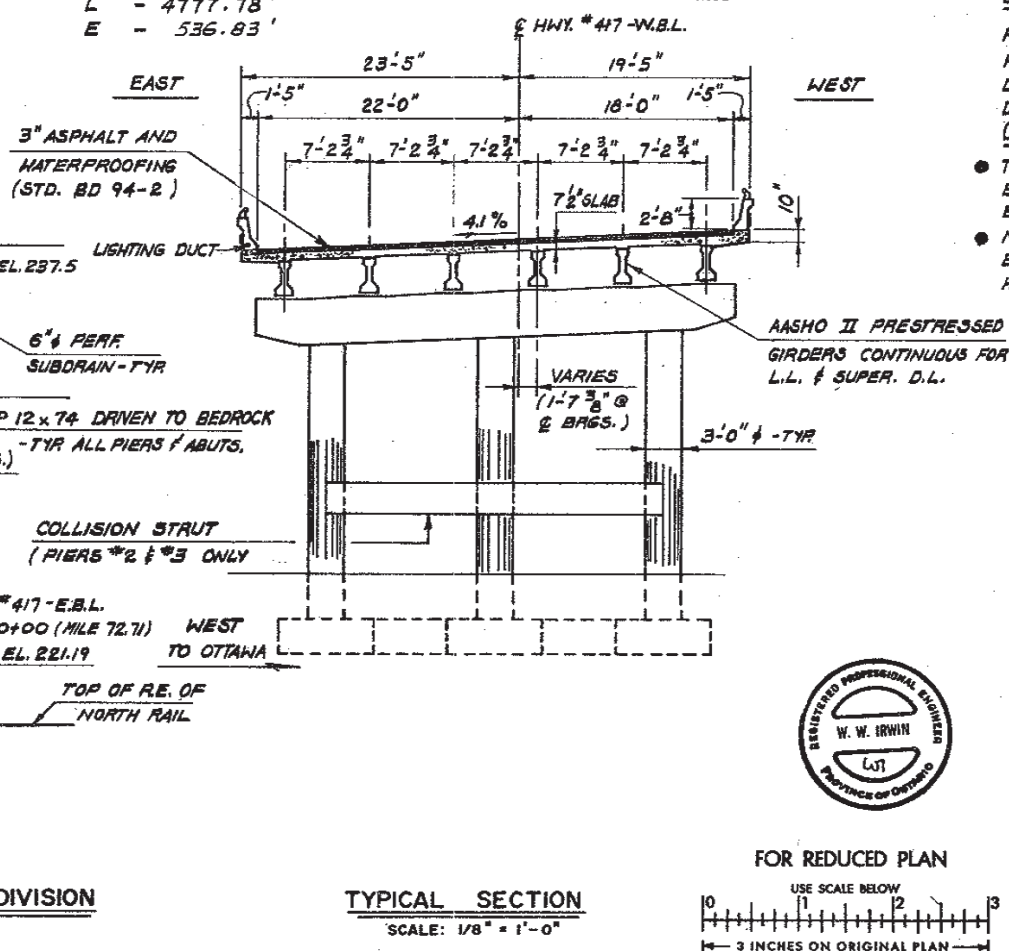
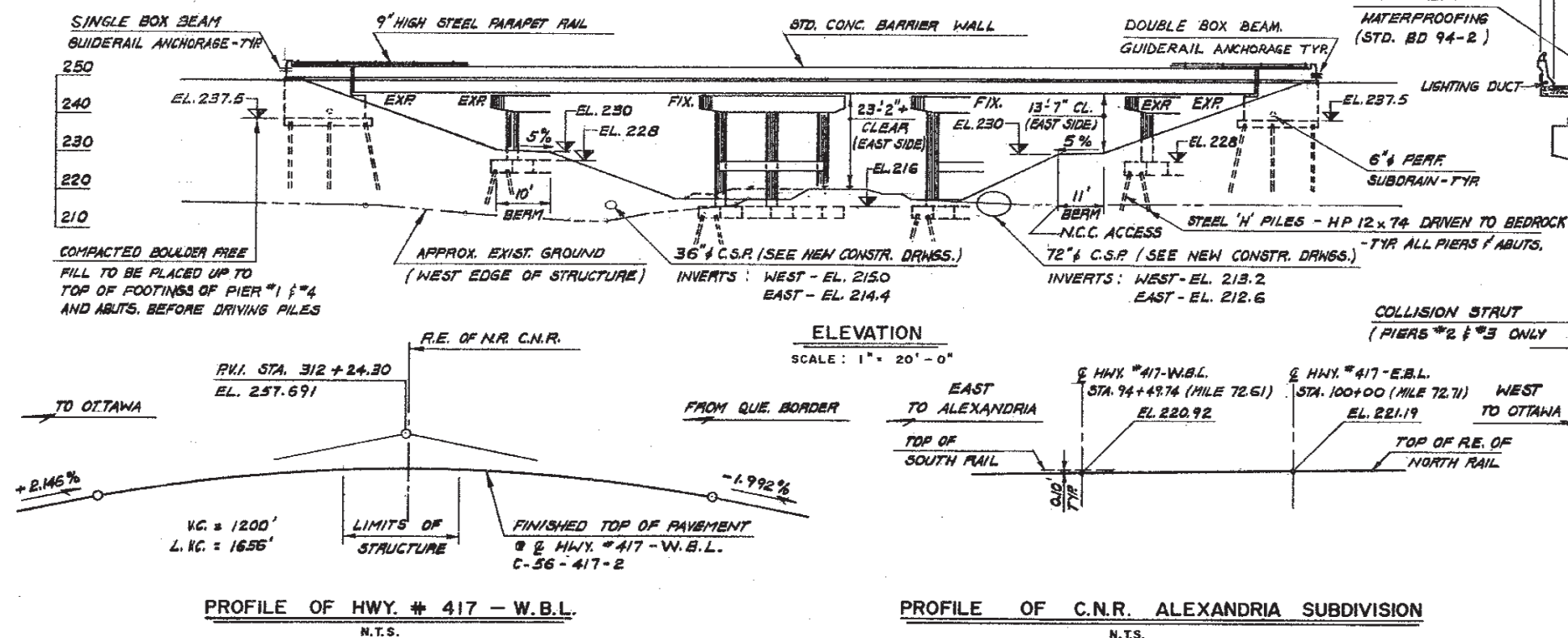


Fred Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

Attachments



REFERENCE BENCH MARK

B.M. 221.39  
GEODETIC DATUM  
TOP NUT ON N.E. CORNER OF HYDRO TOWER  
795' RT. OF 309 + 31 - W.B.L.

**NOTE**

W.P DENOTES WORKING POINT  
T/P DENOTES TOP OF PAVEMENT

## NOTES

CLASS OF CONCRETE

DECK, BARRIER WALLS & APPROACH SLABS	4,000 RS.
PRESTRESSED GIRDERS	5,000 RS.
REMAINDER	3,000 RS.

CLEAR COVER ON REINF. STEEL:

FOOTINGS, ABUTMENTS & PIER COLUMNS	— 3"
PIER CAPS & COLLISION STRUT	— 2"
DECK TOP & BARRIER WALLS	— 1 1/2"
DECK BOTTOM	— 1"

CONSTRUCTION NOTES

- THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF  $\pm \frac{1}{8}$ ".
- NO CONCRETE IS TO BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.

REVISIONS			
	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO

 Consulting Engineers & Planners

C.N.R. OVERHEAD  
(WEST BOUND LANES)

2.2 MILES EAST OF INNES RD.

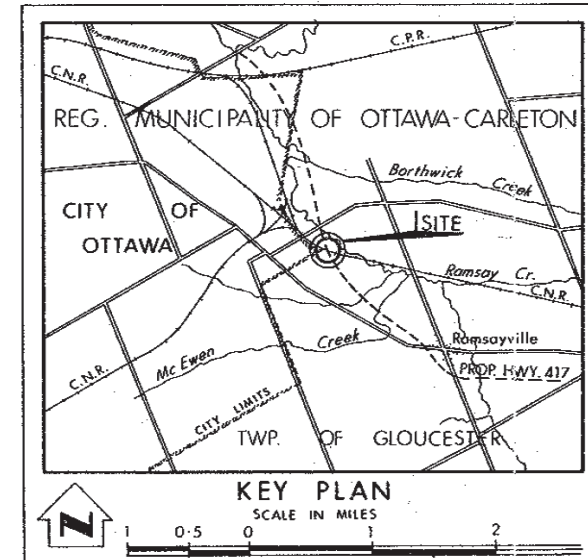
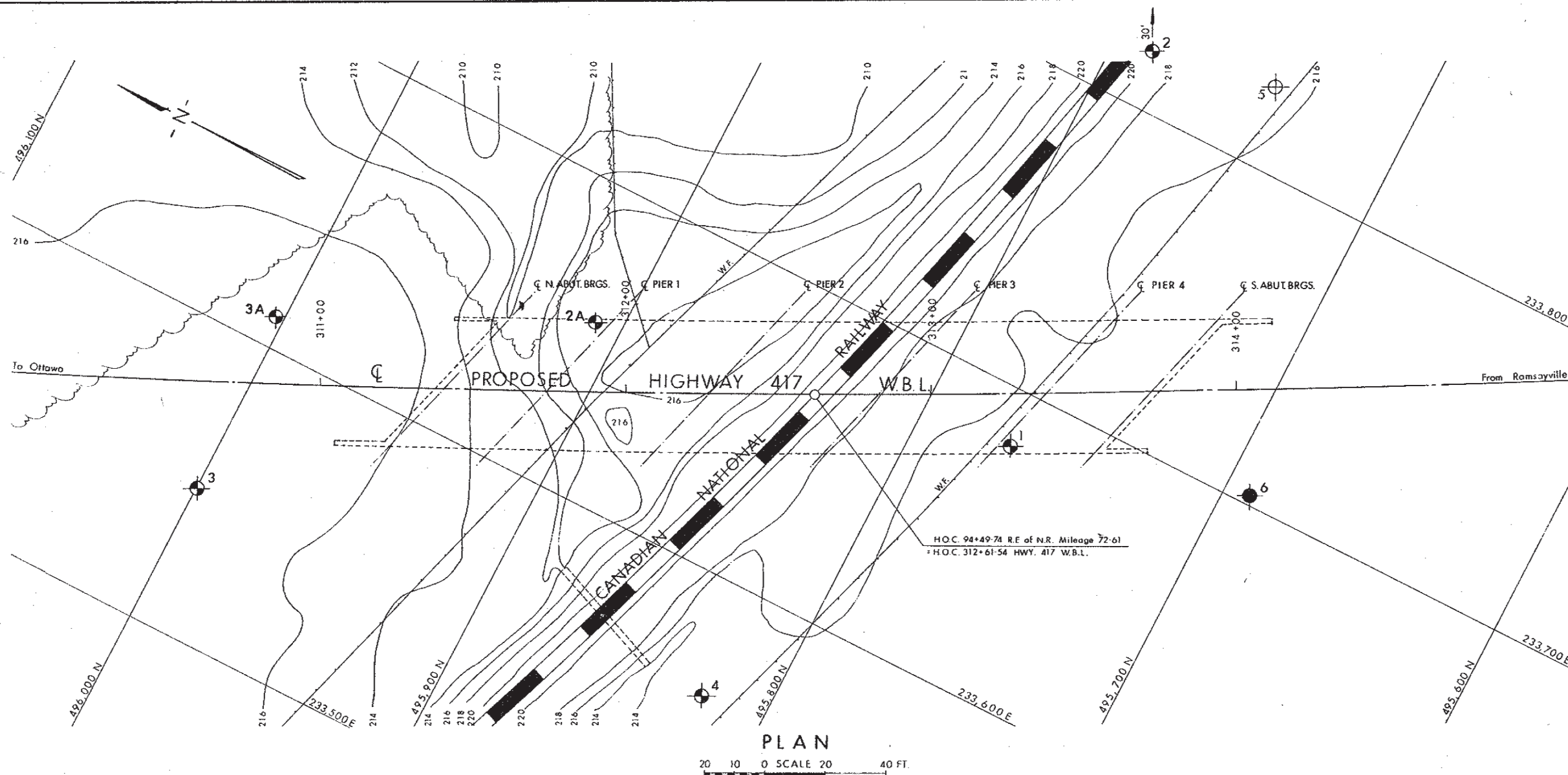
KING'S HIGHWAY No. 417 DIST. No. 9

CO. REG. MUN. OTTAWA - CARLETON

TWP. GLOUCESTER	LOT 2	CON. VI
-----------------	-------	---------

## GENERAL LAYOUT

APPROVED <u>BO</u> STRUCTURAL ENGINEER				SITE No. 3-301 B		W.P. No. 10-69-	
DESIGN <u>WJ</u> CHECK <u>WJ</u>				CONTRACT No.		73 19	
DRAWING C.M.G. CHECK <u>WJ</u>				DRAWING No.		3-301 B - 1	
DATE NOV 22 LOADING MS 20-44							



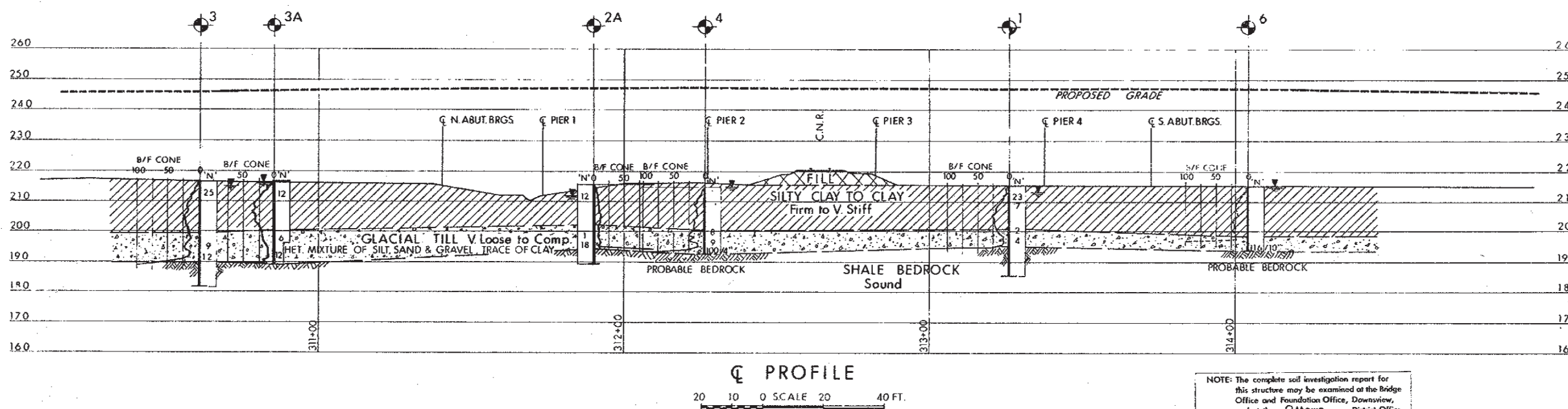
## LEGEND

- Bore Hole
- ⊕ Cone Penetration Test
- ⊕ Bore Hole & Cone Test
- ⊕ Water Levels established at time of field investigation. NOV. & DEC. 71 & APRIL 72.

NO.	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	215.7	495,769	233,682
2	215.7	495,801	233,846
3	216.5	495,999	233,548
4	215.0	495,822	233,562
5	215.9	495,746	233,826
6	215.6	495,692	233,703
2A	215.8	495,909	233,656
3A	217.0	496,003	233,610

## NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.



NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the Ottawa District Office.

MINISTRY OF TRANSPORTATION & COMMUNICATIONS  
DESIGN SERVICES BRANCH - FOUNDATIONS OFFICE

CANADIAN NATIONAL RAILWAY  
(APPROX. 1.3 MILES N. OF RAMSAYVILLE)

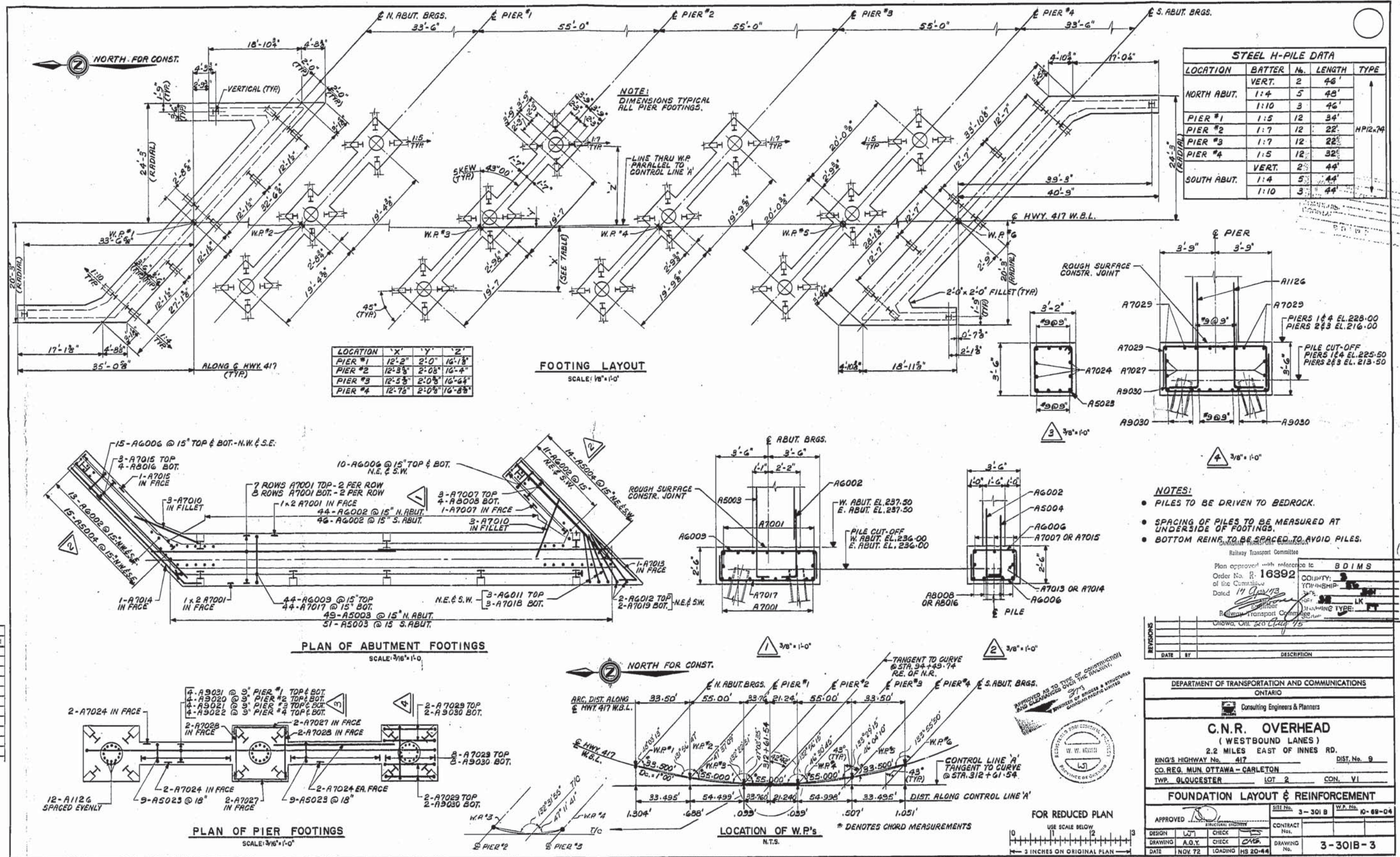
HIGHWAY NO. 417 W.B.L. DIST. NO. 9

CO. REG. MUN. OTTAWA-CARLETON

TWP. GLOUCESTER LOT 2 CON. 6

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. S.A.	CHECKED	W.P. NO. 10-69-04	DRAWING NO.
DRAWN S.O.	CHECKED	JOB NO. 71-11124	71-11124 A
DATE 27 JAN. 1972		SITE NO. 3-3018	BRIDGE DRAWING NO.
APPROVED		CONT. NO. 73-191	3-3018-2
PRINCIPAL FOUNDATION ENGINEER			



- NOTES:**
- PILES TO BE DRIVEN TO BEDROCK.
  - SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTINGS.
  - BOTTOM REIN TO BE SPACED TO AVOID PILES.

Plan approved with reference to B.D.I.M.S.  
Order No. R-16392  
COUNTY: YORK-SHIRE  
Dated: 17 April 73  
Engineer: [Signature]  
Railway Transport Committee  
Ottawa, Ont. 26 April 75

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO  
Consulting Engineers & Planners  
**C.N.R. OVERHEAD**  
(WESTBOUND LANES)  
2.2 MILES EAST OF INNES RD.  
KING'S HIGHWAY No. 417 DIST. No. 9  
CO. REG. MUN. OTTAWA - CARLETON  
TWP. GLOUCESTER LOT 2 CON. VI  
**FOUNDATION LAYOUT & REINFORCEMENT**  
APPROVED: [Signature]  
DESIGN: L.J. CHECK: [Signature]  
DRAWING: A.G.Y. CHECK: [Signature]  
DATE: NOV 72 LOADING: HS 20-44  
CONTRACT No.: 3-301B W.P. No.: 10-88-04  
DRAWING No.: 3-301B-3



**APPENDIX 14**  
**SITE 3-265/1**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Fred J. Griffiths, Ph.D., P.Eng.  
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 EASTBOUND RAMSAYVILLE OVERPASS (SITE 3-265/1) GWP 4074-11-00 GEOCRES 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing eastbound Highway 417 Overpass of Ramsayville Road in Ottawa, Ontario. As part of the preliminary investigation seismic piezocone penetration testing was carried out to confirm the appropriate seismic soil profile for this site. It also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4145-10-01
- Site No. 3-265/1
- GEOCRES No. 31G5-71
- Construction Contract 73-190
- Historic W.P. 34-66-10

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa in the Township of Gloucester approximately 2 km west of Anderson Road. The bridge carries the Highway 417 eastbound lanes (two lanes plus paved shoulders) over Ramsayville Road. Based on the historic General Plan Drawing (copy attached), the bridge is an 11 m wide, 94 m long, five-span, precast concrete structure. The two bridge abutments and four piers are supported by steel 12BP74 piles driven to bedrock.

The natural terrain in the vicinity of the bridge includes the Ramsay Creek valley which is about 8 m deep with a flat valley floor about 37 m wide; the natural valley walls have a slope of about 3H:1V (Horizontal:Vertical). At the time of Thurber's site visit the creek was flowing south to north under the bridge through a corrugated steel pipe culvert and had a depth of approximately 0.3 m.



The approach fills were to be constructed by placing as much as 3.7 m of fill with a slope of 2H:1V. The available site historical documents indicate that the culvert was constructed after completion of Highway 417 Bridge 3-265/1. The creek had to be diverted to between Piers 3 and 4 to allow construction of Ramsayville Road between Piers 2 and 3. Given the elevation of the west side of Ramsayville Road on the west side of Pier 3 and the elevation of the creek bed on the east side of Pier 3, it was concluded that these unequal loads could cause Pier 3 to tilt. The placement of compensating fill to the east of Pier 3 necessitated the inclusion of the steel pipe arch culvert

Ramsayville Road passes beneath the structure in a north-south direction. It includes two 3.66 m lanes and 2.44 m wide gravel shoulders. It was constructed on a fill which has 2H:1V side slopes.

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRETS Report No. 31G5-71 dated September 1968. The investigation consisted of five boreholes, all accompanied by dynamic cone penetration tests. Drawing No. 68-F-54A (copy attached) illustrates the locations of the bridge, the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by a thin clayey silt layer, overlying a sensitive marine clay layer, overlying clayey silt glacial till, overlaying shale bedrock.

#### **3.1 Clayey Silt**

The top of the silty sand to sandy silt layer ranged from 69.3 m and 72.5 m in elevation, and the layer had a thickness of 0.9 m and 1.5 m.

#### **3.2 Clay**

The surface of the sensitive marine clay ranged from 76.1 to 67.7 m in elevation, and the layer had a thickness of 31.4 to 38.1 m. Atterberg Limits test results indicate a liquid limit from 45% to 86%, and a plastic limit from 19% to 32%. The moisture content of the samples tested ranged from 29% to 93%. The shear strength of the clay based on the in-situ field vane tests ranged from 100 kPa at an elevation of 73 m, decreasing with depth to 30 kPa at an elevation of 70 m, increases with depth to 70 kPa at an elevation of 55 m and then remains constant at 70 kPa to the base of the unit. Results of the shear strength testing indicate a very stiff consistency decreasing to firm with depth with a sensitivity between 2 and 11.

Consolidation testing was carried out on two samples and indicated that the deposit is slightly over-consolidated with a pre-consolidation stress about 100 kPa above existing stresses.

#### **3.3 Glacial Till**

Underlying the clay stratum is a clayey silt glacial till deposit. The surface of this deposit ranged from 36.1 to 38.0 m in elevation, and the layer had a thickness of 11.8 to 13.4 m. The standard penetration test 'N' values varied greatly for this deposit ranging from 17 to greater than 100 blows per 0.3 m penetration, indicating a compact to very dense condition. Gradation test results on samples of this material indicate a gravel content between 3% and 34%, sand content between 13% and 53% and a fines content (combined silt and clay content) between 24% and 84%.



The moisture content of the samples tested ranged from 5% to 21%.

### **3.4 Bedrock**

Beneath the glacial till layer shale bedrock was encountered in four boreholes with surface elevations ranging from 23.2 to 24.3 m. The bedrock was described to be in sound condition. Geological mapping suggests the bedrock at this site is of the Carlsbad Formation.

### **3.5 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 71.9 m to 72.2 m along the banks of the valley and at an elevation of 68.3 m to 69.5 m along the valley floor. Artesian conditions were noted in three boreholes with a pressure head rising to elevation of 73.5 m.

## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in March 2012 and again in June 2012 for Bridge 3-265/1 with the report issued December 2012. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in fair to good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering staff during the week of July 14<sup>th</sup>, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

- No evidence of slope stability issues were noted on the well vegetated side slopes of the approach fills
- The foreslopes of both abutments were covered by riprap but some minor erosion was noted on the south side of the east foreslope
- The south slope of the east approach fill was noted to have a very steep slope of about 1.5H:1V down to a swampy area vegetated by cattails
- Undermined riprap erosion protection was noted in the ditch just south of the east abutment
- No major settlement issues were observed at the site but both approach embankments were noted to have minor dips in the road surface just before the abutment approach slabs. The ride across the transition from deck to approaches was relatively smooth



## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

#### **5.2 Seismic Considerations**

##### **5.2.1 Seismic Piezocone Penetration Testing**

Based on the review of available historical data, the existing soil stratigraphy at Site 3-265/1 would be at or near the threshold to be characterized as either a Seismic Soil Profile Type III or Type IV as defined in the Canadian Highway Bridge Design Code (CHBDC–CAN/CSA-S6-06). Seismic piezocone penetration testing (SCPTu) was carried out to determine the in-situ shear wave velocity profile in order to confirm the appropriate Seismic Soil Profile for this site.

Thurber engaged ConeTec Investigations Limited (ConeTec) to carry out the SCPTu investigation for this assignment. The field investigation included advancing a SCPTu sounding, designated SCPT15-01, within the right-of-way of Ramsayville Road. The locations and ground surface elevations of the SCPTu sounding are summarized in Table A and are also illustrated on the SCPTu Location Plan (copy attached).

**Table A: SCPTu Testing Summary**

<b>Sounding ID</b>	<b>Location</b>	<b>Ground Surface Elevation (m)</b>	<b>Depth (m)</b>	<b>Number of Shear Wave Velocity Tests</b>
SCPT15-01	West shoulder of Ramsayville Road 20 m north of EB bridge	73.1	39.9	39

A velocity profile for the soil stratigraphy was developed by carrying out shear wave velocity tests at one meter intervals. A description of the equipment and testing methodology used for this investigation is provided in ConeTec's Report No. 15-05005 (copy attached).

##### **5.2.2 Shear Wave Velocity Test Results and Interpretation**

The shear wave velocity test results are tabulated in ConeTec's report for Sounding SCPT15-02.

The current CHBDC (Section 4.4.6.5) indicates that Soil Profile Type IV is a profile with soft clays or silts greater than 12 m in thickness and that these materials are characterized by a shear wave velocity less than 150 m/s.



The results of the shear wave velocity testing indicate between 10 and 11 m of soil profile with a shear wave velocity of less than 150 m/s and an average shear wave velocity within the upper 30 m of 160 m/s in SCPT15-01.

Based on the shear wave velocity data the site are classified as Soil Profile Type III under the current CHBDC.

### **5.2.3 Liquefaction**

The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the plasticity of the clay at this site, it is classified as “not susceptible” to liquefaction during an earthquake event.

### **5.3 Existing Foundations**

The General Plan Drawing indicates that the abutments and piers of the existing structure are supported on 12BP74 steel piles driven to bedrock. The contract drawings indicate the design load for the piles of is 90 tons/pile or approximately 800 kN/pile driven to sound bedrock. The grade of steel for the existing piles is not indicated.

12BP74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN / HP310x110 pile driven to bedrock. The SLS condition will not govern for piles end-bearing in or on bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Shallow Foundations**

The very stiff to firm silty clay to clay deposit is not considered suitable to carry a bridge structure. The depth of the top of the till deposit was observed to range in excess of 30 m below original grade which is considered too deep for spread footing foundations. As such spread footings within the overburden are not recommended and deep foundations are preferred at this site.

### **6.2 Deep Foundations – Piles**

Based on the review of existing subsurface stratigraphy at the site driven piles are considered to be suitable for the support of abutment and pier foundations. It should be noted that the bedrock surface elevation ranges from 23.2 to 24.3 m.



### **6.2.1 Axial Resistance**

Due to the anticipated length of the piles (over 40 m) and thickness of the glacial till layer beneath the sensitive clay, the piles may reach practical refusal in the lower part of the glacial till layer. Therefore, it is recommended that the design use steel HP section piles driven to practical refusal.

The design parameters for axial resistance of both HP310x110 and HP310x132 piles driven to practical refusal within the glacial till deposit or upper shale bedrock can be taken as:

- 1,800 kN factored geotechnical resistance at ULS; and
- 1,600 kN axial resistance at SLS

It is noted that the piles will penetrate through the deep clay deposit and into or through the glacial till deposit where artesian groundwater conditions have been observed. Due to the thickness of the clay, artesian flow up the pile shaft is not expected to be a concern. Furthermore, it is noted that the existing bridge structures are supported on steel H-Piles driven to similar depths and no problems with artesian flow up the pile shafts were noted in the review of the construction history.

### **6.2.2 Pile Tips**

New piles will be driven through a thick glacial till deposit which may contain cobbles and/or boulders, as such the pile tips should be protected from damage during driving.

### **6.2.3 Integral Abutment Considerations**

As per the General Plan Drawing the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### **6.2.4 Pile Spacing**

Since new piles at this site will be end bearing on a hard layer, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.

### **6.2.5 Downdrag**

Downdrag forces need to be considered for piles in areas where grades have increased from original. This includes the piles supporting the existing abutments and Piers 2 and 3. It is anticipated that the design of new piles required to support a widened foundation at both the abutments and Piers 2 and 3 will also need to include downdrag loads due to approach fill widening. Downdrag loads for piles at Piers 1 and 4 are not anticipated unless grades are modified at these pier locations.



The following pile SLS dead loads were provided by the structural design team in order to determine the downdrag forces acting on Structure 3-265/1:

- Abutment pile loading = 155 kN / pile
- Pier pile loading = 315 kN / pile

The value of downdrag on existing and new piles should be considered for this site as follows:

Existing piles:

- The abutment piles are being subject to unfactored downdrag loads of approximately 300 kN/pile due to the original placement of as much as 3.7 m of fill at the approaches. Placement of additional fill for embankment widening or grade raises will not add further downdrag load to these piles.
- The piles supporting Piers 3 and 4 are being subject to unfactored downdrag loads of approximately 250 kN/pile due to the construction of Ramsayville Road.
- Placement of additional fill for embankment widening or grade raises will not add further downdrag load to these piles.

New Piles:

If there are no modifications to the width or height of the approach fills or in the areas of the piers, new piles at the site will not be subject to downdrag loads. Should modifications be required, assuming the same SLS dead loads listed above and that any additional approach fills will match existing grades; new piles at Bridge Structure 3-265/1 would be subject to the following downdrag loads:

- New abutment piles will be subject to unfactored downdrag loads of approximately 300 kN/pile due to the placement of fill to address widening and/or grade raises at the approaches.
- New piles at Piers 2 and 3 will not be subject to downdrag loads unless modifications to Ramsayville Road are made and/or the grades are modified at these pier locations.

The downdrag load should be multiplied by a load factor of 1.25 as per the CHBDC Commentary Clause C6.8.4 to obtain a factored downdrag load. In accordance with Section 6.8.4 of the CHBDC and clause C6.8.4 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag loads should not exceed the factored structural resistance of a pile. In geotechnical analysis of downdrag, live load effects should not be considered.

## **6.2.6 Lateral Resistance of Piles**

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formulae:



For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

For cohesive soils:

$$k_h = \frac{67S_u}{B}$$

where:

- $k_h$  = coefficient of horizontal subgrade reaction
- $n_h$  = coefficient related to soil density given in Table B
- $z$  = depth of pile embedment (m)
- $B$  = pile width perpendicular to load direction (m)
- $S_u$  = undrained shear strength given in Table B

**Table B:**  $n_h$  values for cohesionless soils and  $S_u$  values for cohesive soils.

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )	$S_u$ (kPa)
Above 73.0	Embankment Fill	3000	-
Between 73.0 and 70.0	Native Clay	-	100 to 30
Between 70.0 and 55.0	Native Clay	-	30 to 70
Between 55.0 and 36.0	Native Clay	-	70
Below 36.0	Glacial Till	2000	-

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table C. Intermediate values may be obtained by linear interpolation.

**Table C:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <b>perpendicular</b> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <b>parallel</b> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

## 6.2.7 Uplift Resistance

Due to the thickness of the clay and till deposits at this site and the fact that uplift resistance is developed through shaft resistance it is not anticipated that uplift resistance will be an issue.

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

$C$  = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table D

$\sigma'_v$  = effective vertical stress (kPa)

**Table D:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 73.0	Embankment Fill	20	0.4
Between 73.0 to 36.0	Native Clay	17	0.25
Below 36.0	Glacial Till	19	0.4

A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.



### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table E are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table E:** Static Lateral Earth Pressure Coefficients

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Clay
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	21.0	20.0	17
Angle of Internal Friction, $\phi$	35°	30°	27°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.50	0.55
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33	0.38

## 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table F.

**Table F:** Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Clay
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21.0	20.0	17
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$	$27^\circ$
<b>Yielding Wall</b>			
$K_{AE}$	0.33	0.4	0.45
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36
<b>Non-Yielding Wall</b>			
$K_{AE}$	0.55	0.66	0.74
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil ( $\text{kN/m}^3$ )

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.



The factors provided in Table F are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## **6.5 Approach Embankments**

Based on the original Foundation Design Report, the embankment soils consist of compact to dense deposits of non-cohesive soil to a maximum height of 3.7 m above the original ground surface. An embankment slope of 2H:1V was recommended to be used. Settlement of up to 150 mm due to the construction of the approaches. The embankment foundation is expected to be stable and no further consolidation settlement is expected unless the fills are reconfigured.

It is anticipated that settlement of the native clay will occur should the embankments be widened or if additional lanes are added to Highway 417. Further settlement of the existing embankment and roadway may also occur due to the increase in stress caused by the project works.

## **6.6 Erosion Control**

The erosion protection measures at the site should be repaired and maintained to prevent erosion of the abutment slopes.

## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The existing embankment fill and native clay at this site should be classified as Type 3 in accordance with the OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that piezometers be installed to better define the groundwater level as



well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

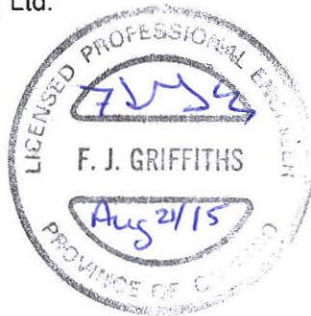
During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or a new structure adjacent to the existing.

If it is determined that the Ramsay Creek Culvert should be removed or replaced a soil investigation and slope stability analysis should be undertaken to ensure the foreslope of the east abutment remains stable. In addition the eccentric loading on Pier 3 should be assessed.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.

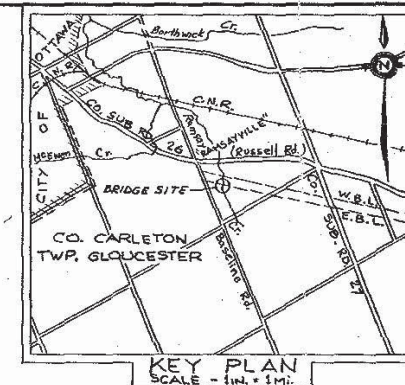
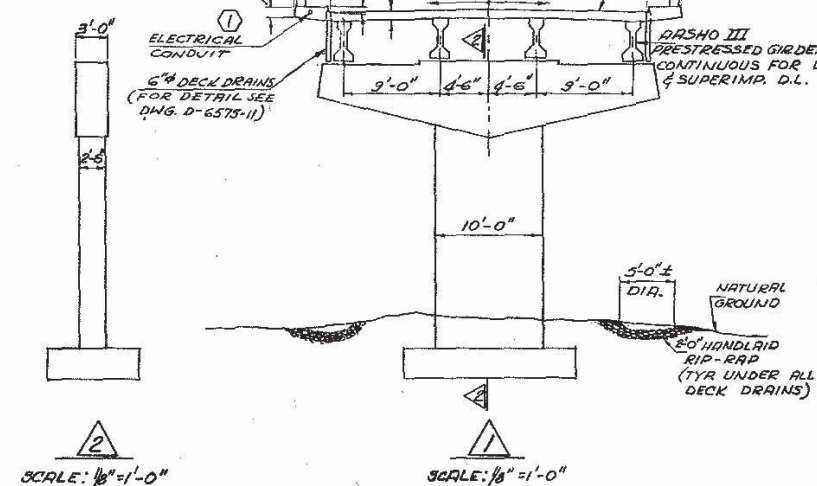
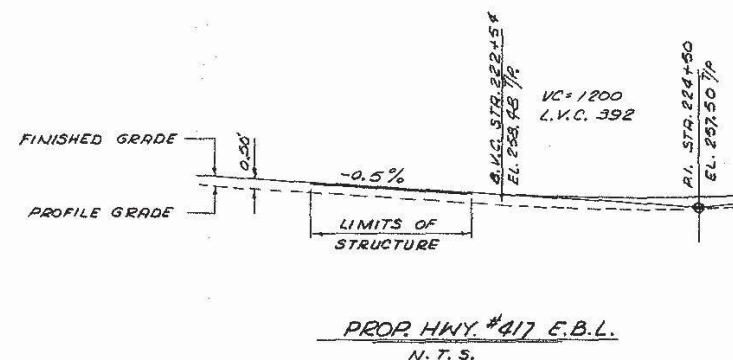
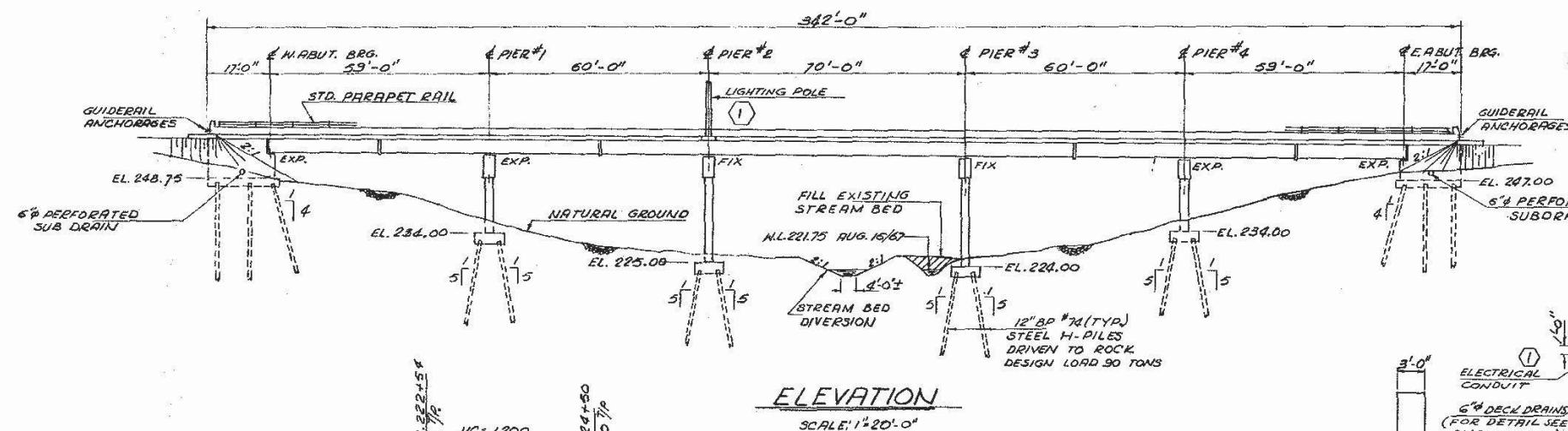
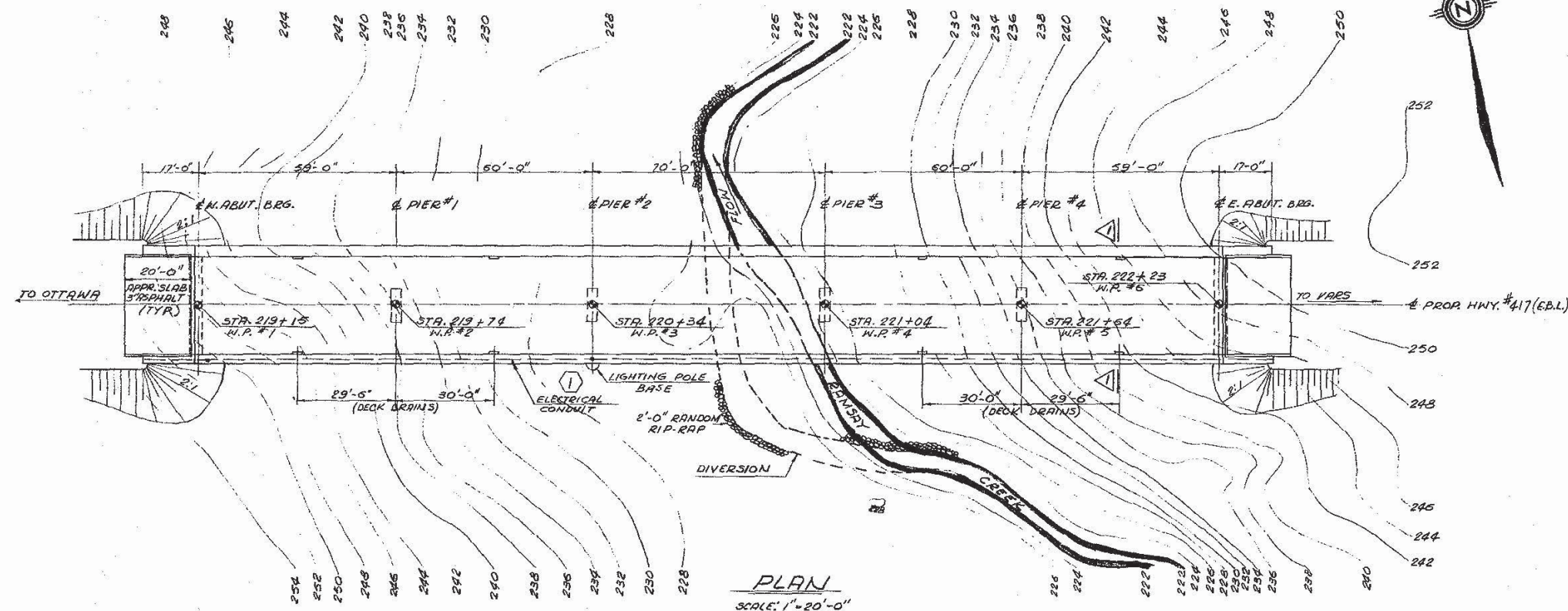


Fred Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

## Attachments

**NOTES:****1. CLASS OF CONCRETE**

PRECAST MEMBERS - 5000 P.S.  
DECK, CURBS & PARAPET WALLS - 4000 P.S.  
REMAINDER - 3000 P.S.

**2. CLEAR COVER ON REINFOR. STEEL**

FTGS., ABUTS & PIERS	CURBS	PARAPET WALLS	DECK
3"	2"	1 1/2"	TOP 1 1/2" BOT. 1"

**3. CONSTRUCTION NOTES:**

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF  $\pm 1/8"$ . NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.

**LIST OF DRAWINGS**

1. GENERAL PLAN
2. BOREHOLE LOCATIONS & SOIL STRATA
3. FOOTING LAYOUT & REINFORCEMENT
4. ABUTMENTS & WINGWALLS
5. PIERS
6. PRESTRESSED GIRDERS & BEARINGS
7. DECK DETAILS & ELEVATIONS
8. PARAPET WALL DETAILS
9. STANDARD STEEL PARAPET RAIL
10. APPROACH SLABS
11. STANDARD DETAILS
12. BRIDGE ELECTRICAL DETAILS



REVISIONS	DATE	BY	DESCRIPTION
1	11/19/69	J.S.	NOTE RE: APPR. SLAB REMOVED; LIGHTING POLE & ELECTRICAL CONDUIT ADDED; 1/2" ADDED

DEPARTMENT OF HIGHWAYS ONTARIO  
BRIDGE DIVISION

**RAMSAY CREEK BRIDGE**

E.B.L. STRUCTURE

(0.4 MILES SOUTHEAST OF RAMSAYVILLE)

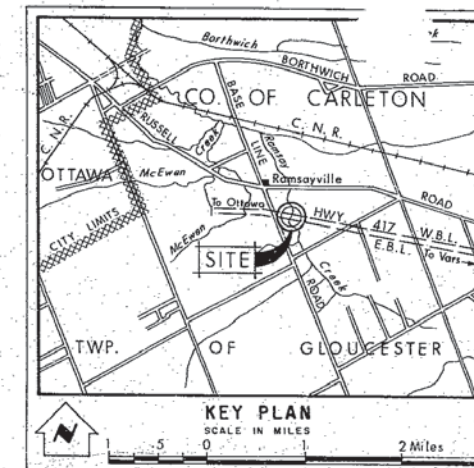
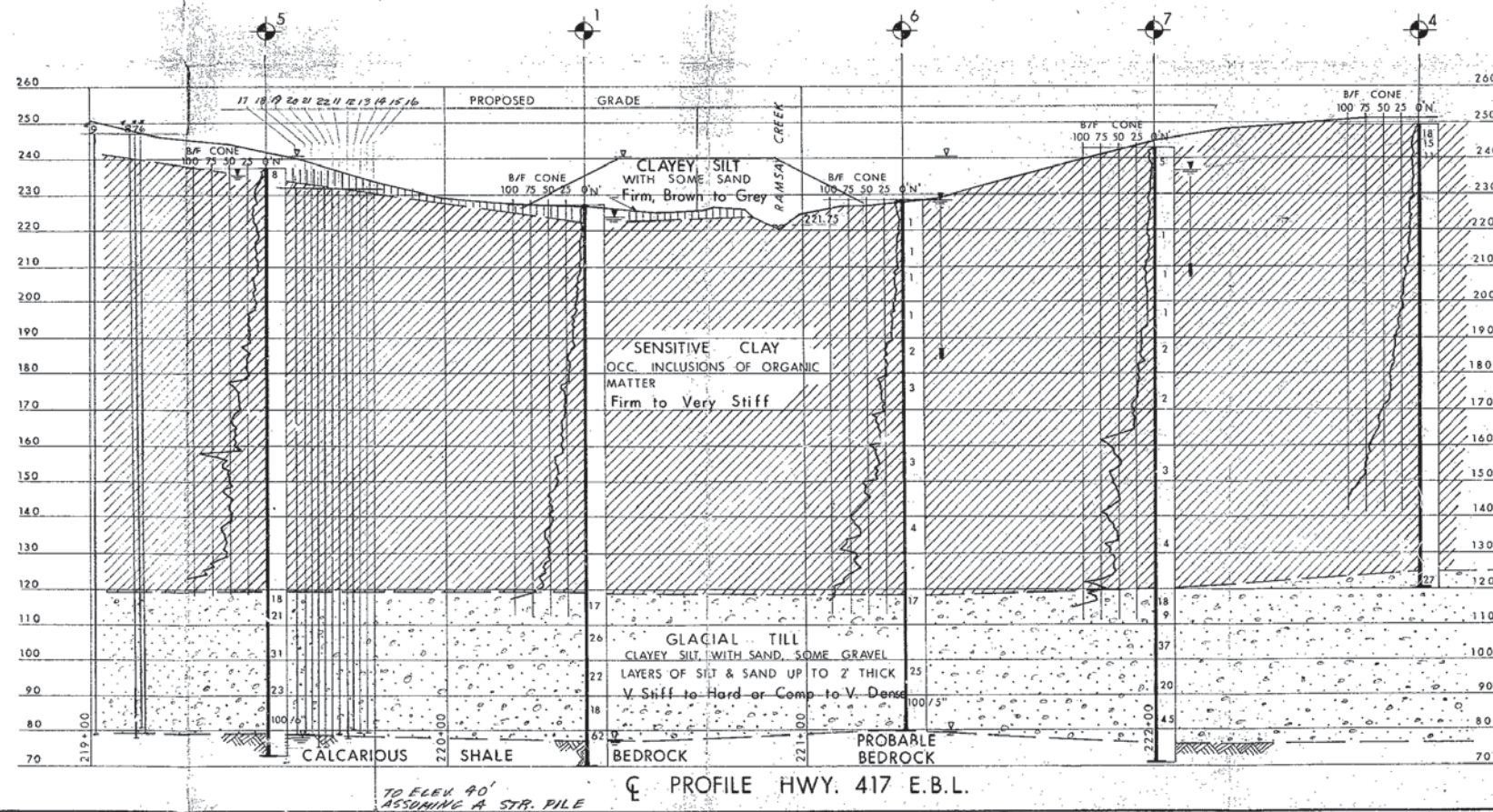
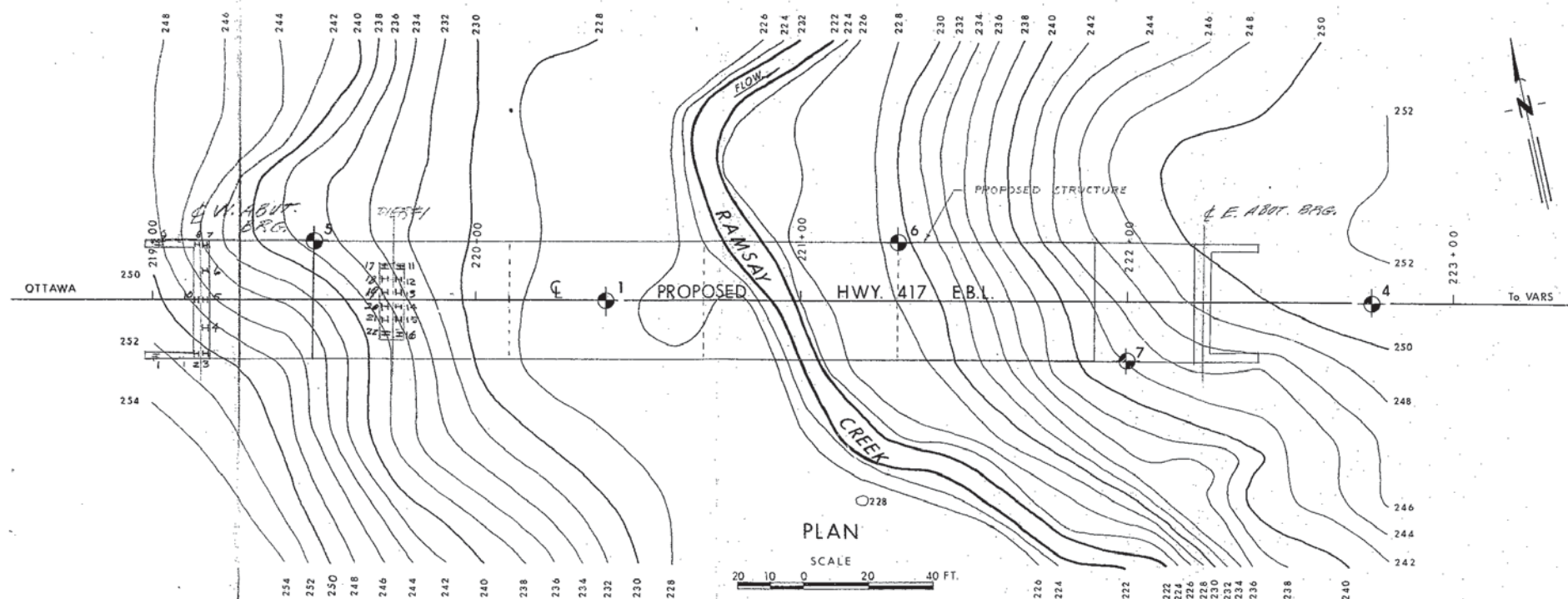
KING'S HIGHWAY No. 417 (E.B.L.) DIST. No. 9

CO. CARLETON LOT 20 CON. 5

TWP. GLOUCESTER

**GENERAL PLAN**

APPROVED	DATE	NOV/56	LOADING	120-22
DESIGN	J.S.	CHECK	S.B.D.	CONTRACT
DRAWING	B.S.	CHECK	S.B.D.	DRAWING
DATE	NOV/56	LOADING	120-22	No.



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation. AUG. 16/1967		
	Head Encountered		
	PIEZOMETER		
NO.	ELEVATION	STATION	OFFSET
1	227.2	220+40	6' E.B.L.
4	249.6	222+75	6' E.B.L.
5	237.8	219+50	18' LT.
6	228.3	221+30	18' LT.
7	243.0	222+00	18' RT.

**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

**RAMSAY CREEK**

KING'S HIGHWAY NO. 417 E.B.L. DIST. NO. 9  
CO. CARLETON  
TWP. GLOUCESTER LOT 20 CON. V

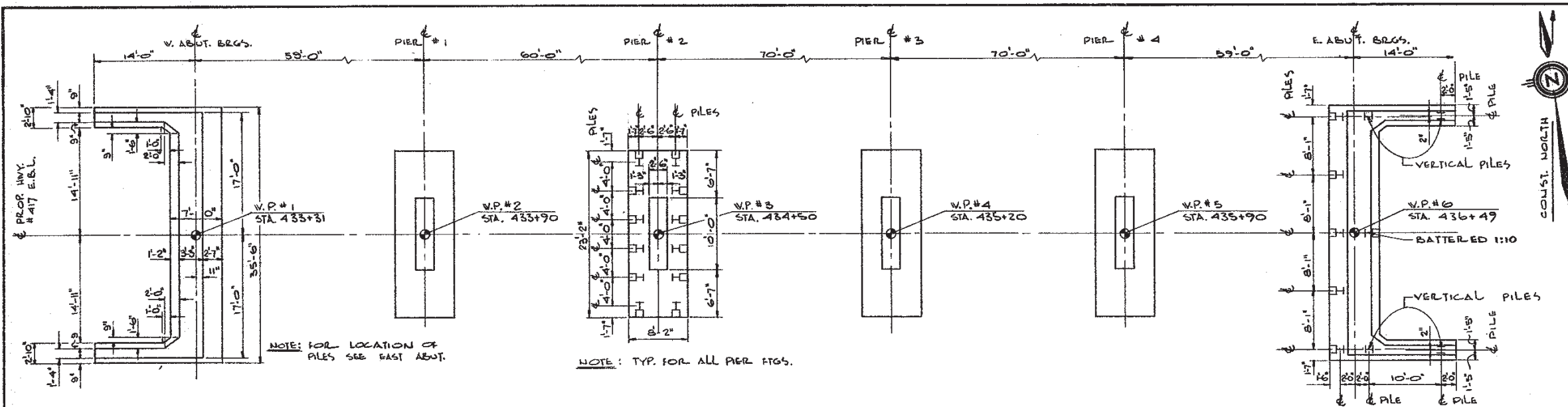
**BOREHOLE LOCATIONS & SOIL STRATA**

SUBMD. W.H. CHECKED ☒ W.P. NO. 34-66-01 M.B.T. DRAWING NO.  
DRAWN A.N. CHECKED ☒ JOB NO. 68-F-54 68-F-54A  
DATE SEPT. 23/1968 SITE NO. BRIDGE DRAWING NO.  
APPROVED *A. J. Thomas* CONT. NO.

PILE NO.	TIP ELEVATION
6	79.3
7	80.8
8	78.5
9	79.0
10	79.0
11	79.0
12	79.0
13	79.0
14	79.0
15	79.0
16	79.0

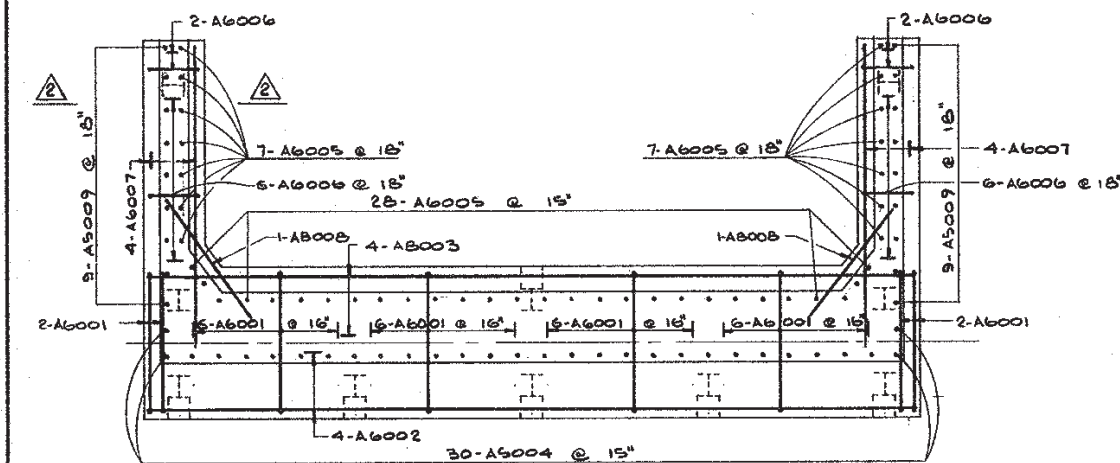
NO. 16 BELY. ASSUMING A STR. PILE

PRINT RECORD	NO.	FOR	DATE

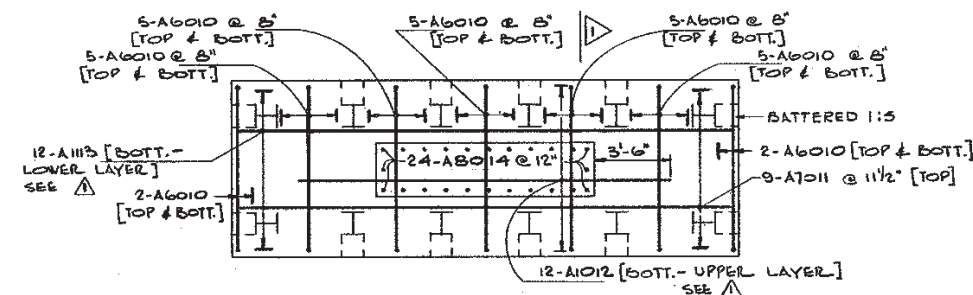


STEEL H-PILE DATA			
LOCATION	NO.	LENGTH	TYPE
WEST ABUT.	10	114'-0"	H-PILES 12B974
PIER 1	12	100'-0"	
PIER 2	12	97'-0"	
PIER 3	12	88'-0"	
PIER 4	12	92'-0"	
EAST ABUT.	10	111'-0"	
DESIGN LOAD - 90 TONS PER PILE			

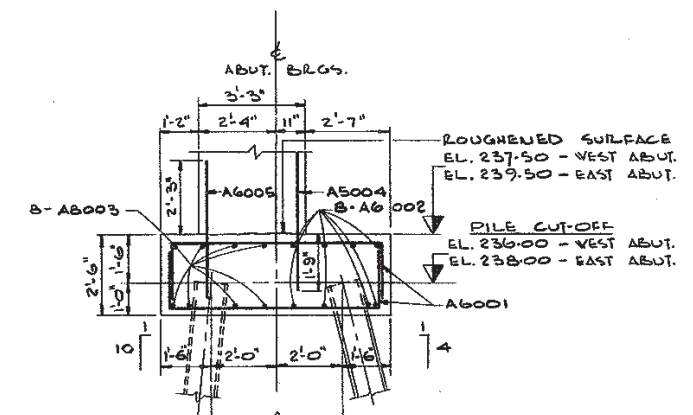
NOTE:  
ALL PILES TO BE DRIVEN TO  
PRACTICAL REFUSAL.



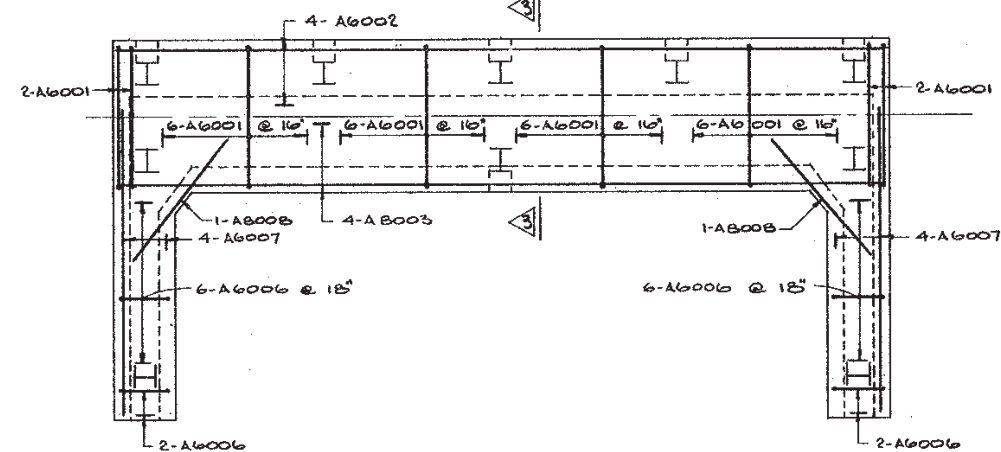
TOP REINFORCEMENT



PLAN OF PIER FOOTINGS



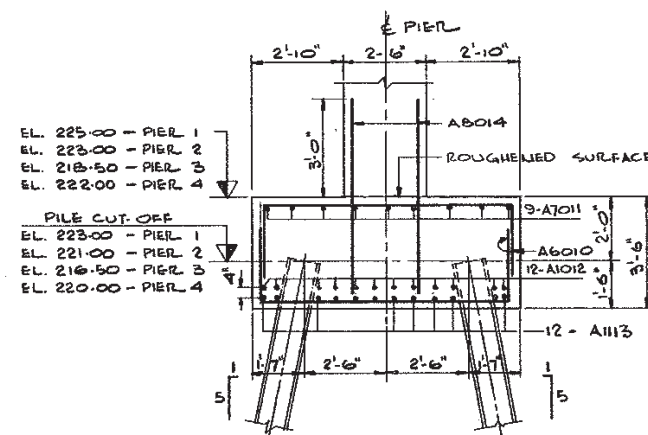
ABUT. BRGS. SCALE: 3/8" = 1'-0"



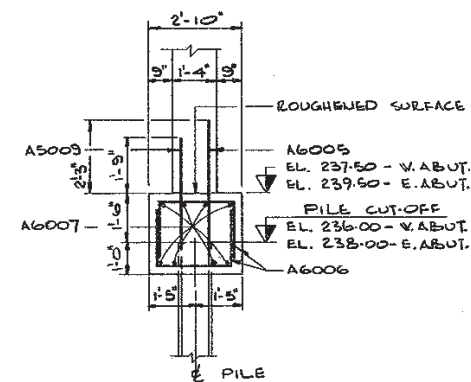
BOTTOM REINFORCEMENT

PLAN OF ABUTMENT FOOTINGS

SCALE: 1/4" = 1'-0"



PIER SCALE: 3/8" = 1'-0"



SCALE: 3/8" = 1'-0"



REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO  
BRIDGE DIVISION

**BEAR BROOK BRIDGE**  
1 MILE SOUTH OF CARLSBAD SPRINGS  
E.B.L. STRUCTURE  
KING'S HIGHWAY No. 417 E.B.L. DIST. No. 9  
CO. CARLETON  
TWP. GLOUCESTER LOT 6 CON. VII

FOOTING LAYOUT & REINFORCEMENT

APPROVED	DESIGN	CHECK	DATE	LOADING	DATE	CONTRACT	DRAWING
J.S.	P	NOV/68				3-266	D-6467-3

## PRESENTATION OF SITE INVESTIGATION RESULTS

**Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd  
Ottawa, Ontario**

*Prepared for:*

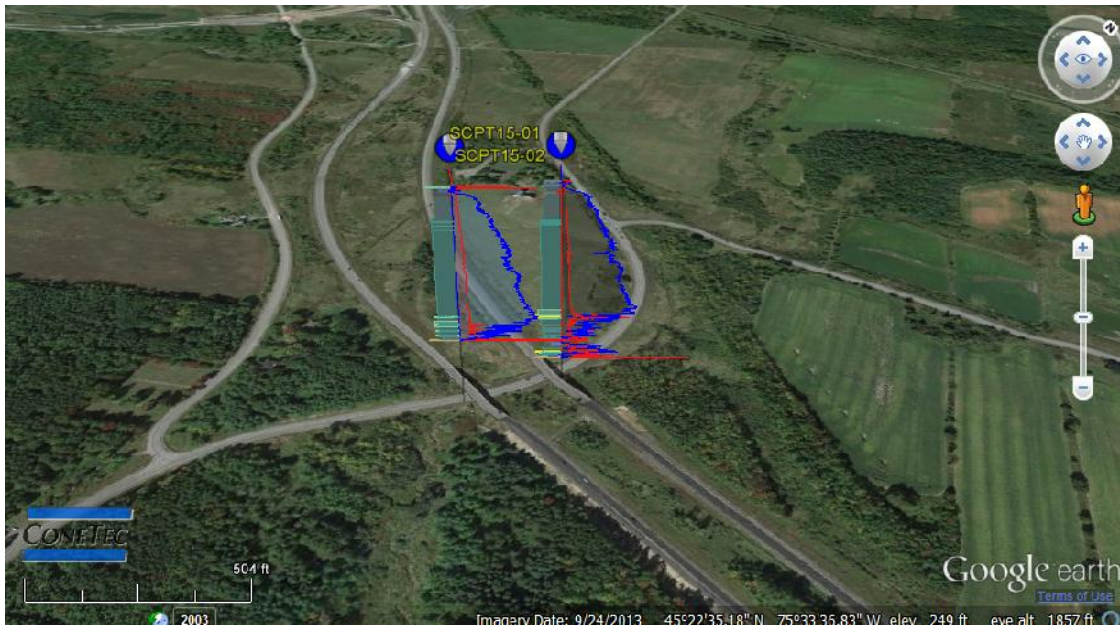
Thurber Engineering

ConeTec Job No: 15-05005

Project Start Date: 18-Feb-2015

Project End Date: 18-Feb-2015

Report Date: 24-Feb-2015



*Prepared by:*

ConeTec Investigations Limited  
9033 Leslie Street, Unit 15  
Richmond Hill, ON L4B 4K3

Tel: (905) 886-2663  
Toll Free: (800) 504-1116

Email: [conetecon@conetec.com](mailto:conetecon@conetec.com)  
[www.conetec.com](http://www.conetec.com)  
[www.conetecdataservices.com](http://www.conetecdataservices.com)



## Introduction

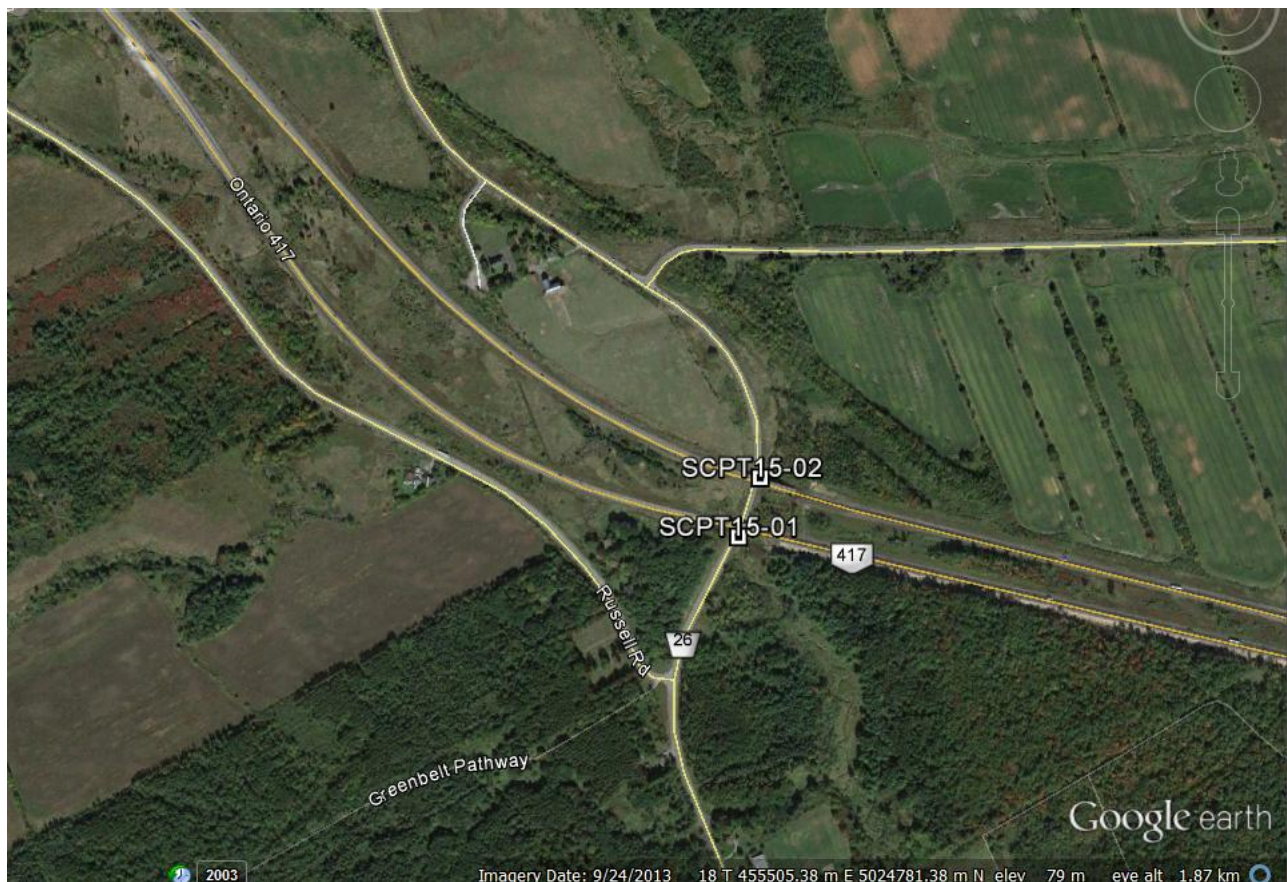
The enclosed report presents the results of a seismic piezocone penetration testing (SCPTu or SCPT) program carried out at the site of a proposed new highway 417 bridge over Ramsayville Road (regional road 26) east of Ottawa, Ontario. The site investigation program was conducted by ConeTec Investigations Limited, under contract to Thurber Engineering of Ottawa, Ontario.

A total of two seismic cone penetration tests were completed at two locations. The SCPT program was performed to evaluate the in situ properties of the soils prior to construction. SCPT sounding locations were selected and numbered under the supervision of Thurber personnel (Mr. Chris Murray).

## Project Information

Project	
Client	Thurber Engineering
Project	Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON
ConeTec project number	15-05005

A map from Google earth including the CPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig	25 ton truck mounted (twin cylinders)	CPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT	GPS (GlobalSat MR-350)	32618 (WGS 84 / UTM North)

Cone Penetration Test (CPT)	
Depth reference	Ground surface at the time of the investigation.
Tip and sleeve data offset	0.1 meter. This has been accounted for in the CPT data files.
Pore pressure dissipation (PPD) tests	One pore pressure dissipation tests were completed primarily to determine excess pore pressure conditions at depth.
Additional Comments	Seismic shear wave velocity testing was performed at two locations at one meter intervals.

Cone Description	Cone Number	Cross Sectional Area (cm <sup>2</sup> )	Sleeve Area (cm <sup>2</sup> )	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
361:T1500F15U500	361	15	225	1500	15	500

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm<sup>2</sup> and 15 cm<sup>2</sup> tip base area configurations in order to maximize signal resolution for various soil conditions. The 15 cm<sup>2</sup> penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm<sup>2</sup> piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u<sub>2</sub>" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

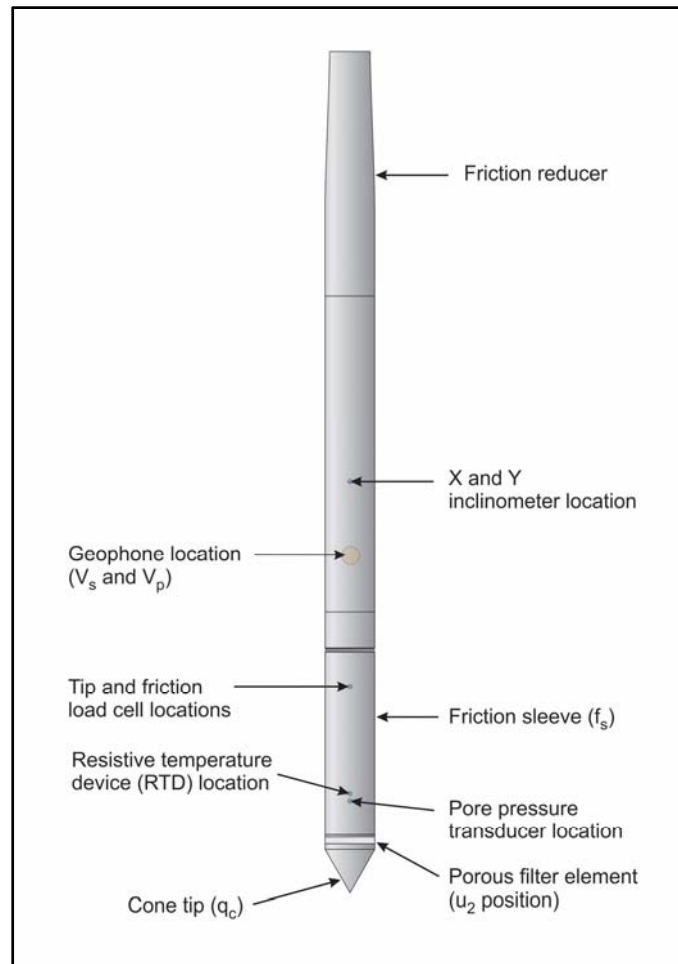


Figure CPTu. Piezocone Penetrometer (15 cm<sup>2</sup>)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance ( $q_c$ )
- Sleeve friction ( $f_s$ )
- Dynamic pore pressure ( $u$ )
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance ( $q_t$ ), sleeve friction ( $f_s$ ) and pore water pressure ( $u$ ). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance ( $q_c$ ) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance ( $q_t$ ) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1-a) \cdot u_2$$

where:  $q_t$  is the corrected tip resistance

$q_c$  is the recorded tip resistance

$u_2$  is the recorded dynamic pore pressure behind the tip ( $u_2$  position)

$a$  is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction ( $f_s$ ) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure ( $u$ ) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio ( $R_f$ ) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high

friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is included in an appendix.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

### References

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420.

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization 4, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355.

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave ( $V_p$ ) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

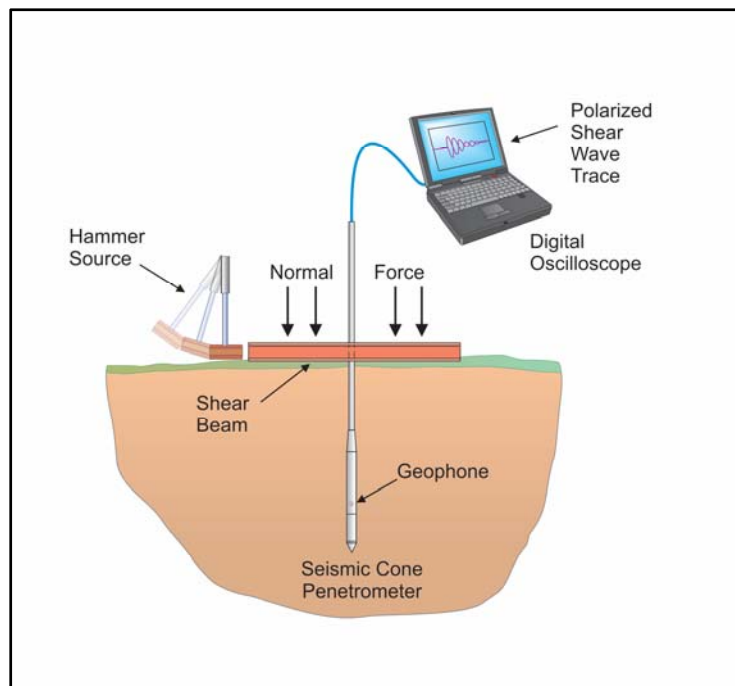


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

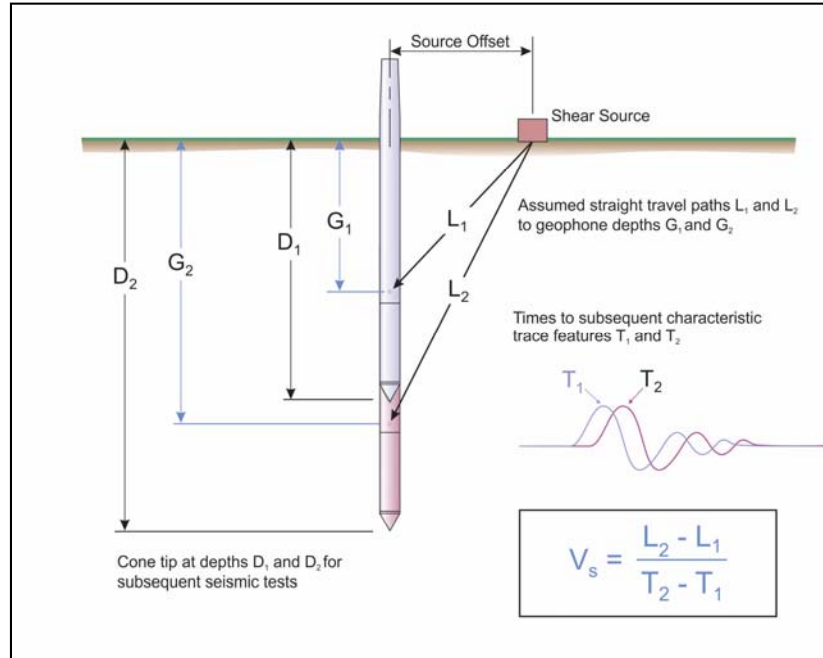


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 30 meters ( $V_{s30}$ ) has been calculated and provided for all applicable soundings using an equation presented in Crow et al., 2012.

$$V_{s30} = \frac{\text{total thickness of all layers (30m)}}{\sum(\text{layer traveltimes})}$$

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

#### References

Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, "National shear wave measurement guidelines for Canadian seismic site assessment", GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure ( $u$ ) with time ( $t$ ).

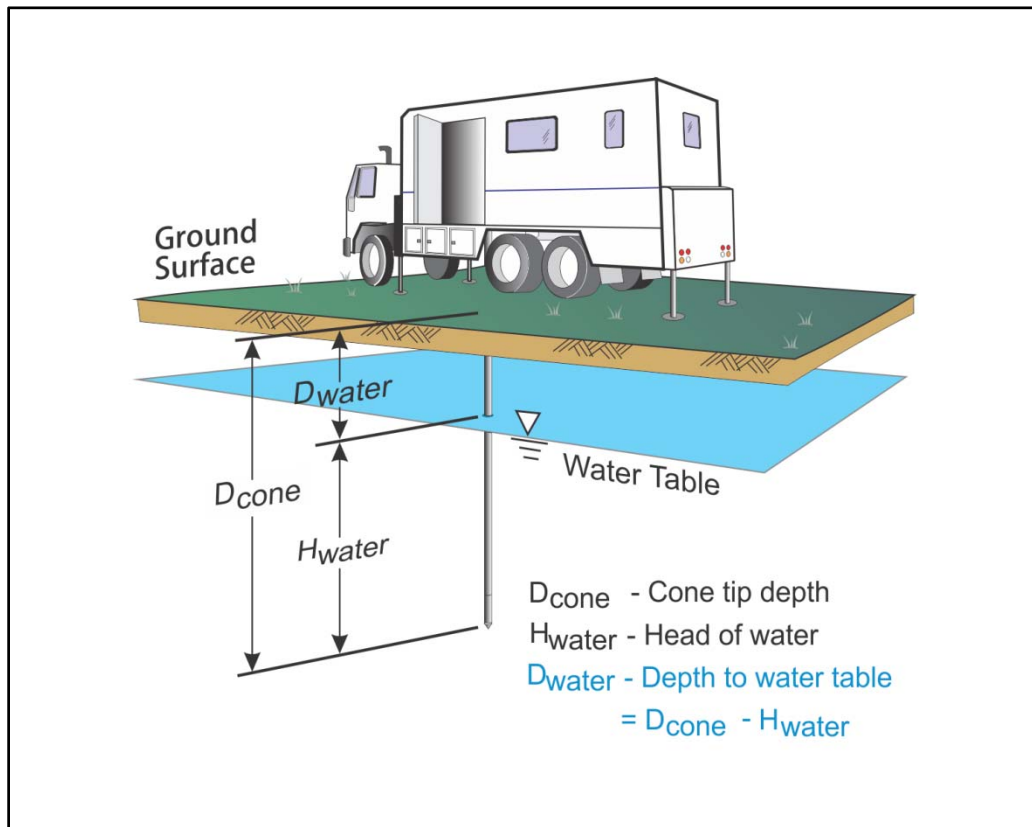


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

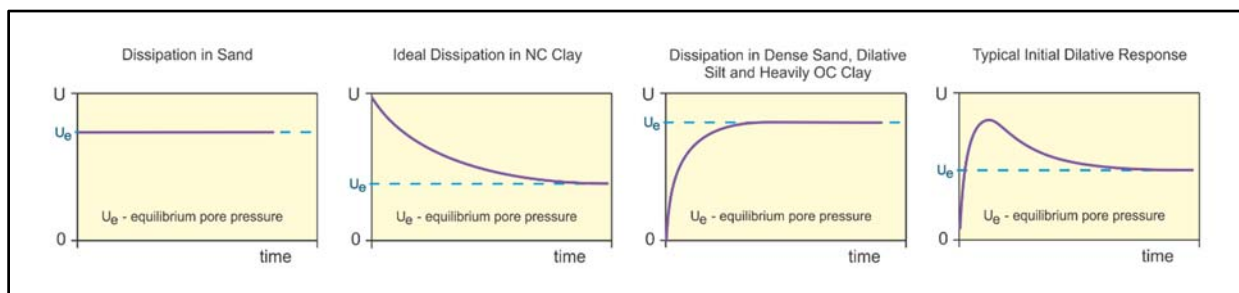


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure ( $u_{eq}$ ) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as  $t_{100}$ . In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to  $t_{100}$ . A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor ( $T^*$ ) may be used to calculate the coefficient of consolidation ( $c_h$ ) at various degrees of dissipation resulting in the expression for  $c_h$  shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

- $T^*$  is the dimensionless time factor (Table Time Factor)  
 $a$  is the radius of the cone  
 $I_r$  is the rigidity index  
 $t$  is the time at the degree of consolidation

Table Time Factor.  $T^*$  versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
$T^* (u_2)$	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time ( $t_{50}$ ) corresponding to a degree of dissipation of 50% ( $u_{50}$ ). In order to determine  $t_{50}$ , dissipation tests must be taken to a pressure less than  $u_{50}$ . The  $u_{50}$  value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as  $u_{100}$ . To estimate  $u_{50}$ , both the initial maximum pore pressure and  $u_{100}$  must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure ( $u$  at  $t_{100}$ ) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly ( $u_{100}$ ), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of  $c_h$  (Teh and Houlsby, 1991),  $t_{50}$  values are estimated from the corresponding pore pressure dissipation curve and a rigidity index ( $I_r$ ) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining  $t_{50}$ . In cases where the time to peak is excessive,  $t_{50}$  values are not calculated.

Due to possible inherent uncertainties in estimating  $I_r$ , the equilibrium pore pressure and the effect of an initial dilatory response on calculating  $t_{50}$ , other methods should be applied to confirm the results for  $c_h$ .

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

### References

Burns, S.E. and Mayne, P.W., 1998, "Monotonic and dilatatory pore pressure decay during piezocone tests", Canadian Geotechnical Journal 26 (4): 1063-1073.

Burns, S.E. and Mayne, P.W., 2002, "Analytical cavity expansion-critical state model cone dissipation in fine-grained soils", Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, "The piezometer probe: a useful investigation tool", Proceedings, 10<sup>th</sup> International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, "Estimating coefficient of consolidation from piezocone tests", Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, "An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils", Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Normalized Cone Penetration Test Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

# Cone Penetration Test Summary and Standard Cone Penetration Test Plots



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Start Date: 18-Feb-2015  
End Date: 18-Feb-2015

### ***CONE PENETRATION TEST SUMMARY***

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface <sup>1</sup> (m)	Final Depth (m)	Shear Wave Velocity Tests	Northing <sup>2</sup> (m)	Easting (m)
SCPT15-01	15-05005_SP01	18-Feb-2015	361:T1500F15U500	4.0	39.90	39	5024585	455661
SCPT15-02	15-05005_SP02	18-Feb-2015	361:T1500F15U500	4.0	47.95	46	5024679	455697
Totals	2 soundings				87.85	85		

1. Assumed phreatic surface depths were derived from pore pressure dissipation test data. Hydrostatic data were used for interpretation tables.
2. Coordinates are WGS 84 / UTM Zone 18 and were collected using MR350 GlobalSat GPS Receiver.



# Thurber Engineering

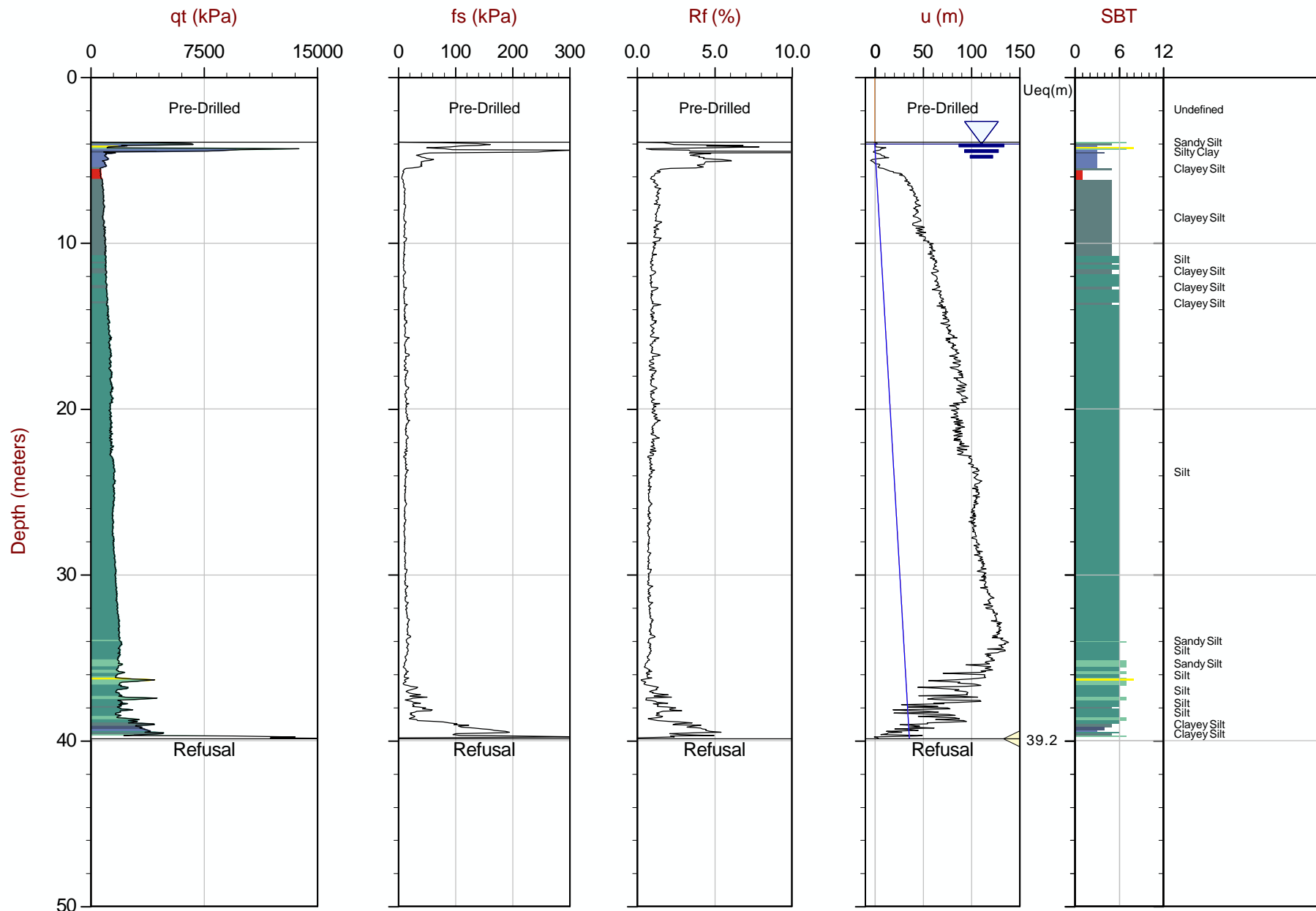
Job No: 15-05005

Date: 02:18:15 10:08

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-01

Cone: 361:T1500F15U500



Max Depth: 39.900 m / 130.90 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP01.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024585 E: 455661  
Page No: 1 of 1



# Thurber Engineering

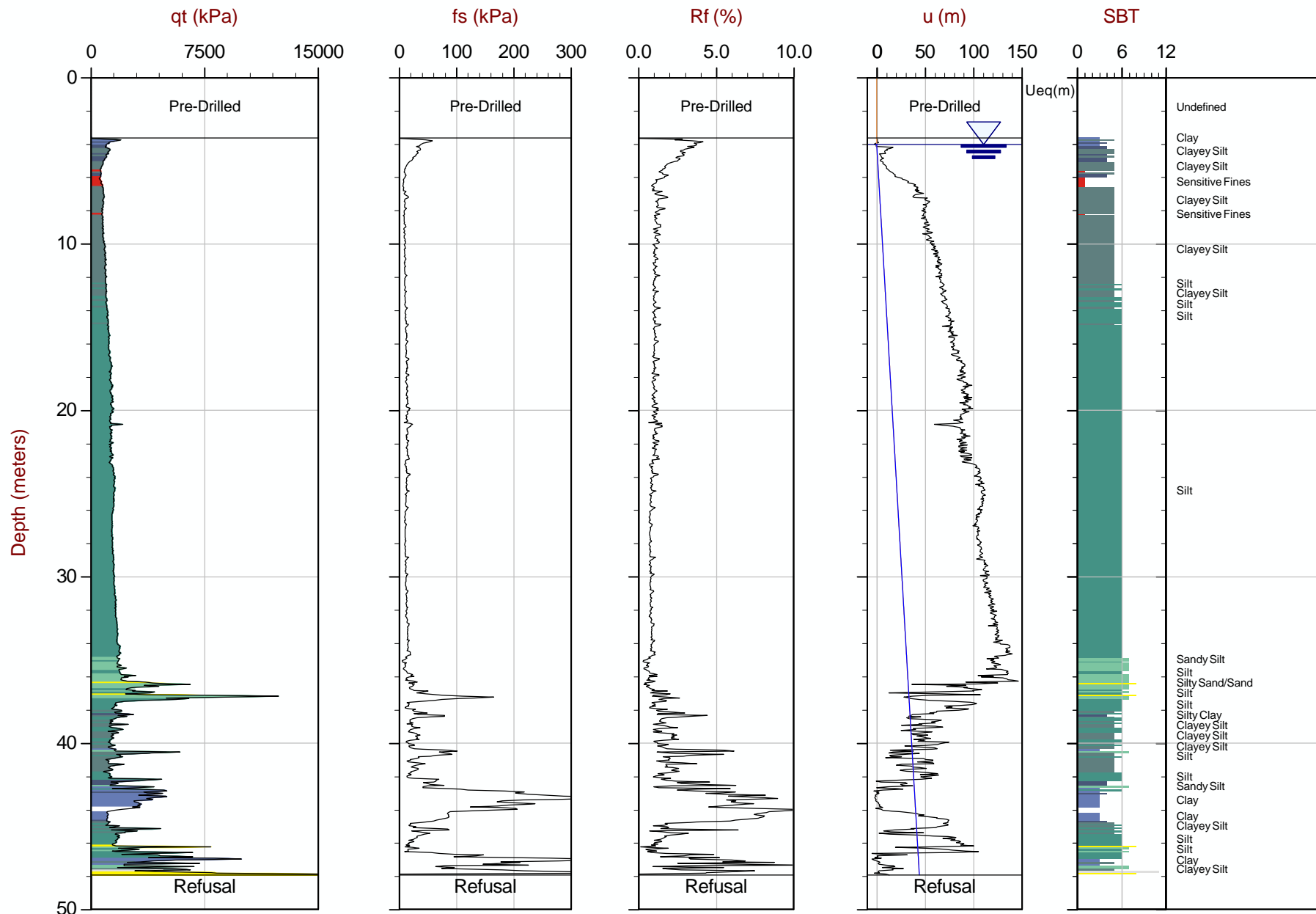
Job No: 15-05005

Date: 02:18:15 12:52

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-02

Cone: 361:T1500F15U500



Max Depth: 47.950 m / 157.31 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP02.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024679 E: 455697  
Page No: 1 of 1

## Normalized Cone Penetration Test Plots



# Thurber Engineering

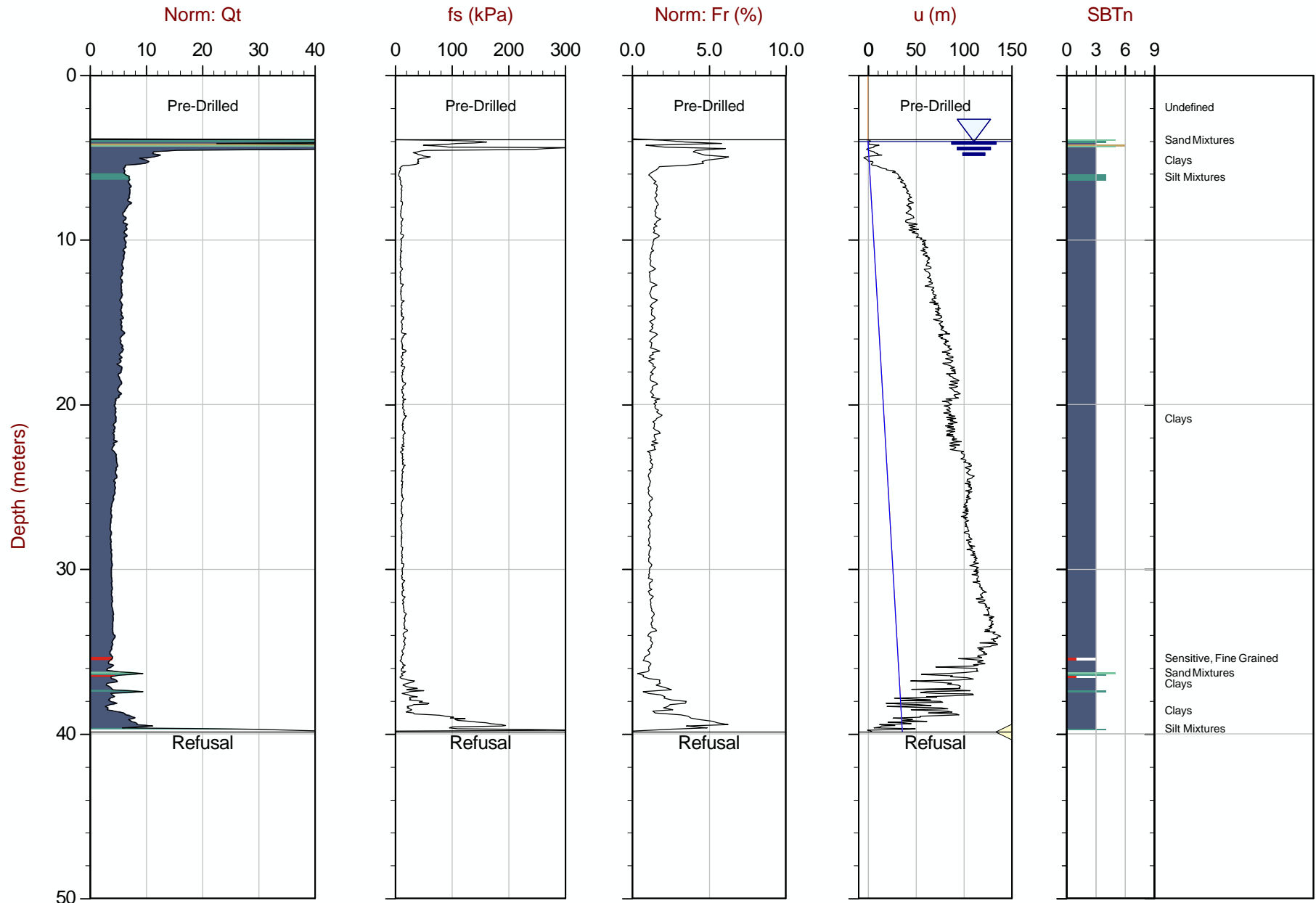
Job No: 15-05005

Date: 02:18:15 10:08

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-01

Cone: 361:T1500F15U500



Max Depth: 39.900 m / 130.90 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP01.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024585 E: 455661  
Page No: 1 of 1



# Thurber Engineering

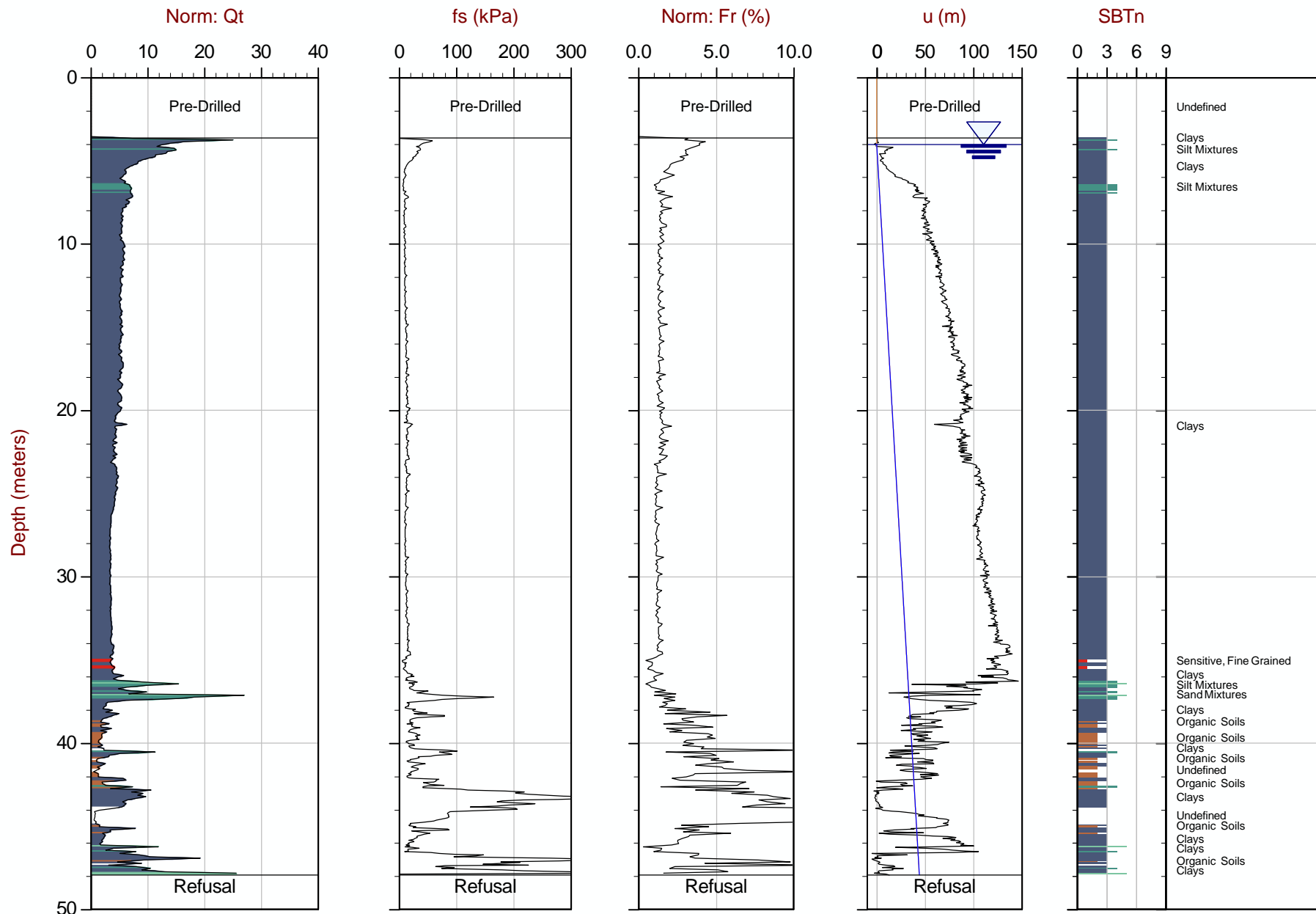
Job No: 15-05005

Date: 02:18:15 12:52

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-02

Cone: 361:T1500F15U500

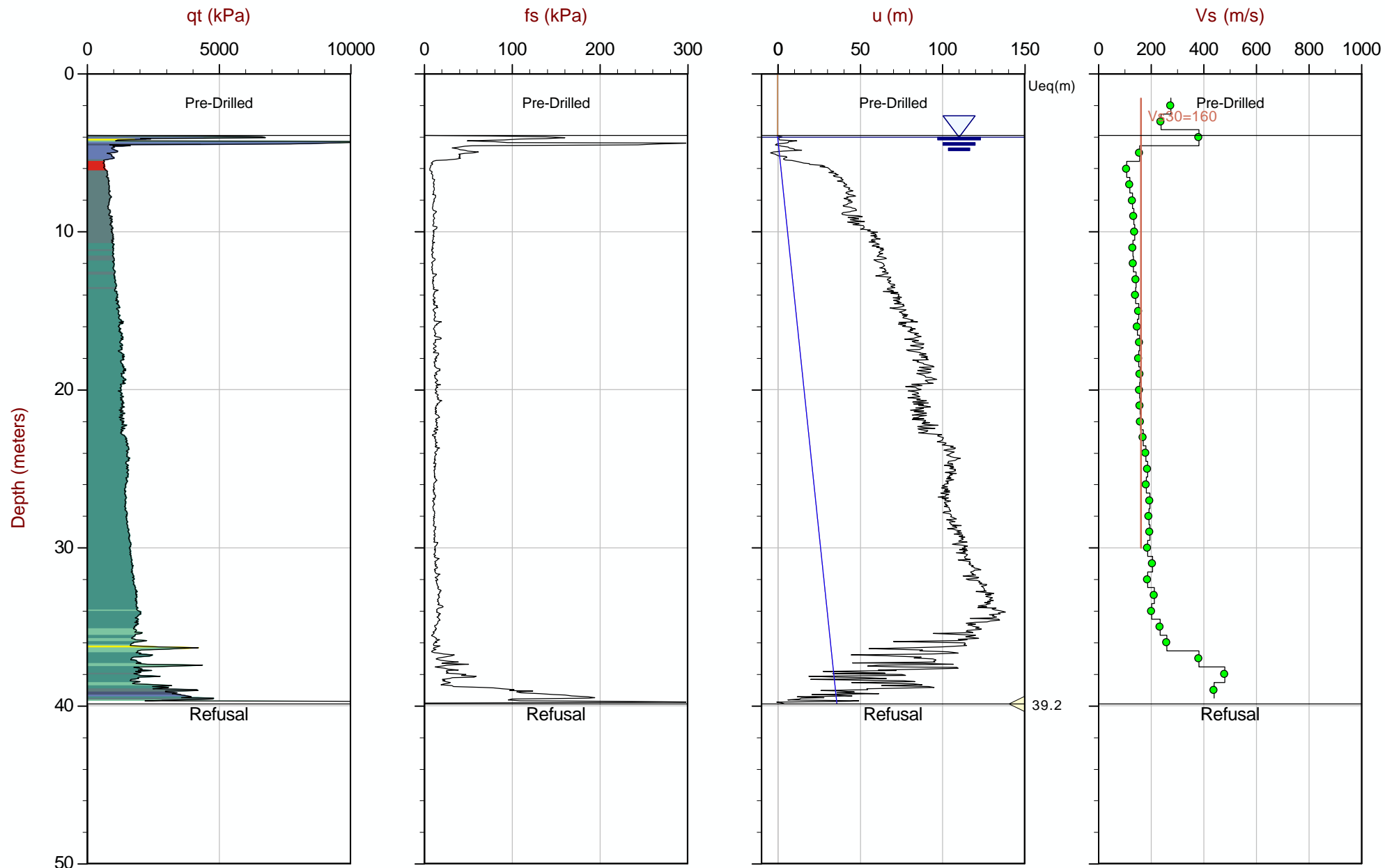


Max Depth: 47.950 m / 157.31 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP02.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024679 E: 455697  
Page No: 1 of 1

## Seismic Cone Penetration Test Plots



Max Depth: 39.900 m / 130.90 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP01.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024585 E: 455661  
Page No: 1 of 1



# Thurber Engineering

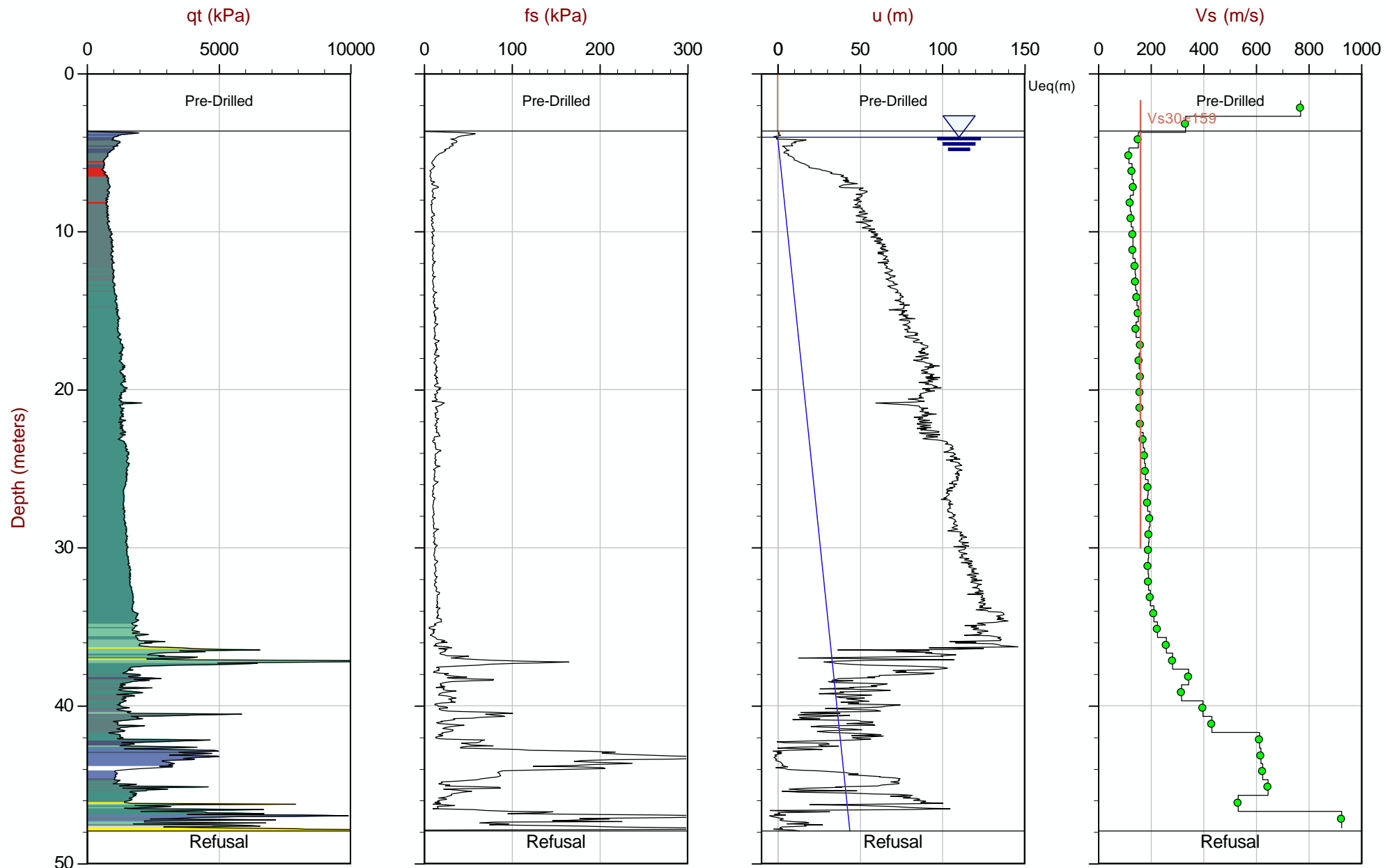
Job No: 15-05005

Date: 02:18:15 12:52

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-02

Cone: 361:T1500F15U500



Max Depth: 47.950 m / 157.31 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP02.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024679 E: 455697  
Page No: 1 of 1

## Seismic Cone Penetration Test Tabular Results



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Sounding ID: SCPT15-01  
Date: 18-Feb-2015

Seismic Source: Beam  
Source Offset (m): 0.60  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### **SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>**

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.75	1.55	1.66			
2.75	2.55	2.62	0.96	3.49	275
3.75	3.55	3.60	0.98	4.12	238
4.75	4.55	4.59	0.99	2.59	382
5.75	5.55	5.58	0.99	6.36	156
6.75	6.55	6.58	1.00	9.18	108
7.75	7.55	7.57	1.00	8.32	120
8.75	8.55	8.57	1.00	7.72	129
9.75	9.55	9.57	1.00	7.46	134
10.75	10.55	10.57	1.00	7.26	138
11.75	11.55	11.57	1.00	7.54	132
12.75	12.55	12.56	1.00	7.54	133
13.75	13.55	13.56	1.00	7.01	143
14.75	14.55	14.56	1.00	7.04	142
15.75	15.55	15.56	1.00	6.48	154
16.75	16.55	16.56	1.00	6.70	149
17.75	17.55	17.56	1.00	6.39	156
18.75	18.55	18.56	1.00	6.47	154
19.75	19.55	19.56	1.00	6.32	158
20.75	20.55	20.56	1.00	6.37	157
21.75	21.55	21.56	1.00	6.34	158
22.75	22.55	22.56	1.00	6.21	161
23.75	23.55	23.56	1.00	5.88	170
24.75	24.55	24.56	1.00	5.55	180
25.75	25.55	25.56	1.00	5.34	187
26.75	26.55	26.56	1.00	5.50	182
27.75	27.55	27.56	1.00	5.13	195
28.75	28.55	28.56	1.00	5.21	192
29.75	29.55	29.56	1.00	5.14	195



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Sounding ID: SCPT15-01  
Date: 18-Feb-2015

Seismic Source: Beam  
Source Offset (m): 0.60  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
30.75	30.55	30.56	1.00	5.31	188
31.75	31.55	31.56	1.00	4.87	206
32.75	32.55	32.56	1.00	5.36	187
33.75	33.55	33.56	1.00	4.69	213
34.75	34.55	34.56	1.00	4.96	202
35.75	35.55	35.56	1.00	4.28	234
36.75	36.55	36.55	1.00	3.85	260
37.75	37.55	37.55	1.00	2.62	382
38.75	38.55	38.55	1.00	2.08	480
39.75	39.55	39.55	1.00	2.27	440



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Sounding ID: SCPT15-02  
Date: 18-Feb-2015

Seismic Source: Beam  
Source Offset (m): 0.60  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### **SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>**

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.90	1.70	1.80			
2.90	2.70	2.77	0.96	1.25	768
3.90	3.70	3.75	0.98	2.96	332
4.90	4.70	4.74	0.99	6.50	152
5.90	5.70	5.73	0.99	8.57	116
6.90	6.70	6.73	1.00	7.76	128
7.90	7.70	7.72	1.00	7.47	133
8.90	8.70	8.72	1.00	8.14	122
9.90	9.70	9.72	1.00	7.97	125
10.90	10.70	10.72	1.00	7.64	131
11.90	11.70	11.72	1.00	7.62	131
12.90	12.70	12.71	1.00	7.11	140
13.90	13.70	13.71	1.00	7.07	141
14.90	14.70	14.71	1.00	6.79	147
15.90	15.70	15.71	1.00	6.63	151
16.90	16.70	16.71	1.00	6.94	144
17.90	17.70	17.71	1.00	6.23	160
18.90	18.70	18.71	1.00	6.44	155
19.90	19.70	19.71	1.00	6.19	161
20.90	20.70	20.71	1.00	6.28	159
21.90	21.70	21.71	1.00	6.30	159
22.90	22.70	22.71	1.00	6.22	161
23.90	23.70	23.71	1.00	5.84	171
24.90	24.70	24.71	1.00	5.73	175
25.90	25.70	25.71	1.00	5.57	179
26.90	26.70	26.71	1.00	5.28	189
27.90	27.70	27.71	1.00	5.36	187
28.90	28.70	28.71	1.00	5.14	195
29.90	29.70	29.71	1.00	5.18	193



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Sounding ID: SCPT15-02  
Date: 18-Feb-2015

Seismic Source: Beam  
Source Offset (m): 0.60  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### **SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>**

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
30.90	30.70	30.71	1.00	5.22	191
31.90	31.70	31.71	1.00	5.28	189
32.90	32.70	32.71	1.00	5.24	191
33.90	33.70	33.71	1.00	5.08	197
34.90	34.70	34.71	1.00	4.73	211
35.90	35.70	35.71	1.00	4.45	225
36.90	36.70	36.70	1.00	3.87	259
37.90	37.70	37.70	1.00	3.53	283
38.90	38.70	38.70	1.00	2.91	344
39.90	39.70	39.70	1.00	3.16	316
40.90	40.70	40.70	1.00	2.52	398
41.90	41.70	41.70	1.00	2.32	431
42.90	42.70	42.70	1.00	1.63	613
43.90	43.70	43.70	1.00	1.62	617
44.90	44.70	44.70	1.00	1.60	624
45.90	45.70	45.70	1.00	1.55	645
46.90	46.70	46.70	1.00	1.88	532
47.95	47.75	47.75	1.05	1.14	925

Pore Pressure Dissipation Summary and  
Pore Pressure Dissipation Plots



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Start Date: 18-Feb-2015  
End Date: 18-Feb-2015

### ***CPT<sub>u</sub> PORE PRESSURE DISSIPATION SUMMARY***

Sounding ID	File Name	Cone Area (cm <sup>2</sup> )	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U <sub>eq</sub> (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)
SCPT15-01	15-05005_SP01	15	305	40	39.22	0.7	4.0
Totals	1 Dissipations		5.1 min				



*Thurber Engineering*

Job No: 15-05005

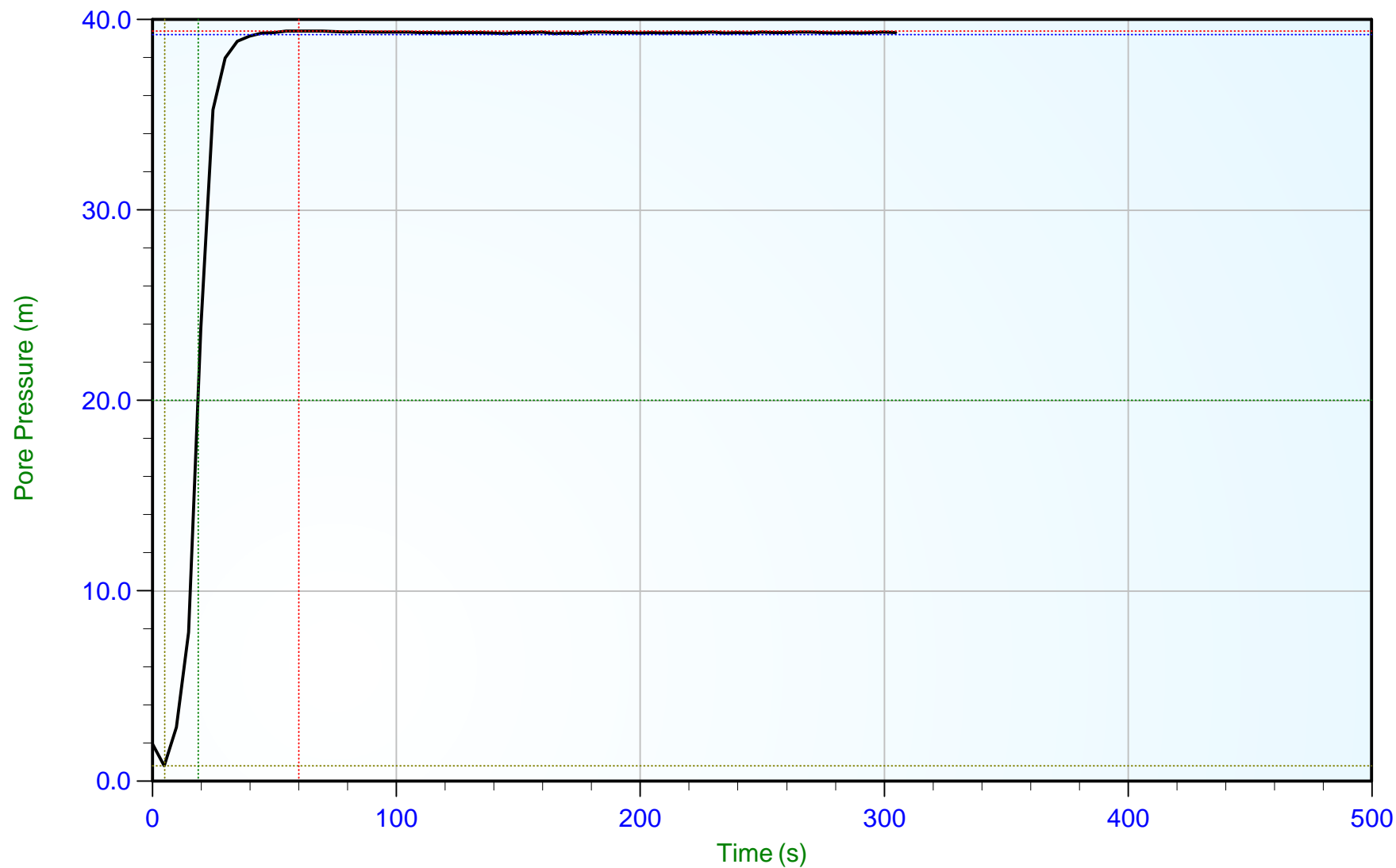
Date: 18-Feb-2015 10:08:56

Site: Hwy 417 & Ramsayville Rd

Sounding: SCPT15-01

Cone: AD361

Cone Area: 15 sq cm



Trace Summary: Filename: 15-05005\_SP01.PPD U Min: 0.8 m WT: 0.676 m / 2.217 ft  
Depth: 39.900 m / 130.904 ft U Max: 39.4 m Ueq: 39.2 m  
Duration: 305.0 s



**APPENDIX 15**  
**SITE 3-265/2**



## **MEMORANDUM**

To: Laura Donaldson, P.Eng.  
McIntosh Perry Consulting Engineers

Date: August 21, 2015

From: Fred J. Griffiths, Ph.D., P.Eng.  
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-3405-3

### **PRELIMINARY FOUNDATION DESIGN HIGHWAY 417 WESTBOUND RAMSAYVILLE ROAD OVERPASS (SITE 3-265/2) GWP 4074-11-00 GEOCRES 31G5-263**

## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing westbound Highway 417 Overpass of Ramsayville Road in Ottawa, Ontario. As part of the preliminary investigation seismic piezocone penetration testing was carried out to confirm the appropriate seismic soil profile for this site. The memo also presents preliminary geotechnical recommendations for use in an assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating the options and will need to be refined during subsequent design stages.

The following reference numbers apply to this site:

- Current W.P. 4145-10-01
- Site No. 3-265.2
- GEOCRES No. 31G5-190
- Construction Contract 73-190
- Historic W.P. 10-69-07

### **2 SITE DESCRIPTION**

The site is located in eastern Ottawa in the Township of Gloucester approximately 2 km west of Anderson Road. The bridge carries the Highway 417 westbound lanes (two lanes plus paved shoulders) over Ramsayville Road. Based on the historic General Layout Drawing (copy attached), the bridge is a 13 m wide, 94 m long, five-span prestressed concrete structure. The two bridge abutments and four piers are supported by steel HP12x74 piles driven to bedrock.

The natural terrain in the vicinity of the bridge includes the Ramsay Creek valley which is about 8 m deep with a flat floor about 37 m wide; the natural valley walls have a slope of about



3H:1V (Horizontal:Vertical). At the time of Thurber's site visit the creek was flowing south to north through the valley in an incised channel approximately 1.5 m deep and 4 m wide with nearly vertical walls. The water level in the creek was about 0.3 m at the time the site inspection. The approach fills were to be constructed by placing as much as 3.7 m of fill with a side slope of 2H:1V.

Ramsayville Road also passes beneath the structure in a north-south direction. It includes two 3.7 m lanes and 2.4 m wide gravel shoulders, constructed on a fill with 2H:1V side slopes.

### **3 SUBSURFACE CONDITIONS**

A site investigation was carried out by the MTO Foundations Office; GEOCRE Report No. 31G5-190 dated May 1972. The investigation consisted of five boreholes, all accompanied by dynamic cone penetration tests. Before construction of the westbound lanes, the alignment was changed and an additional soils investigation was carried out in April and May of 1972 that consisted of five additional boreholes. Drawing No. 72-11052A (copy attached) illustrates the locations of the bridge, the investigation boreholes, as well as the soil strata plot for the investigation. The stratigraphy in the area of the bridge is generally characterized by a thin clayey silt layer, overlaying a sensitive marine clay layer, overlaying glacial till, underlain by shale bedrock.

#### **3.1 Clayey Silt**

The top of the clayey silt layer ranged from 68.8 m to 69.9 m in elevation, and the layer had a thickness of 1.5 m to 2.7 m. This deposit was encountered exclusively in boreholes advanced on the valley floor.

#### **3.2 Clay**

A clay stratum was encountered in all boreholes advanced at the site. The surface of this deposit ranged from 76.4 to 67.0 m in elevation, and the layer had a thickness of 30.8 to 38.6 m. Atterberg Limits test results indicate a liquid limit from 48% to 82%, and a plastic limit from 23% to 32%. The moisture content of the samples tested ranged from 34% to 87%. The shear strength of the clay based on the in-situ field vane tests ranged from 100 kPa at an elevation of 73 m, decreasing with depth to 30 kPa at an elevation of 70 m, increasing with depth to 70 kPa at an elevation of 55 m and then remains relatively constant at 70 kPa to the base of the unit. Results of the shear strength testing indicate a very stiff consistency decreasing to firm with depth, with a sensitivity between 2 and 11.

Consolidation testing was carried out on two samples acquired in Borehole 2 which was located in the valley and indicated that the deposit is slightly over-consolidated with the pre-consolidation stress about 100 kPa above existing stresses.

#### **3.3 Glacial Till**

Underlying the clay stratum is a glacial till deposit. The surface of this deposit ranged from 35.0 to 37.8 m in elevation, and the layer had a thickness of 11.1 to 13.6 m. The standard penetration 'N' values varied greatly for this deposit ranging from 9 per 0.3 m of penetration to 150 blows per



0.1 m of penetration, indicating a loose to very dense condition. Gradation test results indicate a gravel content from 5% and 37%, sand content from 33% and 63%, silt content between 13% and 36%, and clay content of 3% and 21%.

### **3.4 Bedrock**

Beneath the glacial till layer, shale bedrock was encountered with surface elevations ranging from 23.0 to 24.3 m. The bedrock was described to be in sound condition. Geological mapping suggests the bedrock at this site is of the Carlsbad Formation.

### **3.5 Groundwater**

Groundwater levels were measured in the open boreholes prior to backfilling at elevations ranging from 73.2 m to 74.6 m along the banks of the valley and at an elevation of 65.1 m to 69.7 m along the valley floor. Artesian conditions were noted in four boreholes drilled in the valley with a pressure head rising to elevations ranging from 69.6 m to 72.8 m.

## **4 SITE OBSERVATIONS**

A structure inspection was conducted by MTO in March 2012 and again in June 2012 for Bridge 3-265/2 with the report issued December 2012. Condition data outlined in the report for the bridge structure ranged from poor to good but typically the bridge was rated in fair to good condition. The report recommended major rehabilitation of the bridge structure components within 1 to 5 years of the inspection.

The site was inspected by Thurber Engineering staff during the week of July 14<sup>th</sup>, 2014. Several photographs of the site are attached.

At the time of the inspection, the following observations were made:

- No evidence of slope stability issues were noted on the well vegetated side slopes of the approach fills
- The foreslopes of both abutments were covered by riprap but some minor erosion undermining the riprap was noted on the north side of the east abutment foreslope
- No settlement issues were observed in the approach embankments and the ride across the transition from deck to approaches was relatively smooth
- A transverse crack was noted in the asphalt at the west abutment approach embankment and slab transition



## **PART 2: ENGINEERING DISCUSSION AND PRELIMINARY RECOMMENDATIONS**

### **5 GEOTECHNICAL ASSESSMENT**

#### **5.1 Frost Considerations**

The frost penetration depth at this site is 1.8 m (OPSD 3090.101).

#### **5.2 Seismic Considerations**

##### **5.2.1 Seismic Piezocone Penetration Testing**

Based on the review of available historical data, the existing soil stratigraphy at Site 3-265/2 would be at or near the threshold to be characterized as either a Seismic Soil Profile Type III or Type IV as defined in the Canadian Highway Bridge Design Code (CHBDC–CAN/CSA-S6-06). Seismic piezocone penetration testing (SCPTu) was carried out to determine the in-situ shear wave velocity profile in order to confirm the appropriate Seismic Soil Profile for this site.

Thurber engaged ConeTec Investigations Limited (ConeTec) to carry out the SCPTu investigation for this assignment. The field investigation included advancing a SCPTu sounding, designated SCPT15-02, within the right-of-way of Ramsayville Road. The locations and ground surface elevations of the SCPTu sounding are summarized in Table A and are also illustrated on the SCPTu Location Plan (copy attached).

**Table A: SCPTu Testing Summary**

<b>Sounding ID</b>	<b>Location</b>	<b>Ground Surface Elevation (m)</b>	<b>Depth (m)</b>	<b>Number of Shear Wave Velocity Tests</b>
SCPT15-02	West shoulder of Ramsayville Road 20 m north of WB bridge	72.8	47.9	46

A velocity profile for the soil stratigraphy was developed by carrying out shear wave velocity tests at one meter intervals. A description of the equipment and testing methodology used for this investigation is provided in ConeTec's Report No. 15-05005 (copy attached).

##### **5.2.2 Shear Wave Velocity Test Results and Interpretation**

The shear wave velocity test results are tabulated in ConeTec's report for Sounding SCPT15-02.

The current CHBDC (Section 4.4.6.5) indicates that Soil Profile Type IV is a profile with soft clays or silts greater than 12 m in thickness and that these materials are characterized by a shear wave velocity less than 150 m/s.



The results of the shear wave velocity testing indicate between 10 and 11 m of soil profile with a shear wave velocity of less than 150 m/s and an average shear wave velocity within the upper 30 m of 159 m/s in SCPT15-02.

Based on the shear wave velocity data the site are classified as Soil Profile Type III under the current CHBDC.

### **5.2.3 Liquefaction**

The zonal acceleration ratio for this site is 0.20 as per Table A3.1.1 of the CHBDC.

Based on the plasticity of the clay at this site, it is classified as “not susceptible” to liquefaction during an earthquake event.

### **5.3 Existing Foundations**

As per the Foundation Layout and Reinforcement (copy attached) the abutments and piers of the existing structure are supported by HP12x74 steel piles driven to bedrock. The contract drawings do not indicate the design loads or grade of steel for the piles, however the Foundation Design Report indicates that for end-bearing piles driven to bedrock the recommended safe design load for foundations is 95 tons / HP12x74 pile or approximately 845 kN/pile.

12BP74 piles are nominally equivalent in dimension and mass to HP310x110 piles. The current MTO practice allows for a vertical geotechnical resistance at ULS of 2000 kN / HP310x110 pile driven to bedrock. The SLS condition will not govern for piles end-bearing in or on bedrock. OPSS 903 requires the use of Grade 350W steel for H-piles.

## **6 GEOTECHNICAL RECOMMENDATIONS**

Based on the available data regarding the ground conditions at this site, the following sections present preliminary geotechnical recommendations for the assessment of the existing structure and for preliminary design for potential new structures or modifications, if required. It is noted that the proposed construction options for this site are not yet defined. The preliminary recommendations provided below are for assistance in developing and evaluating options and will need to be refined during subsequent design stages.

### **6.1 Shallow Foundations**

The very stiff to firm clay deposit is not considered suitable to carry a bridge structure. The depth of the top of the till deposit was observed to range in excess of 30 m below original grade which is considered too deep for spread footing foundations. As such spread footings within the overburden are not recommended and deep foundations are preferred at this site.



## **6.2 Deep Foundations – Piles**

Based on the review of existing subsurface stratigraphy at the site driven piles are considered to be suitable for the support of abutment and pier foundations. It should be noted that the bedrock surface elevation ranges from 23.0 to 24.3 m.

### **6.2.1 Axial Resistance**

Due to the anticipated length of the piles (over 40 m) and thickness of the glacial till layer beneath the sensitive clay, the piles may reach practical refusal in the lower part of the glacial till layer. Therefore, it is recommended that the design use steel HP section piles driven to practical refusal.

The design parameters for axial resistance of both HP310x110 and HP310x132 piles driven to practical refusal within the glacial till deposit or upper shale bedrock can be taken as:

- 1,800 kN factored geotechnical resistance at ULS; and
- 1,600 kN axial resistance at SLS

It is noted that the piles will penetrate through the deep clay deposit and into or through the glacial till deposit where artesian groundwater conditions have been observed. Due to the thickness of the clay, artesian flow up the pile shaft is not expected to be a concern. Furthermore, it is noted that the existing bridge structures are supported on steel H-Piles driven to similar depths and no problems with artesian flow up the pile shafts were noted in the review of the construction history.

### **6.2.2 Pile Tips**

New piles will be driven through a thick glacial till deposit which may contain cobbles and/or boulders, as such the pile tips should be protected from damage during driving.

### **6.2.3 Integral Abutment Considerations**

The Foundation Layout and Reinforcement Drawing indicates that the existing abutments are supported by pile groups that include battered piles. Integral abutments are therefore not feasible for widening of the existing structure.

If replacement of the structure is required, then the ground conditions within the approach fills at the site are generally suitable for integral abutments, though to maintain flexibility, the upper 3 m of the pile should be encased in a sand filled CSP sleeve.

### **6.2.4 Pile Spacing**

Since new piles at this site will be end bearing on a hard layer, the vertical resistance will not be significantly affected by the pile spacing. However, it is recommended that a minimum centre-to-centre spacing of 750 mm be maintained, as provided in the CHBDC.



### 6.2.5 Downdrag

Downdrag forces need to be considered for piles in areas where grades have increased from original. This includes the piles supporting the existing abutments and Piers 1 and 2. It is anticipated that the design of new piles required to support a widened foundation at both the abutments and Piers 1 and 2 will also need to include downdrag loads due to approach fill widening. Downdrag loads for piles at Piers 3 and 4 are not anticipated unless grades are modified at these pier locations.

The following pile SLS dead loads were provided by the structural design team in order to determine the downdrag forces acting on Structure 3-265/2:

- Abutment pile loading = 153 kN / pile
- Pier pile loading = 320 kN / pile

The value of downdrag on existing and new piles should be considered for this site as follows:

Existing piles:

- The abutment piles are being subject to unfactored downdrag loads of approximately 500 kN/pile due to the original placement of as much as 4.6 m of fill at the approaches. Placement of additional fill for embankment widening or grade raises will not add further downdrag load to these piles.
- The piles supporting Piers 1 and 2 are being subject to unfactored downdrag loads of approximately 250 kN/pile due to the construction of Ramsayville Road.
- Placement of additional fill for embankment widening or grade raises will not add further downdrag load to these piles.

New Piles:

If there are no modifications to the width or height of the approach fills or in the areas of the piers, new piles at the site will not be subject to downdrag loads. Should modifications be required, assuming the same SLS dead loads listed above and that any additional approach fills will match existing grades; new piles at Bridge Structure 3-265/2 would be subject to the following downdrag loads:

- New abutment piles will be subject to downdrag loads of approximately 500 kN/pile due to the placement of fill to address widening and/or grade raises at the approaches.
- New piles at Piers 1 and 2 will not be subject to downdrag loads unless modifications to Ramsayville Road are made and/or the grades are modified at these pier locations.

The downdrag load should be multiplied by a load factor of 1.25 as per the CHBDC Commentary Clause C6.8.4 to obtain a factored downdrag load. In accordance with Section 6.8.4 of the CHBDC and clause C6.8.4 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag loads should not exceed the factored structural



resistance of a pile. In geotechnical analysis of downdrag, live load effects should not be considered.

### 6.2.6 Lateral Resistance of Piles

The lateral resistance of both existing and new driven piles can be assessed based on the method outlined in the CHBDC. For a driven H-pile, the lateral soil resistance may be calculated using the following formulae:

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

For cohesive soils:

$$k_h = \frac{67S_u}{B}$$

where:

- $k_h$  = coefficient of horizontal subgrade reaction
- $n_h$  = coefficient related to soil density given in Table B
- $z$  = depth of pile embedment (m)
- $B$  = pile width perpendicular to load direction (m)
- $S_u$  = undrained shear strength given in Table B

**Table B:**  $n_h$  values for cohesionless soils and  $S_u$  values for cohesive soils.

Elevation (m)	Soil Description	$n_h$ (kN/m <sup>3</sup> )	$S_u$ (kPa)
Above 76.0	Embankment Fill	3000	-
Between 76.0 and 73.0	Native Clay	-	100
Between 73.0 and 70.0	Native Clay	-	100 to 30
Between 70.0 and 55.0	Native Clay	-	30 to 70
Between 55.0 and 36.0	Native Clay	-	70
Below 36.0	Glacial Till	2000	-

The coefficient of horizontal subgrade reaction may have to be reduced, based on the pile spacing, to account for pile group effects. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in the Table C. Intermediate values may be obtained by linear interpolation.

**Table C:** Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre*	Horizontal Subgrade Reaction Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

(\*) where D is the width of pile

### 6.2.7 Uplift Resistance

Due to the thickness of the clay and till deposits at this site and the fact that uplift resistance is developed through shaft resistance it is not anticipated that uplift resistance will be an issue.

The unfactored uplift resistance of new or existing piles can be calculated using the following formula:

$$R = \sum_{z=0}^L C q_s \Delta z + W_p$$

where:

C = pile circumference (m)

$q_s$  = soil unit shaft friction

$\Delta z$  = subdivided segment of the embedded length (m)

$W_p$  = pile weight (kN)

The shaft friction can be calculated for each soil layer using the following formula:

$$q_s = \beta \sigma'_v$$

where:

$\beta$  = combined shaft resistance coefficient given in Table D

$\sigma'_v$  = effective vertical stress (kPa)

**Table D:**  $\beta$  values for driven piles

Elevation (m)	Soil Description	Unit Weight, $\gamma$ , (kN/m <sup>3</sup> )	$\beta$
Above 76.0	Embankment Fill	20	0.4
Between 76.0 and 36.0	Native Clay	17	0.25
Below 36.0	Glacial Till	19	0.4



A geotechnical resistance factor of 0.3 should be applied to the unfactored uplift resistance as recommended in Table 6.1 of the CHBDC.

### 6.3 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table E.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with the Section 6.9.3 of the CHBDC. Surcharge loading from traffic can be ignored if the approach slabs are supported by the abutments (CHBDC Section 6.9.5).

The parameters provided in Table E are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for hydrostatic pressures should be considered.

**Table E:** Static Lateral Earth Pressure Coefficients

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Clay
Soil Unit Weight, kN/m <sup>3</sup> , $\gamma$	21.0	20.0	17
Angle of Internal Friction, $\phi$	35°	30°	27°
Coefficient of at Rest Earth Pressure, $K_0$ (Restrained Wall)	0.43	0.50	0.55
Coefficient of Active Earth Pressure, $K_a$ (Unrestrained Wall)	0.27	0.33	0.38

## 6.4 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section 4.6.4 of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with  $k_h = A/2$  for structures that allow lateral yielding and  $k_h = 3/2A$  for non-yielding walls, where  $A$  is the zonal acceleration ratio. An outward displacement of  $250A$  (i.e. 50 mm) would be required for the wall to be considered a yielding structure.

The recommended unfactored seismic lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table F.

**Table F:** Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B Type I	Native Clay
Soil Unit Weight, $\text{kN/m}^3$ , $\gamma$	21.0	20.0	17
Angle of Internal Friction, $\phi$	$35^\circ$	$30^\circ$	$27^\circ$
<b>Yielding Wall</b>			
$K_{AE}$	0.33	0.4	0.45
$K_{AE}$ Load application height from base as a ratio of wall height	0.37	0.36	0.36
<b>Non-Yielding Wall</b>			
$K_{AE}$	0.55	0.66	0.74
$K_{AE}$ Load application height from base as a ratio of wall height	0.44	0.43	0.43

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a) \gamma (H - d)$$

where:

$\sigma_h$  = lateral earth pressure at depth,  $d$  (kPa)

$d$  = depth below the top of the wall (m)

$K_a$  = static active earth pressure coefficient

$\gamma$  = unit weight of the backfill soil ( $\text{kN/m}^3$ )

$K_{AE}$  = combined static and seismic earth pressure coefficient

$H$  = total height of the wall (m)

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. OPSS Granular A or B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. OPSS Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.



The factors provided in Table F are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC.

## **6.5 Approach Embankments**

Based on the Foundation Design Report, the embankment soils consist of compact to dense deposits of non-cohesive soil to a maximum height of 3.7 m above the original ground surface. An embankment slope of 2H:1V was recommended to be used. Settlement of up to 125 mm was predicted with 50% of the consolidation occurring within 12 to 18 months of fill placement. The embankment foundation is expected to be stable and no further consolidation settlement is expected unless the fills are reconfigured.

It is anticipated that settlement of the native clay will occur should the embankments be widened or if additional lanes are added to Highway 417. Further settlement of the existing embankment and roadway may also occur due to the increase in stress caused by the project works.

## **6.6 Erosion Control**

The erosion protection measures at the site should be repaired and maintained to prevent erosion of the abutment slopes.

## **6.7 Excavations and Backfilling**

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The existing embankment fill and native clay at this site should be classified as Type 3 in accordance with the OHSA.

Where excavations will extend below the groundwater level prevailing at the time of construction, the contractor will need to implement groundwater control and ground support systems as are required to carry out the construction in a safe, stable, and unwatered excavation.

Excavations and backfill for rigid structures should be in accordance with OPSS 902. Backfill for the abutment must consist of free draining granular material conforming to OPSS Granular A or B material specifications and must be placed to the extent shown on OPSD 3101.150.

Subgrade preparation and placement of the foundations must be carried out in the dry.

## **7 ADDITIONAL INVESTIGATIONS**

Further foundation investigations and analysis will be warranted should the structure be replaced or widened or if the height, geometry, or location of the approach fills are to be modified for either short-term construction staging or for vertical or horizontal realignments. Should excavations be anticipated within the approach fills to repair or modify the abutments, expansion joints, or approach slabs, consideration should be given to drilling foundation boreholes in the approaches to determine the nature of the fill materials and to allow generation of roadway protection design. It is also recommended that piezometers be installed to better define the groundwater level as

well as the hydraulic conductivity at the site. This information will be required to allow for the design of excavation dewatering systems.

During detailed design, the need for vibration monitoring will need to be assessed should options include structure widening or a new structure adjacent to the existing.

## 8 CLOSURE

We trust this information provided in this technical memorandum meets your present purposes. Please let us know if you have any questions or need additional information.

Thurber Engineering Ltd.

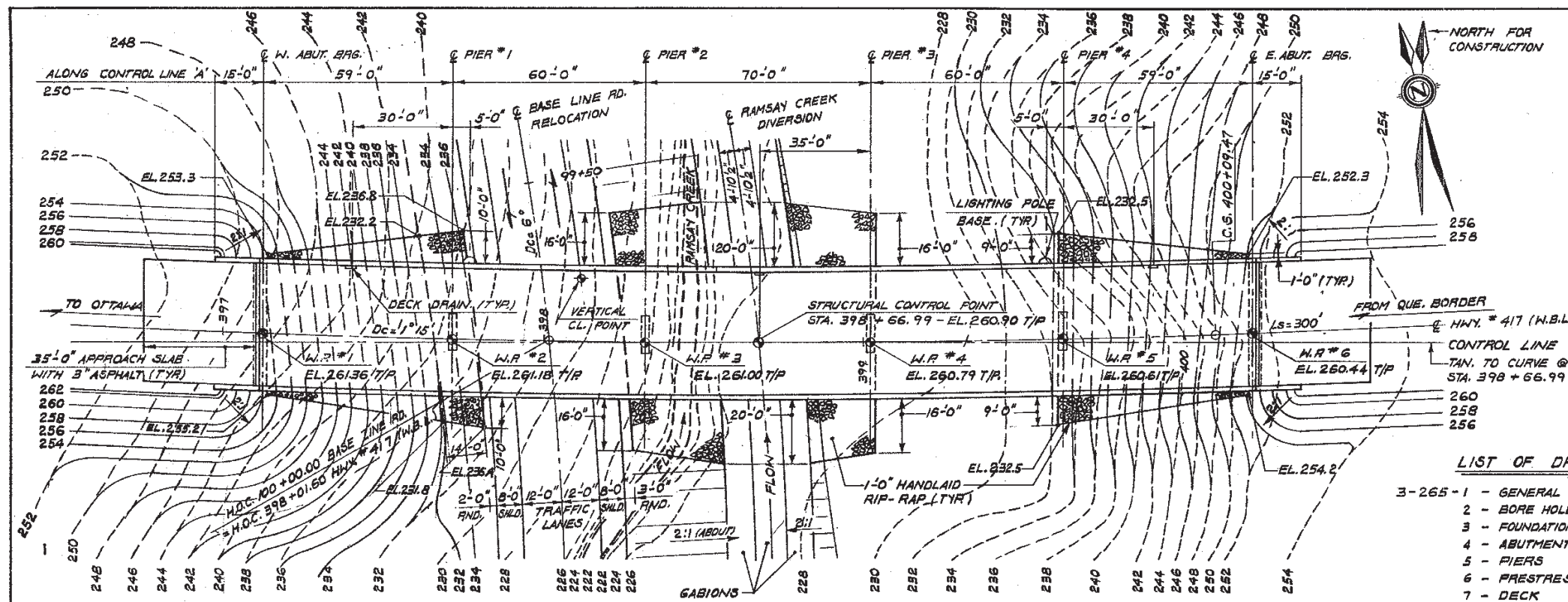


Fred J. Griffiths, P.Eng.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact

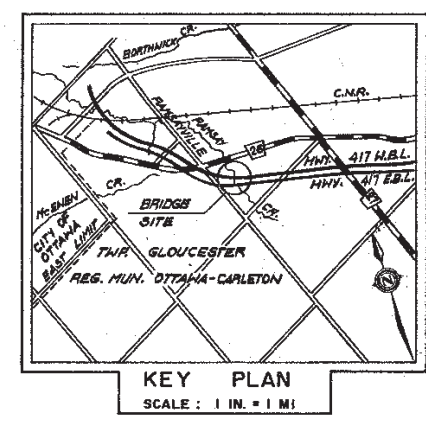
Attachments



**NOTE**  
• W.P. DENOTES WORKING POINT  
• T/P DENOTES TOP OF PAVEMENT

**CURVE DATA**  
HWY. 417 - W.B.L.

Δ	= 34° 56' 50"
ΔC	= 31° 11' 50"
Ts	= 1593.16'
Lc	= 2495.78'
Es	= 222.60'
Ls	= 300.00'
Θs	= 1° 52' 30"

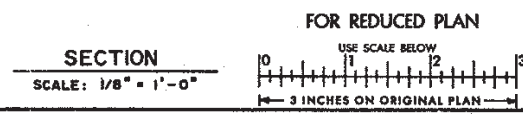
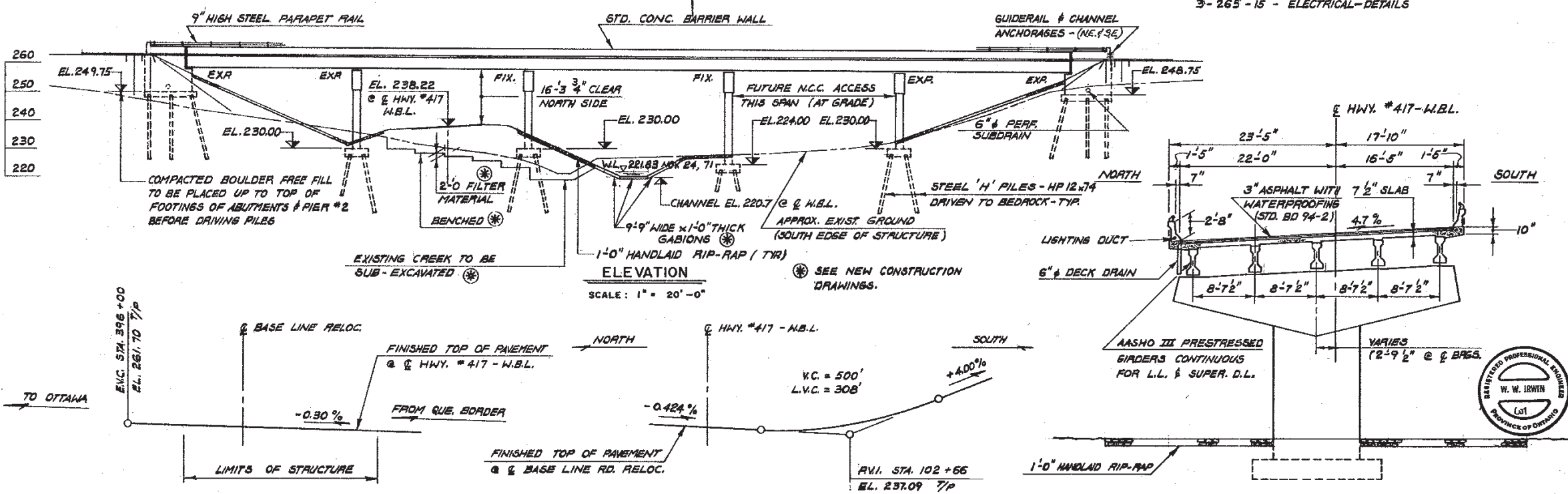


- LIST OF DRAWINGS**
- 3-265-1 - GENERAL LAYOUT
  - 2 - BORE HOLE LOCATION & SOIL STRATA
  - 3 - FOUNDATION LAYOUT & REINFORCEMENT
  - 4 - ABUTMENTS
  - 5 - PIERS
  - 6 - PRESTRESSED GIRDERS & BEARINGS
  - 7 - DECK
  - 8 - CONCRETE BARRIER WALL (2'-8" HIGH)
  - 9 - DETAILS OF 9" HIGH STEEL PARAPET RAILING
  - 10 - 35 FT. APPROACH SLAB FOR BARRIER WALL
  - 11 - STANDARD DETAILS - I
  - 12 - STANDARD DETAILS - II
  - 13 - ELECTRICAL-PLAN & TYPICAL SECTION
  - 14 - ELECTRICAL-DETAILS
  - 3-265-15 - ELECTRICAL-DETAILS

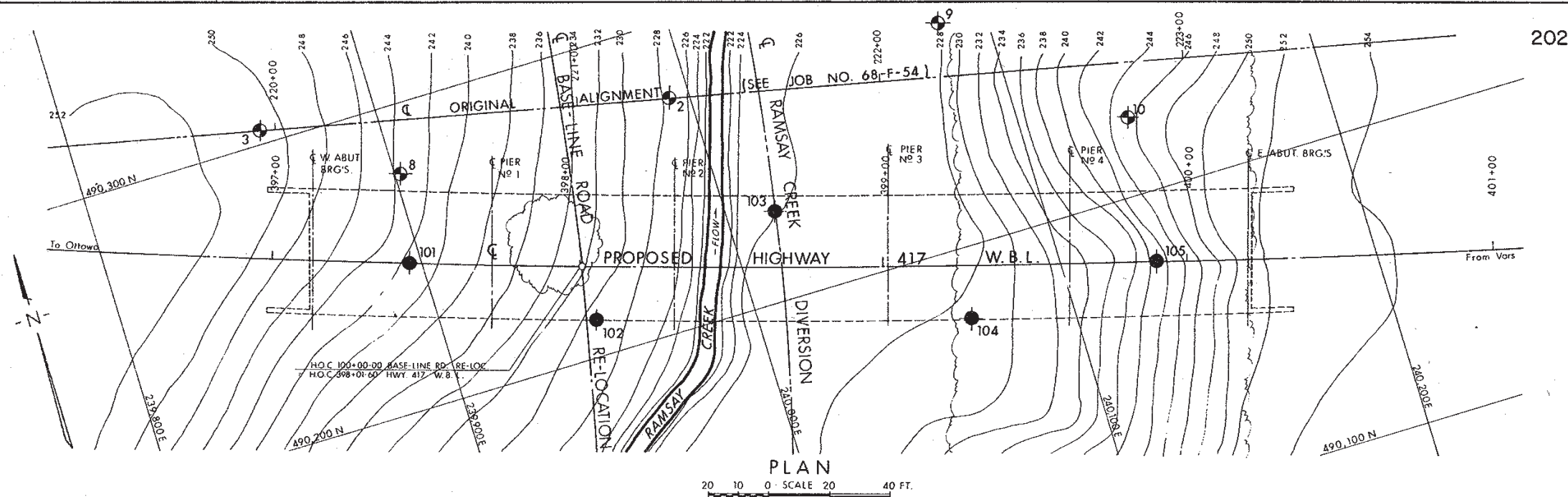
**REFERENCE BENCH MARK**  
B.M. 260.53  
GEODETIC DATUM  
CUT CROSS ON CONC. WALK AT N.E. CORNER OF BRIDGE  
256.0' RT. OF 396 + 38 (W.B.L.)

**NOTES:**  
**CLASS OF CONCRETE**  
DECK, BARRIER WALLS & APPROACH SLABS — 4,000 P.S.I.  
PRESTRESSED GIRDERS — 5,000 P.S.I.  
REMAINDER — 3,000 P.S.I.

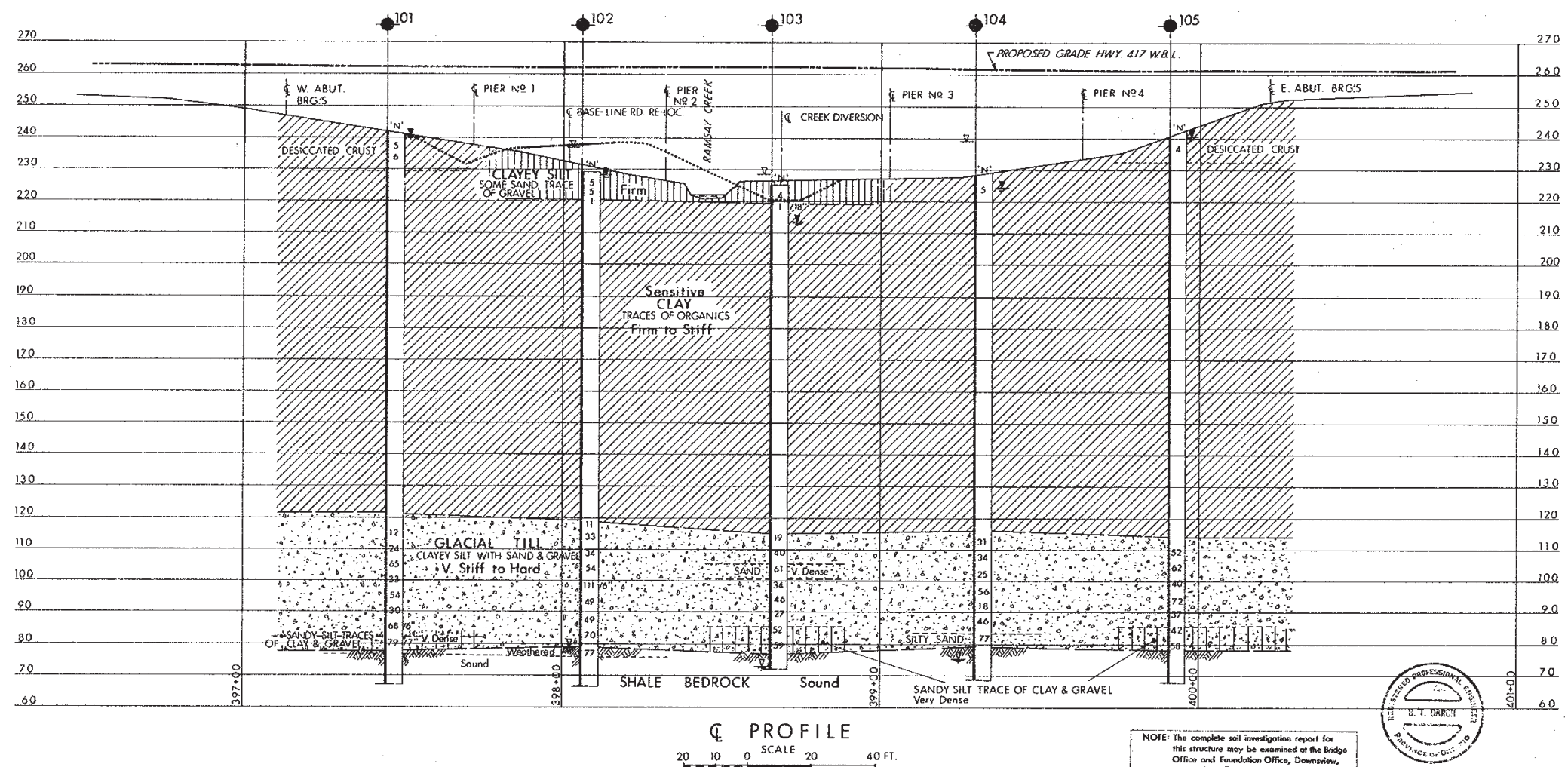
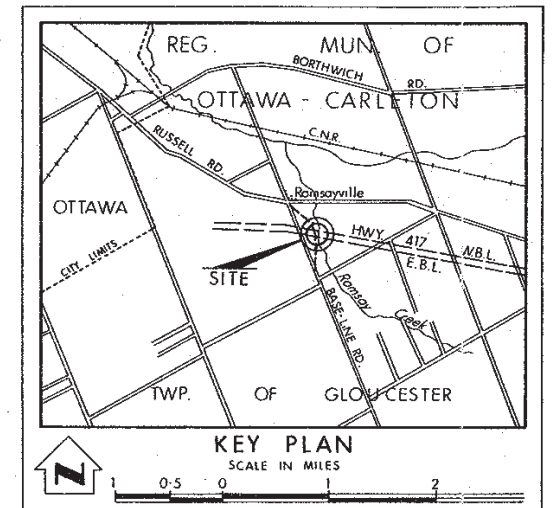
**CLEAR COVER ON REINF. STEEL**  
FOOTINGS, ABUTMENTS & PIER SHAFT — 3"  
PIER CAPS — 2"  
DECK TOP & BARRIER WALLS — 1 1/2"  
DECK BOTTOM — 1"










REVISIONS	
DATE	DESCRIPTION
DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
Consulting Engineers & Planners	
<b>RAMSAY CREEK BRIDGE</b> (WESTBOUND LANES) 4.1 MILES EAST OF INNES RD. KING'S HIGHWAY No. 417 DIST. No. 9 CO. REG. MUN. OTTAWA-CARLETON TWP. GLOUCESTER LOT 20 CON. 5	
<b>GENERAL LAYOUT</b>	
APPROVED	SITE No. 3-265 W.P. No. 10-69-07
DESIGN L.J.T. CHECK	CONTRACT No. 73-195
DRAWING C.M.G. CHECK L.J.T.	DRAWING No. 3-265-1
DATE DEC 72	LOADING HS20-44



202



# LEGEND

	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation, APR. 1972		
	Head		
	Encountered		
	ARTESIAN CONDITION		
NO.	ELEVATION	CO-ORDINATES	
		NORTH	EAST
101	241.4	490,248	239,896
102	229.2	490,212	239,949
103	225.7	490,230	240,016
104	228.5	490,177	240,067
105	240.7	490,177	240,131
ORIGINAL BORE HOLES 68-F-54			
2	227.1	490,275	239,994
3	250.8	490,304	239,862
8	243.0	490,277	239,902
9	228.5	490,273	240,085
10	245.6	490,226	240,135

# NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION & COMMUNICATIONS  
DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

**RAMSAY CREEK & BASE-LINE RD.**

HIGHWAY NO. 417 W.B.L. DIST NO. 9  
REG. MUN. OTTAWA-CARLETON  
TWP. GLOUCESTER LOT 20 CON. 5

**BORE HOLE LOCATIONS & SOIL STRATA**

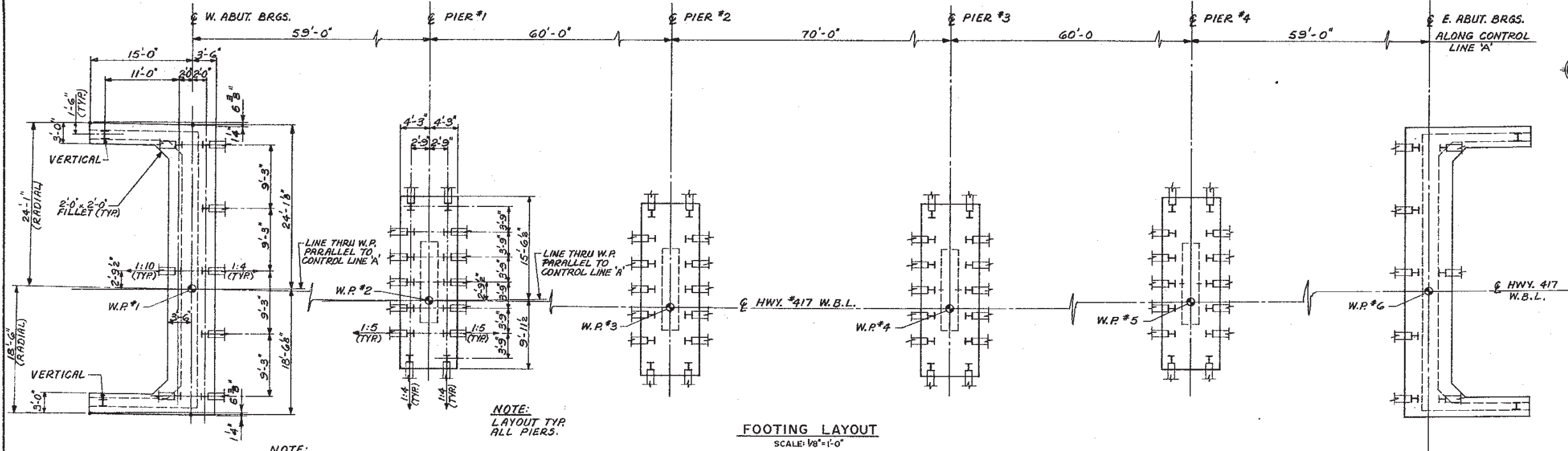
SUBMD. B.D. CHECKED	W.P. NO. 10-69-07	DRAWING NO.
DRAWN S.O. CHECKED	JOB NO. 72-11052	72-11052 A
DATE 13 JUNE 1972	SITE NO. 3-265	BRIDGE DRAWING NO.
APPROVED	CONT. NO. 73-90	3-265-2

PRINCIPAL FOUNDATION ENGINEER

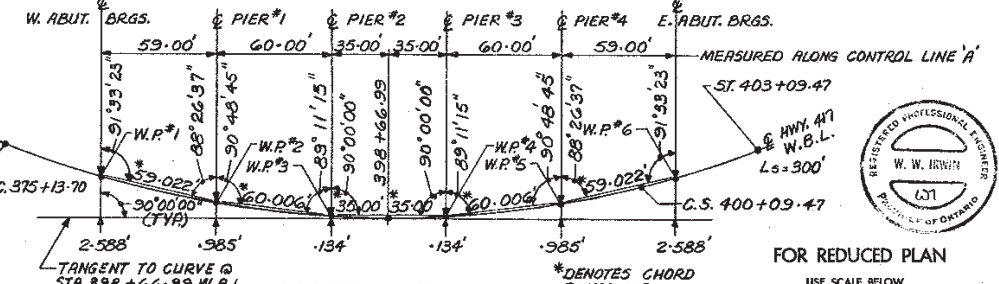
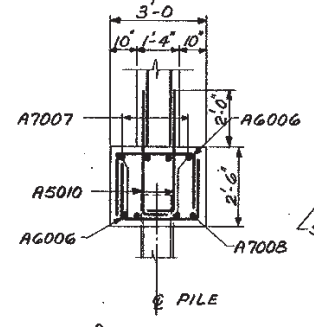
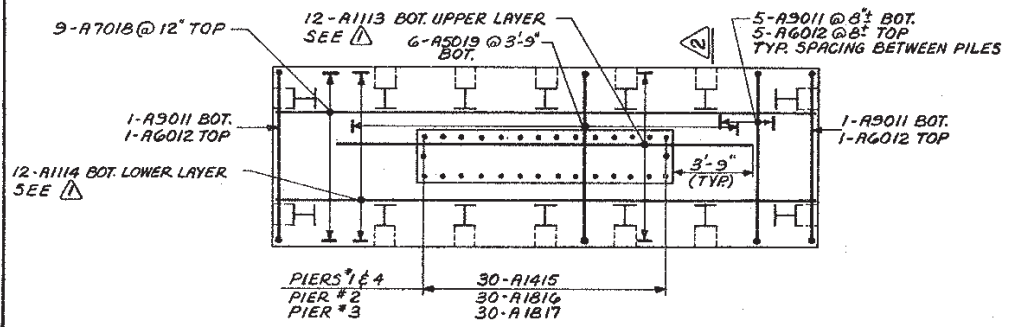
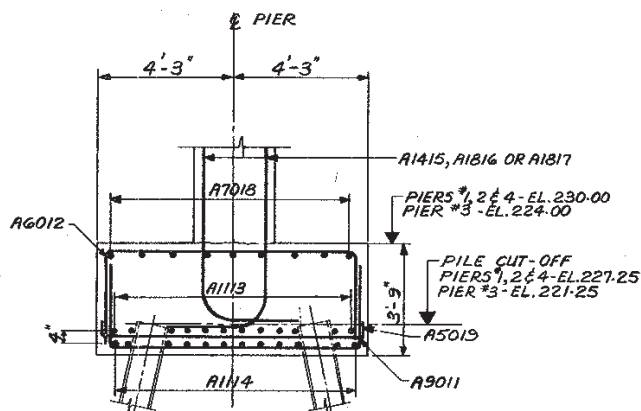
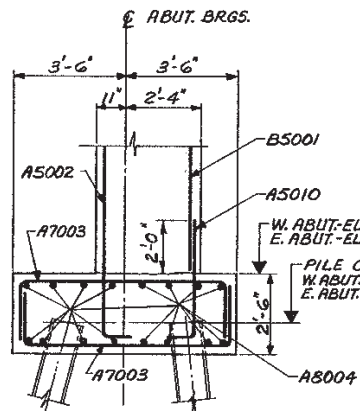
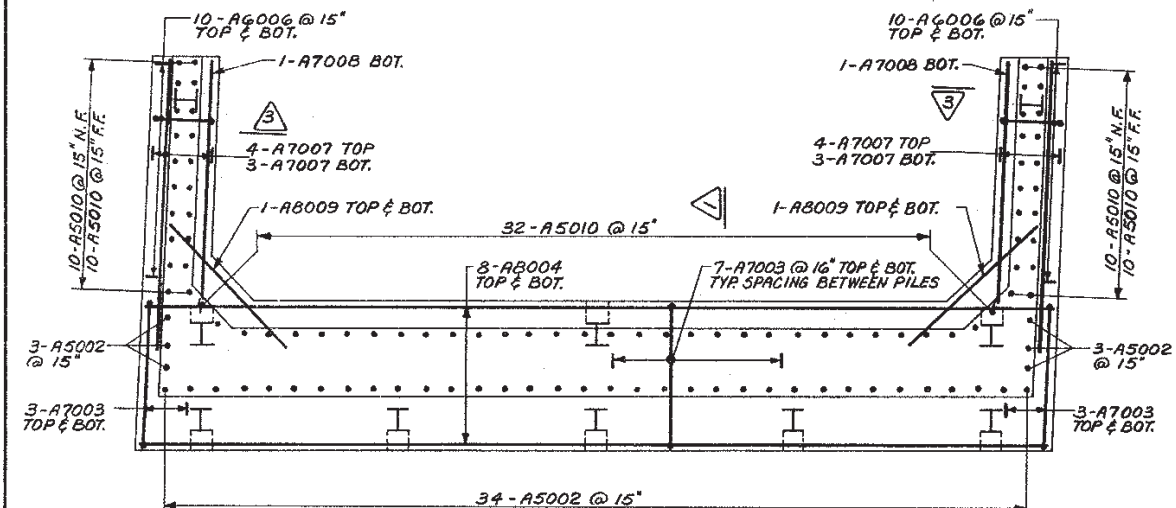
NOTE: The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the Ottawa District Office.



REF. NO. E-5230-1



STEEL H-PILE DATA				
LOCATION	BATTER	No.	LENGTH	TYPE
WEST ABUT.	VERT.	2	172'	HPI 34
	1:4	5	178'	
PIER #1	1:10	3	174'	
	1:5	10	154'	
PIER #2	1:4	4	156'	
	1:5	10	154'	
PIER #3	1:4	4	156'	
	1:5	10	148'	
PIER #4	1:4	4	156'	
	1:5	10	154'	
EAST ABUT.	VERT.	2	172'	
	1:4	5	176'	



- NOTES:**
- PILES TO BE DRIVEN TO BEDROCK
  - SPACING OF PILES TO BE MEASURED AT UNDERSIDE OF FOOTINGS.
  - BOTTOM REINF. TO BE SPACED TO AVOID PILES.
  - N.F. DENOTES NEAR FACE
  - F.F. DENOTES FAR FACE

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO

Consulting Engineers & Planners

**RAMSAY CREEK BRIDGE**  
(WESTBOUND LANES)  
4.1 MILES EAST OF INNES RD.  
KING'S HIGHWAY No. 417  
CO. REG. MUN. OTTAWA-CARLETON  
TWP. GLOUCESTER LOT 20 CON. 5

**FOUNDATION LAYOUT & REINFORCEMENT**

APPROVED: [Signature] SITE No. 3-265 W.P. No. 10-89-07  
DESIGN: [Signature] CHECK: [Signature] CONTRACT No. [Signature]  
DRAWING: A.G.Y. CHECK: J.C. DRAWING No. 3-265-3  
DATE: DEC. 72 LOADING: HS 20-44

## PRESENTATION OF SITE INVESTIGATION RESULTS

**Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd  
Ottawa, Ontario**

*Prepared for:*

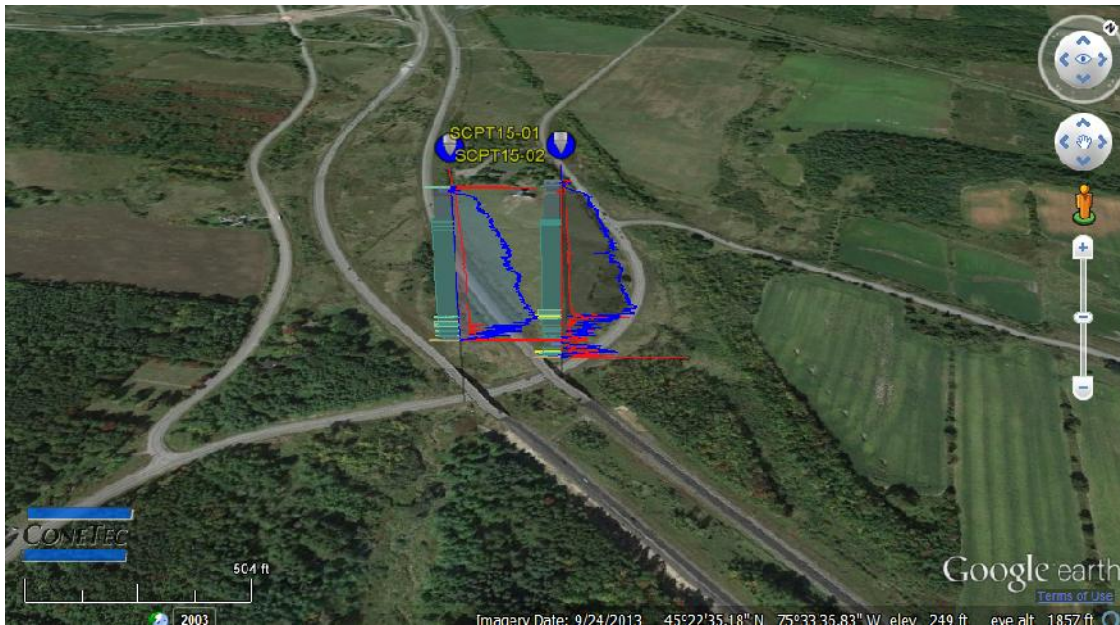
Thurber Engineering

ConeTec Job No: 15-05005

Project Start Date: 18-Feb-2015

Project End Date: 18-Feb-2015

Report Date: 24-Feb-2015



*Prepared by:*

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## Introduction

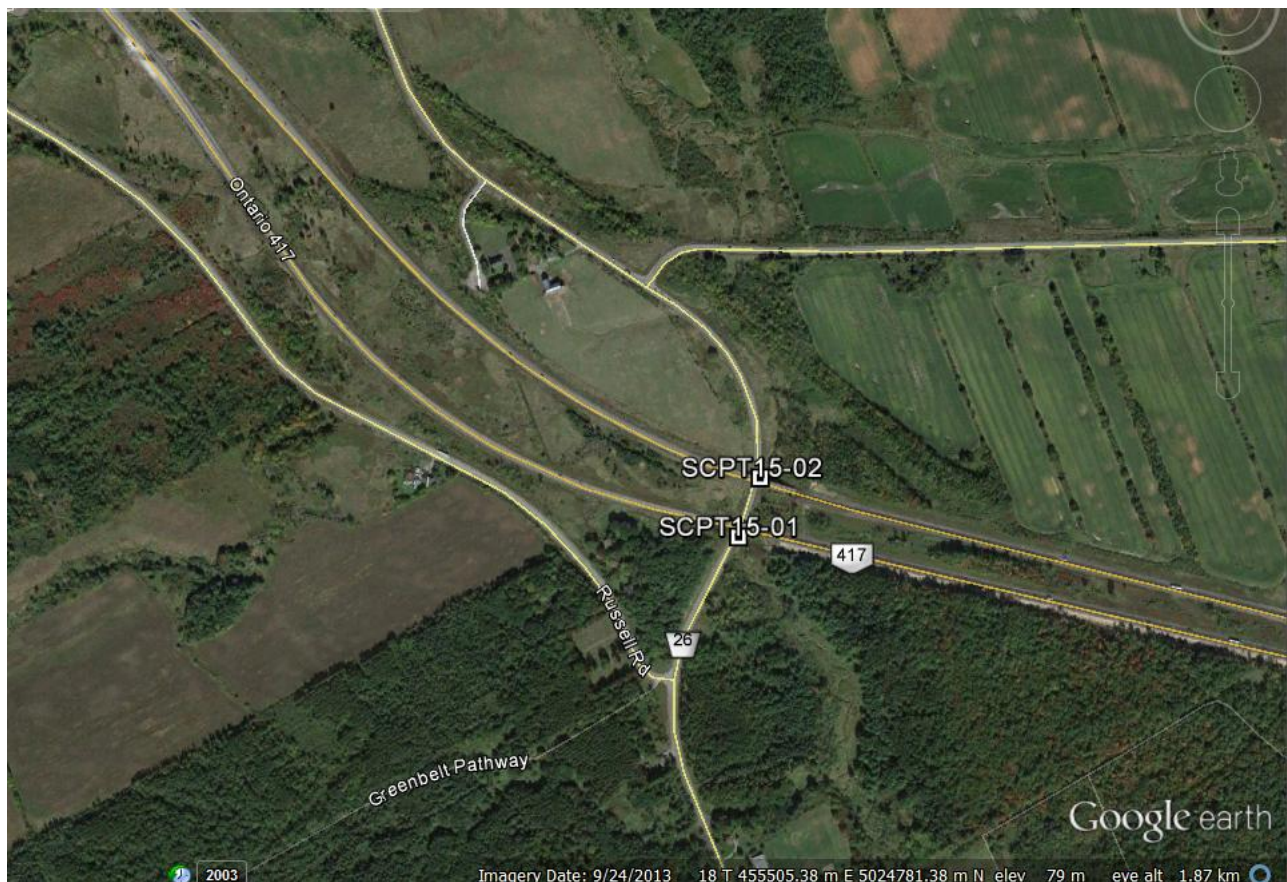
The enclosed report presents the results of a seismic piezocone penetration testing (SCPTu or SCPT) program carried out at the site of a proposed new highway 417 bridge over Ramsayville Road (regional road 26) east of Ottawa, Ontario. The site investigation program was conducted by ConeTec Investigations Limited, under contract to Thurber Engineering of Ottawa, Ontario.

A total of two seismic cone penetration tests were completed at two locations. The SCPT program was performed to evaluate the in situ properties of the soils prior to construction. SCPT sounding locations were selected and numbered under the supervision of Thurber personnel (Mr. Chris Murray).

## Project Information

Project	
Client	Thurber Engineering
Project	Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON
ConeTec project number	15-05005

A map from Google earth including the CPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig	25 ton truck mounted (twin cylinders)	CPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT	GPS (GlobalSat MR-350)	32618 (WGS 84 / UTM North)

Cone Penetration Test (CPT)	
Depth reference	Ground surface at the time of the investigation.
Tip and sleeve data offset	0.1 meter. This has been accounted for in the CPT data files.
Pore pressure dissipation (PPD) tests	One pore pressure dissipation tests were completed primarily to determine excess pore pressure conditions at depth.
Additional Comments	Seismic shear wave velocity testing was performed at two locations at one meter intervals.

Cone Description	Cone Number	Cross Sectional Area (cm <sup>2</sup> )	Sleeve Area (cm <sup>2</sup> )	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
361:T1500F15U500	361	15	225	1500	15	500

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm<sup>2</sup> and 15 cm<sup>2</sup> tip base area configurations in order to maximize signal resolution for various soil conditions. The 15 cm<sup>2</sup> penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm<sup>2</sup> piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u<sub>2</sub>" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

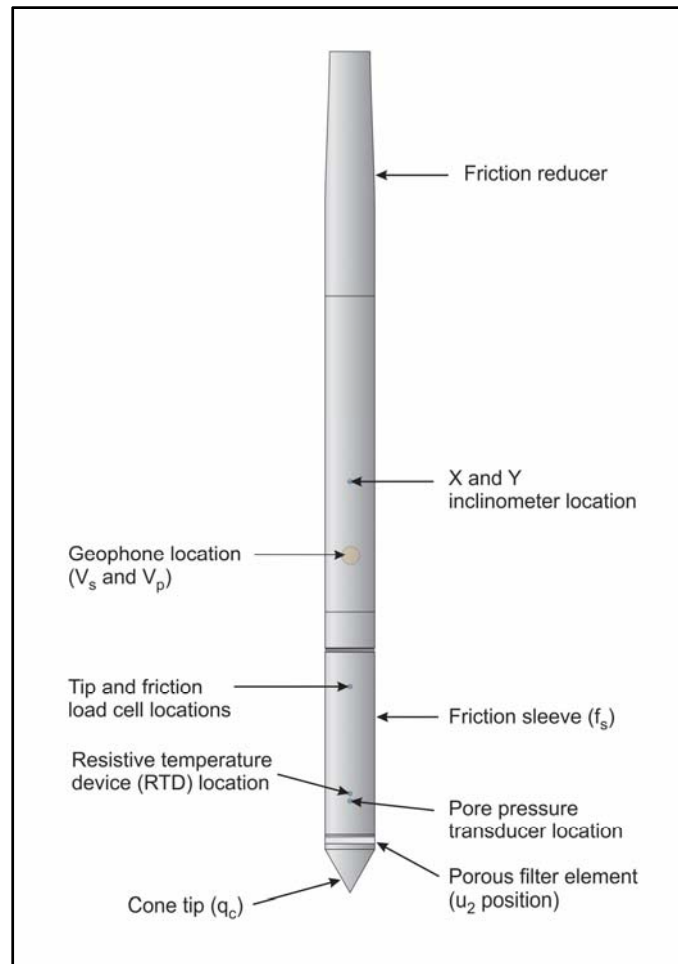


Figure CPTu. Piezocone Penetrometer (15 cm<sup>2</sup>)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance ( $q_c$ )
- Sleeve friction ( $f_s$ )
- Dynamic pore pressure ( $u$ )
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance ( $q_t$ ), sleeve friction ( $f_s$ ) and pore water pressure ( $u$ ). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance ( $q_c$ ) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance ( $q_t$ ) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1-a) \cdot u_2$$

where:  $q_t$  is the corrected tip resistance

$q_c$  is the recorded tip resistance

$u_2$  is the recorded dynamic pore pressure behind the tip ( $u_2$  position)

$a$  is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction ( $f_s$ ) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure ( $u$ ) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio ( $R_f$ ) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high

friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is included in an appendix.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

### References

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420.

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization 4, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355.

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave ( $V_p$ ) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

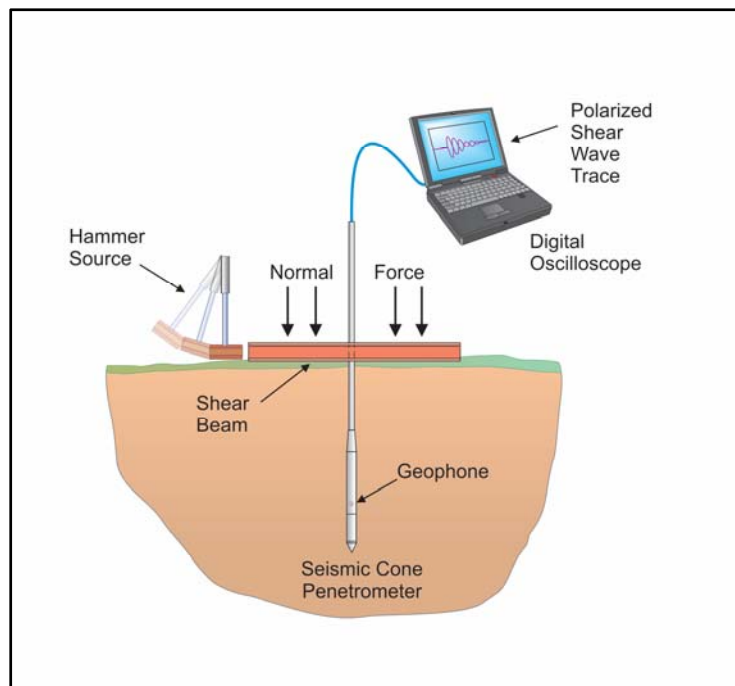


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

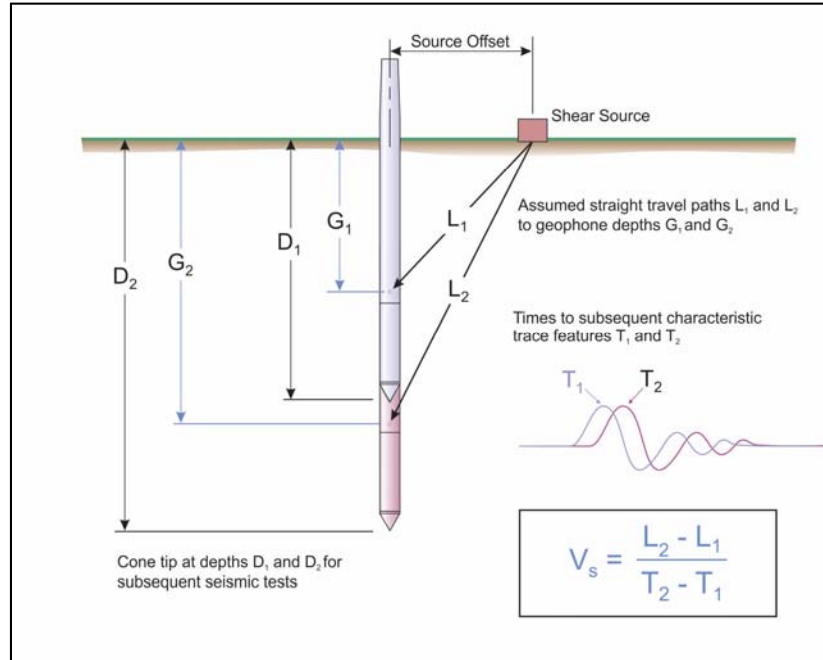


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 30 meters ( $V_{s30}$ ) has been calculated and provided for all applicable soundings using an equation presented in Crow et al., 2012.

$$V_{s30} = \frac{\text{total thickness of all layers (30m)}}{\sum(\text{layer traveltimes})}$$

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

#### References

Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, "National shear wave measurement guidelines for Canadian seismic site assessment", GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure ( $u$ ) with time ( $t$ ).

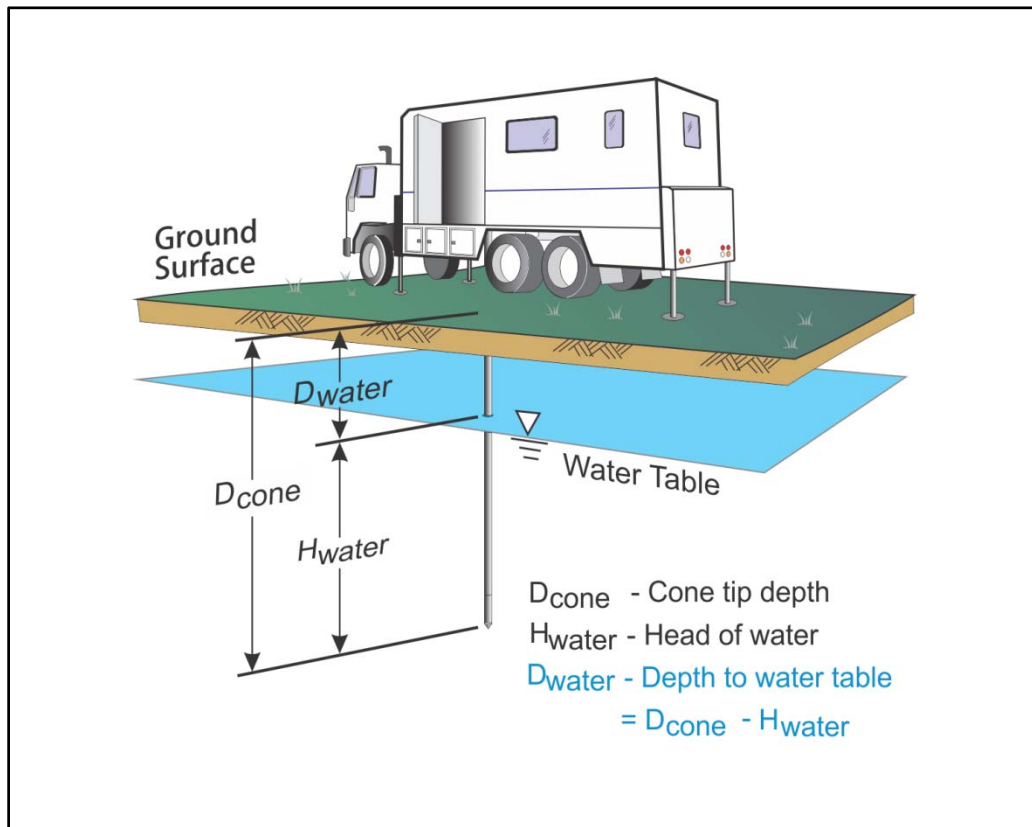


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

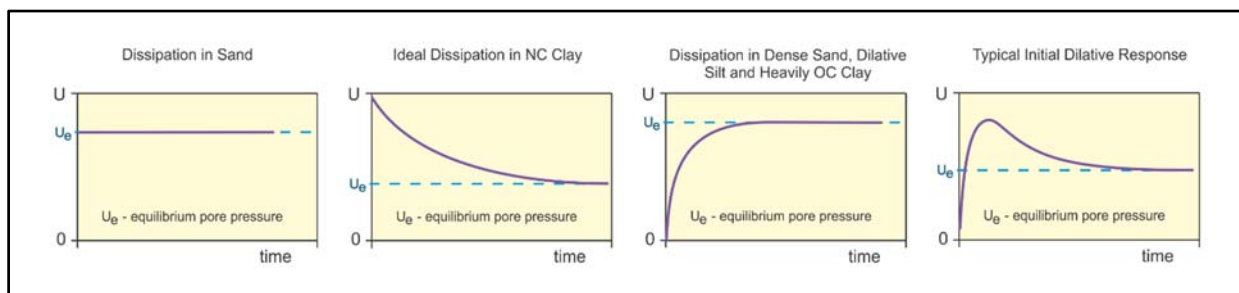


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure ( $u_{eq}$ ) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as  $t_{100}$ . In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to  $t_{100}$ . A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor ( $T^*$ ) may be used to calculate the coefficient of consolidation ( $c_h$ ) at various degrees of dissipation resulting in the expression for  $c_h$  shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

- $T^*$  is the dimensionless time factor (Table Time Factor)  
 $a$  is the radius of the cone  
 $I_r$  is the rigidity index  
 $t$  is the time at the degree of consolidation

Table Time Factor.  $T^*$  versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
$T^* (u_2)$	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time ( $t_{50}$ ) corresponding to a degree of dissipation of 50% ( $u_{50}$ ). In order to determine  $t_{50}$ , dissipation tests must be taken to a pressure less than  $u_{50}$ . The  $u_{50}$  value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as  $u_{100}$ . To estimate  $u_{50}$ , both the initial maximum pore pressure and  $u_{100}$  must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure ( $u$  at  $t_{100}$ ) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly ( $u_{100}$ ), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of  $c_h$  (Teh and Houlsby, 1991),  $t_{50}$  values are estimated from the corresponding pore pressure dissipation curve and a rigidity index ( $I_r$ ) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining  $t_{50}$ . In cases where the time to peak is excessive,  $t_{50}$  values are not calculated.

Due to possible inherent uncertainties in estimating  $I_r$ , the equilibrium pore pressure and the effect of an initial dilatory response on calculating  $t_{50}$ , other methods should be applied to confirm the results for  $c_h$ .

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

### References

Burns, S.E. and Mayne, P.W., 1998, "Monotonic and dilatatory pore pressure decay during piezocone tests", Canadian Geotechnical Journal 26 (4): 1063-1073.

Burns, S.E. and Mayne, P.W., 2002, "Analytical cavity expansion-critical state model cone dissipation in fine-grained soils", Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, "The piezometer probe: a useful investigation tool", Proceedings, 10<sup>th</sup> International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, "Estimating coefficient of consolidation from piezocone tests", Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, "An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils", Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Normalized Cone Penetration Test Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

# Cone Penetration Test Summary and Standard Cone Penetration Test Plots



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Start Date: 18-Feb-2015  
End Date: 18-Feb-2015

### ***CONE PENETRATION TEST SUMMARY***

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface <sup>1</sup> (m)	Final Depth (m)	Shear Wave Velocity Tests	Northing <sup>2</sup> (m)	Easting (m)
SCPT15-01	15-05005_SP01	18-Feb-2015	361:T1500F15U500	4.0	39.90	39	5024585	455661
SCPT15-02	15-05005_SP02	18-Feb-2015	361:T1500F15U500	4.0	47.95	46	5024679	455697
Totals	2 soundings				87.85	85		

1. Assumed phreatic surface depths were derived from pore pressure dissipation test data. Hydrostatic data were used for interpretation tables.
2. Coordinates are WGS 84 / UTM Zone 18 and were collected using MR350 GlobalSat GPS Receiver.



# Thurber Engineering

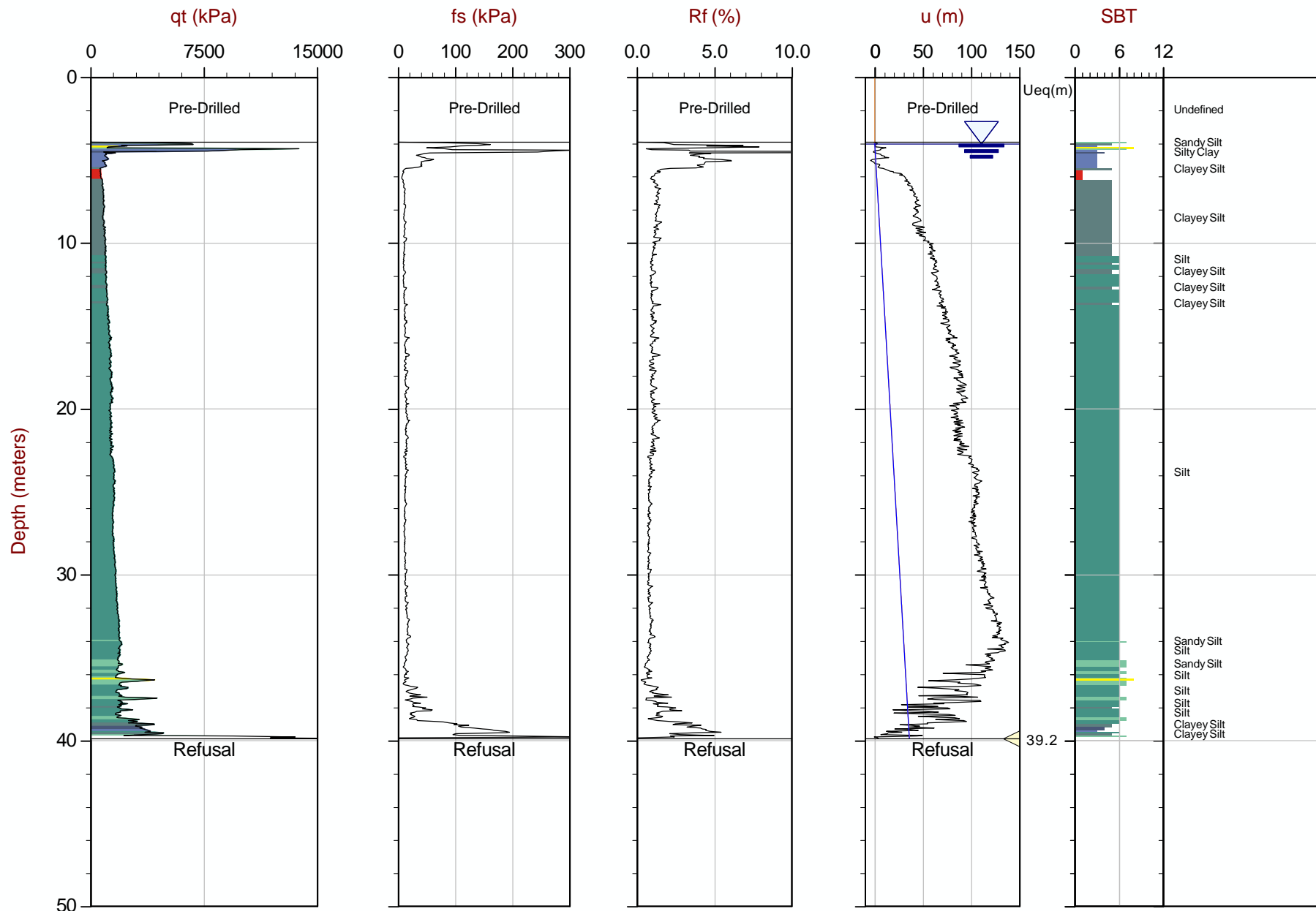
Job No: 15-05005

Date: 02:18:15 10:08

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-01

Cone: 361:T1500F15U500



Max Depth: 39.900 m / 130.90 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP01.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024585 E: 455661  
Page No: 1 of 1



# Thurber Engineering

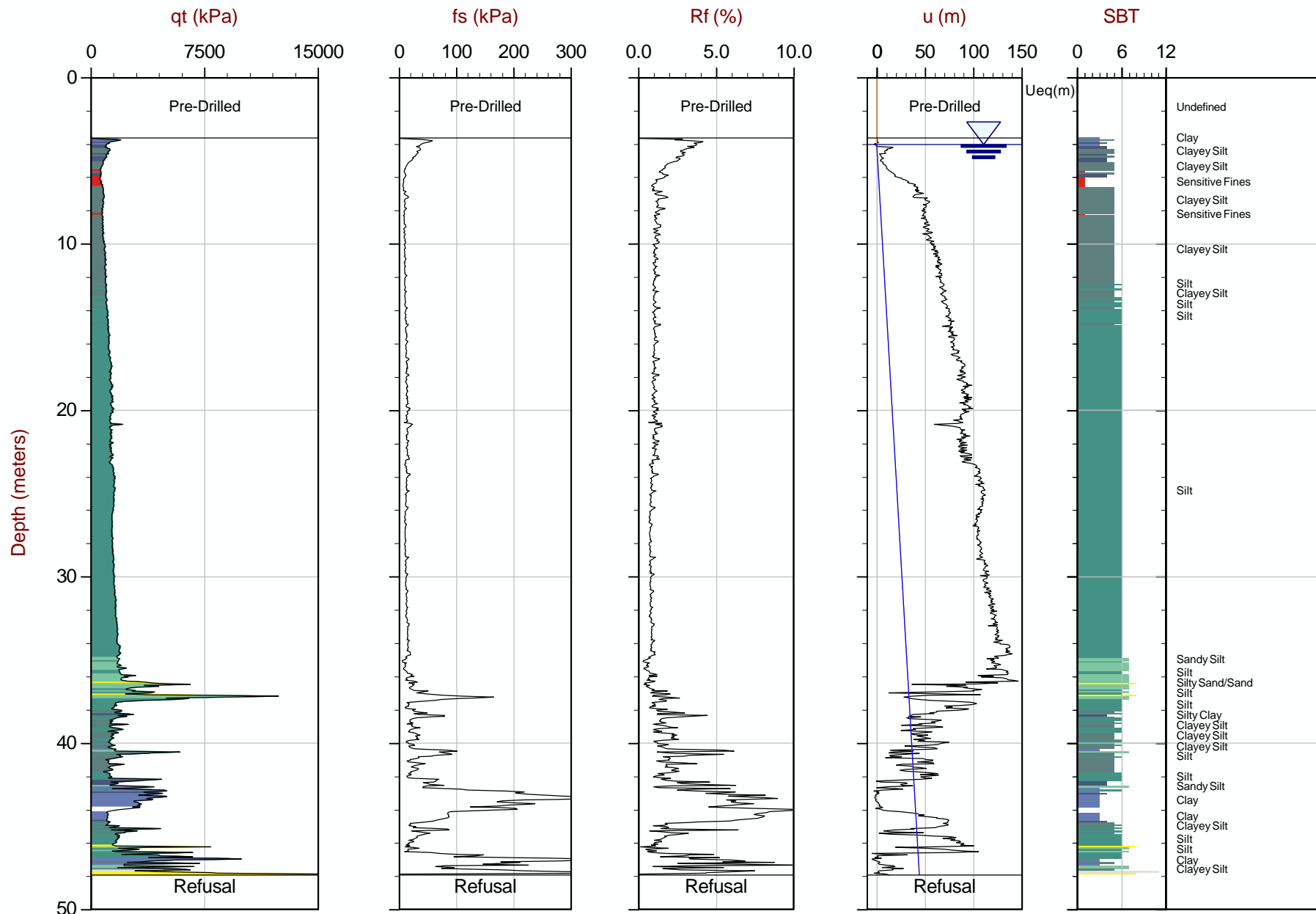
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Date: 02:18:15 12:52

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-02

Cone: 361:T1500F15U500



Max Depth: 47.950 m / 157.31 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP02.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024679 E: 455697  
Page No: 1 of 1

## Normalized Cone Penetration Test Plots



# Thurber Engineering

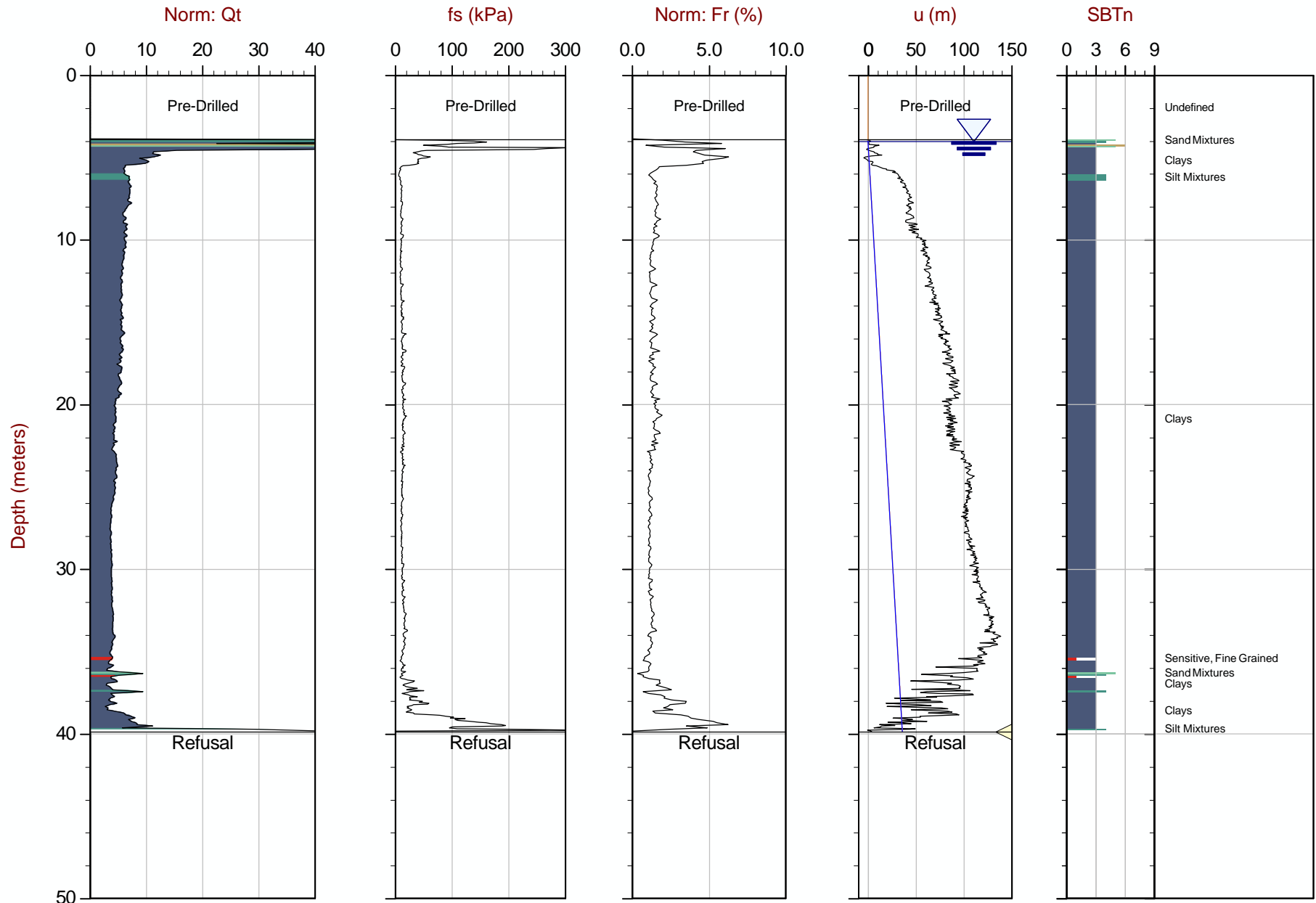
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Date: 02:18:15 10:08

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-01

Cone: 361:T1500F15U500



Max Depth: 39.900 m / 130.90 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP01.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024585 E: 455661  
Page No: 1 of 1



# Thurber Engineering

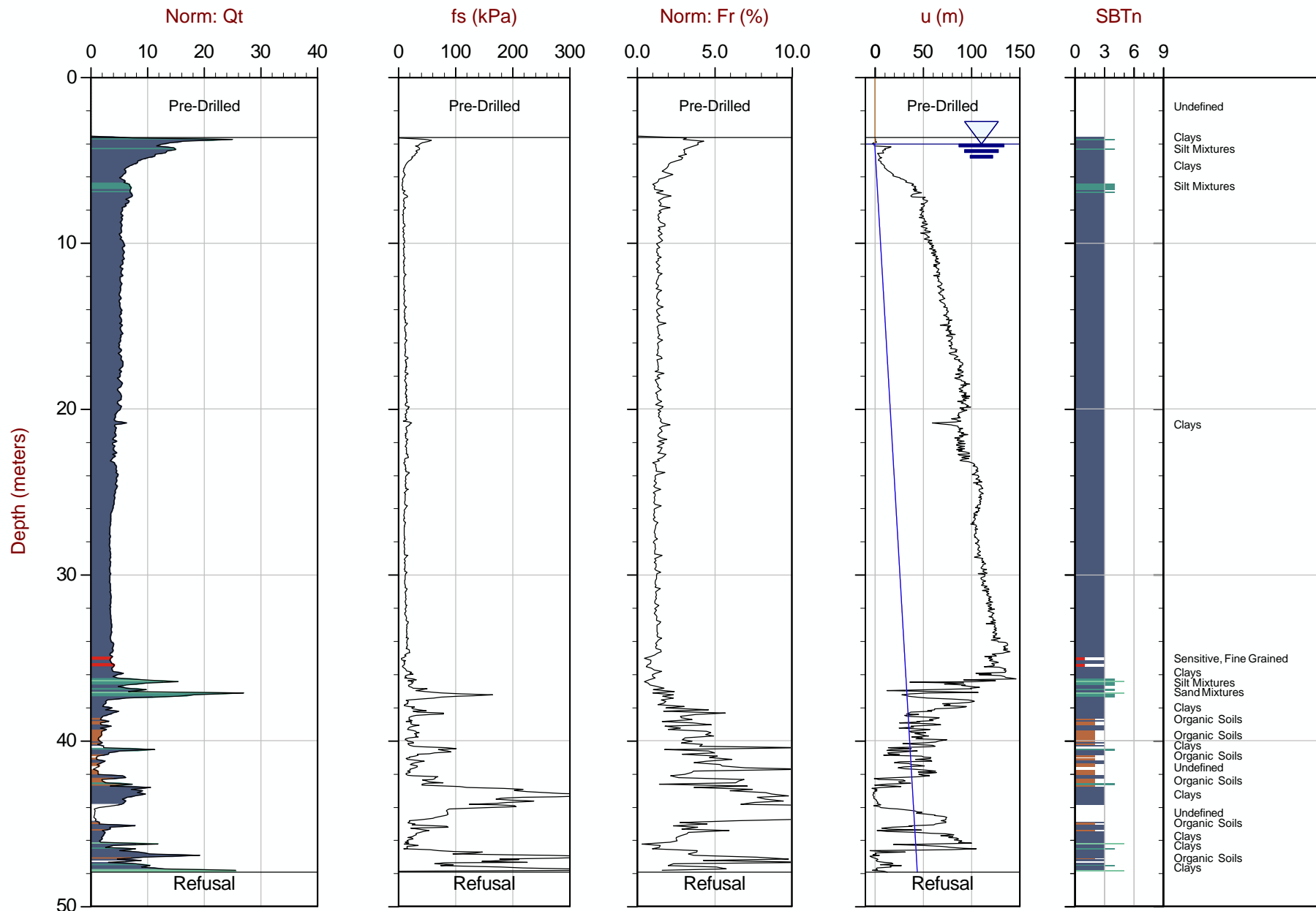
Job No: 15-05005

Date: 02:18:15 12:52

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-02

Cone: 361:T1500F15U500

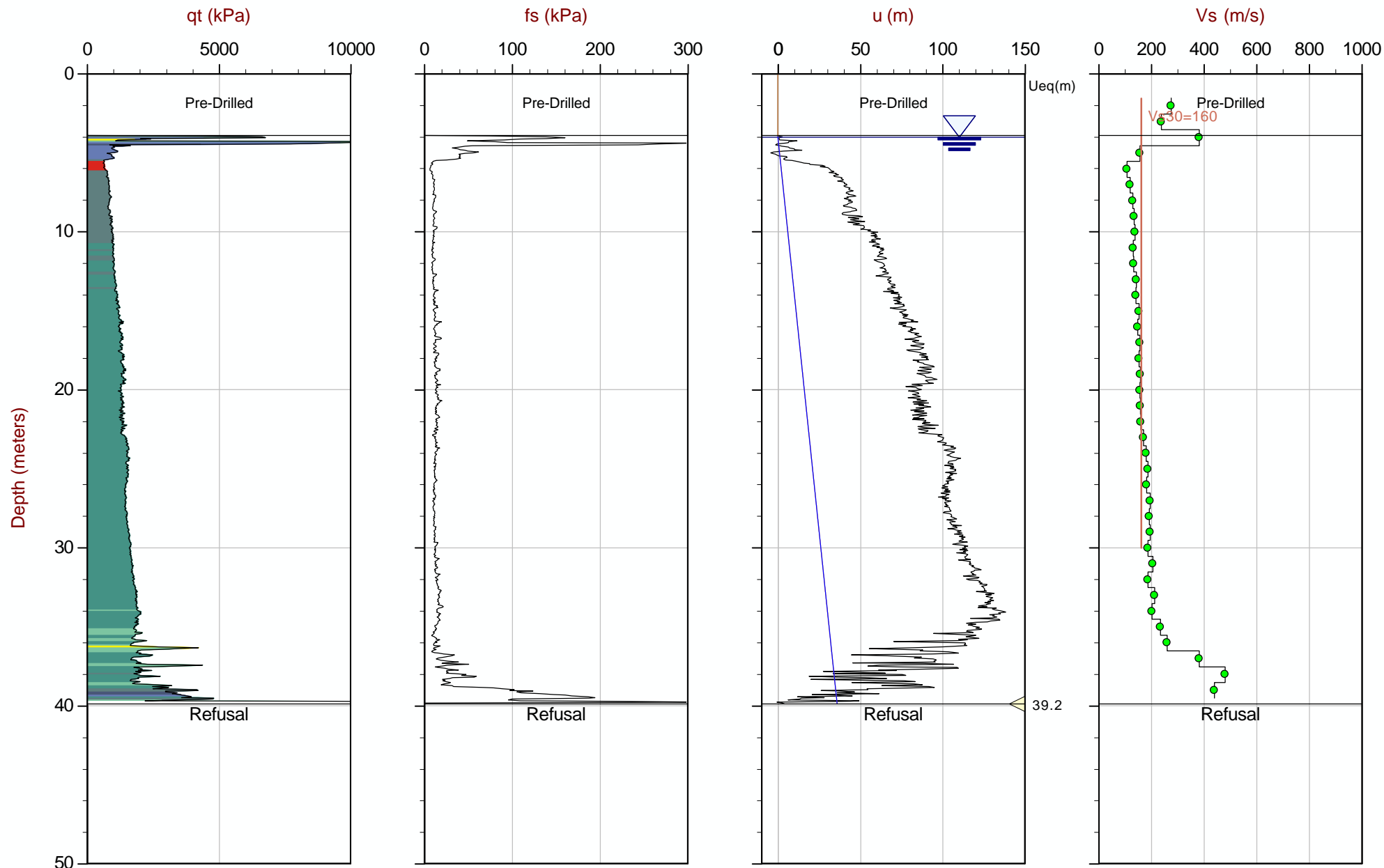


Max Depth: 47.950 m / 157.31 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP02.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024679 E: 455697  
Page No: 1 of 1

## Seismic Cone Penetration Test Plots



Max Depth: 39.900 m / 130.90 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP01.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024585 E: 455661  
Page No: 1 of 1



# Thurber Engineering

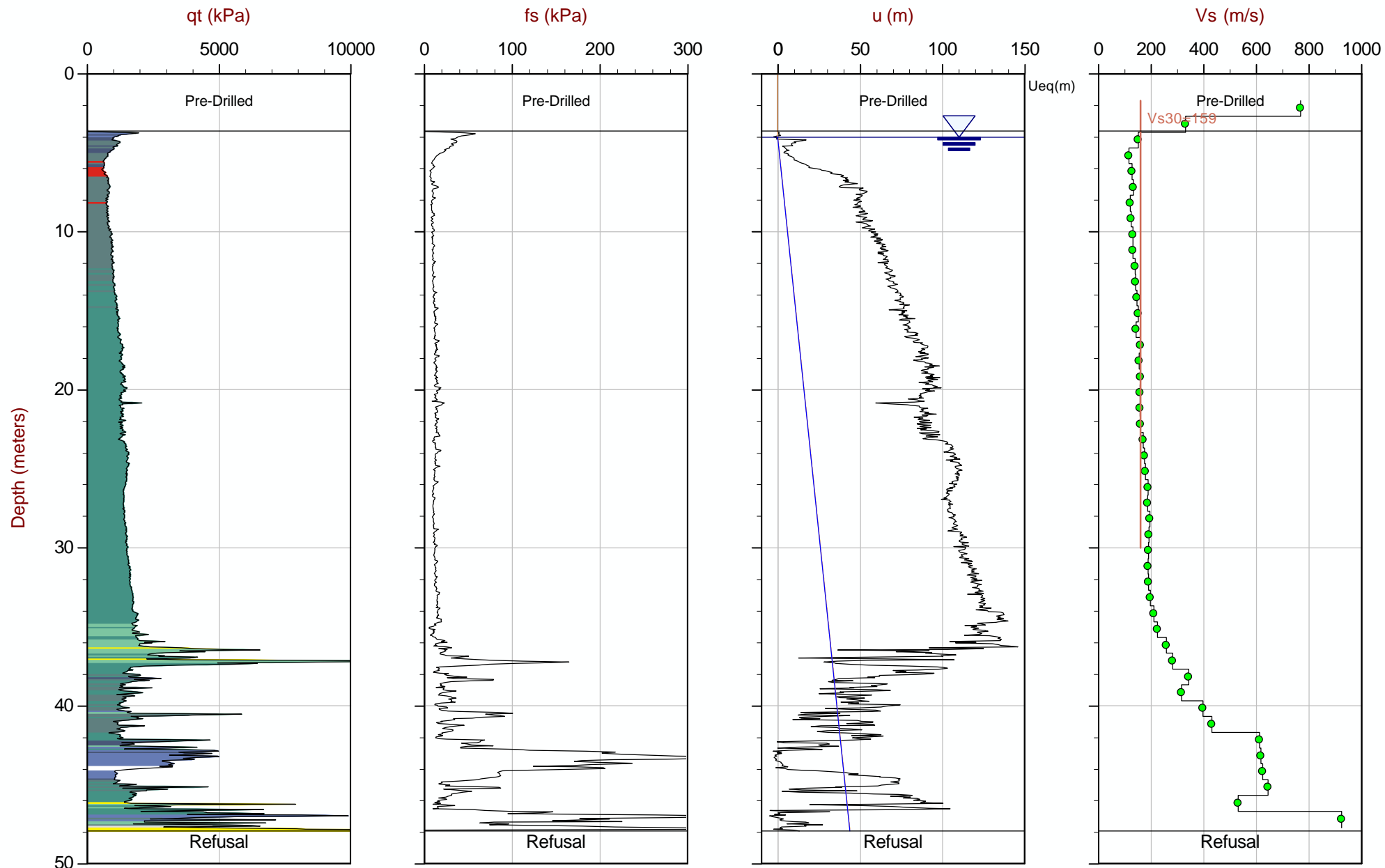
Job No: 15-05005

Date: 02:18:15 12:52

Site: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON

Sounding: SCPT15-02

Cone: 361:T1500F15U500



Max Depth: 47.950 m / 157.31 ft  
Depth Inc: 0.050 m / 0.164 ft  
Avg Int: 0.100 m

File: 15-05005\_SP02.COR

SBT: Lunne, Robertson and Powell, 1997  
Coords: UTM Zone 18 N: 5024679 E: 455697  
Page No: 1 of 1

## Seismic Cone Penetration Test Tabular Results



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Sounding ID: SCPT15-01  
Date: 18-Feb-2015

Seismic Source: Beam  
Source Offset (m): 0.60  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### **SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>**

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.75	1.55	1.66			
2.75	2.55	2.62	0.96	3.49	275
3.75	3.55	3.60	0.98	4.12	238
4.75	4.55	4.59	0.99	2.59	382
5.75	5.55	5.58	0.99	6.36	156
6.75	6.55	6.58	1.00	9.18	108
7.75	7.55	7.57	1.00	8.32	120
8.75	8.55	8.57	1.00	7.72	129
9.75	9.55	9.57	1.00	7.46	134
10.75	10.55	10.57	1.00	7.26	138
11.75	11.55	11.57	1.00	7.54	132
12.75	12.55	12.56	1.00	7.54	133
13.75	13.55	13.56	1.00	7.01	143
14.75	14.55	14.56	1.00	7.04	142
15.75	15.55	15.56	1.00	6.48	154
16.75	16.55	16.56	1.00	6.70	149
17.75	17.55	17.56	1.00	6.39	156
18.75	18.55	18.56	1.00	6.47	154
19.75	19.55	19.56	1.00	6.32	158
20.75	20.55	20.56	1.00	6.37	157
21.75	21.55	21.56	1.00	6.34	158
22.75	22.55	22.56	1.00	6.21	161
23.75	23.55	23.56	1.00	5.88	170
24.75	24.55	24.56	1.00	5.55	180
25.75	25.55	25.56	1.00	5.34	187
26.75	26.55	26.56	1.00	5.50	182
27.75	27.55	27.56	1.00	5.13	195
28.75	28.55	28.56	1.00	5.21	192
29.75	29.55	29.56	1.00	5.14	195



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Sounding ID: SCPT15-01  
Date: 18-Feb-2015

Seismic Source: Beam  
Source Offset (m): 0.60  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### ***SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>***

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
30.75	30.55	30.56	1.00	5.31	188
31.75	31.55	31.56	1.00	4.87	206
32.75	32.55	32.56	1.00	5.36	187
33.75	33.55	33.56	1.00	4.69	213
34.75	34.55	34.56	1.00	4.96	202
35.75	35.55	35.56	1.00	4.28	234
36.75	36.55	36.55	1.00	3.85	260
37.75	37.55	37.55	1.00	2.62	382
38.75	38.55	38.55	1.00	2.08	480
39.75	39.55	39.55	1.00	2.27	440



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Sounding ID: SCPT15-02  
Date: 18-Feb-2015

Seismic Source: Beam  
Source Offset (m): 0.60  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### **SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>**

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.90	1.70	1.80			
2.90	2.70	2.77	0.96	1.25	768
3.90	3.70	3.75	0.98	2.96	332
4.90	4.70	4.74	0.99	6.50	152
5.90	5.70	5.73	0.99	8.57	116
6.90	6.70	6.73	1.00	7.76	128
7.90	7.70	7.72	1.00	7.47	133
8.90	8.70	8.72	1.00	8.14	122
9.90	9.70	9.72	1.00	7.97	125
10.90	10.70	10.72	1.00	7.64	131
11.90	11.70	11.72	1.00	7.62	131
12.90	12.70	12.71	1.00	7.11	140
13.90	13.70	13.71	1.00	7.07	141
14.90	14.70	14.71	1.00	6.79	147
15.90	15.70	15.71	1.00	6.63	151
16.90	16.70	16.71	1.00	6.94	144
17.90	17.70	17.71	1.00	6.23	160
18.90	18.70	18.71	1.00	6.44	155
19.90	19.70	19.71	1.00	6.19	161
20.90	20.70	20.71	1.00	6.28	159
21.90	21.70	21.71	1.00	6.30	159
22.90	22.70	22.71	1.00	6.22	161
23.90	23.70	23.71	1.00	5.84	171
24.90	24.70	24.71	1.00	5.73	175
25.90	25.70	25.71	1.00	5.57	179
26.90	26.70	26.71	1.00	5.28	189
27.90	27.70	27.71	1.00	5.36	187
28.90	28.70	28.71	1.00	5.14	195
29.90	29.70	29.71	1.00	5.18	193



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Sounding ID: SCPT15-02  
Date: 18-Feb-2015

Seismic Source: Beam  
Source Offset (m): 0.60  
Source Depth (m): 0.00  
Geophone Offset (m): 0.20

### **SCPT<sub>u</sub> SHEAR WAVE VELOCITY TEST RESULTS - V<sub>s</sub>**

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
30.90	30.70	30.71	1.00	5.22	191
31.90	31.70	31.71	1.00	5.28	189
32.90	32.70	32.71	1.00	5.24	191
33.90	33.70	33.71	1.00	5.08	197
34.90	34.70	34.71	1.00	4.73	211
35.90	35.70	35.71	1.00	4.45	225
36.90	36.70	36.70	1.00	3.87	259
37.90	37.70	37.70	1.00	3.53	283
38.90	38.70	38.70	1.00	2.91	344
39.90	39.70	39.70	1.00	3.16	316
40.90	40.70	40.70	1.00	2.52	398
41.90	41.70	41.70	1.00	2.32	431
42.90	42.70	42.70	1.00	1.63	613
43.90	43.70	43.70	1.00	1.62	617
44.90	44.70	44.70	1.00	1.60	624
45.90	45.70	45.70	1.00	1.55	645
46.90	46.70	46.70	1.00	1.88	532
47.95	47.75	47.75	1.05	1.14	925

Pore Pressure Dissipation Summary and  
Pore Pressure Dissipation Plots



Job No: 15-05005  
Client: Thurber Engineering  
Project: Sites 3-265/1 and 3-265/2, Hwy 417 & Ramsayville Rd, Ottawa, ON  
Start Date: 18-Feb-2015  
End Date: 18-Feb-2015

### ***CPT<sub>u</sub> PORE PRESSURE DISSIPATION SUMMARY***

Sounding ID	File Name	Cone Area (cm <sup>2</sup> )	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U <sub>eq</sub> (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)
SCPT15-01	15-05005_SP01	15	305	40	39.22	0.7	4.0
Totals	1 Dissipations		5.1 min				



*Thurber Engineering*

Job No: 15-05005

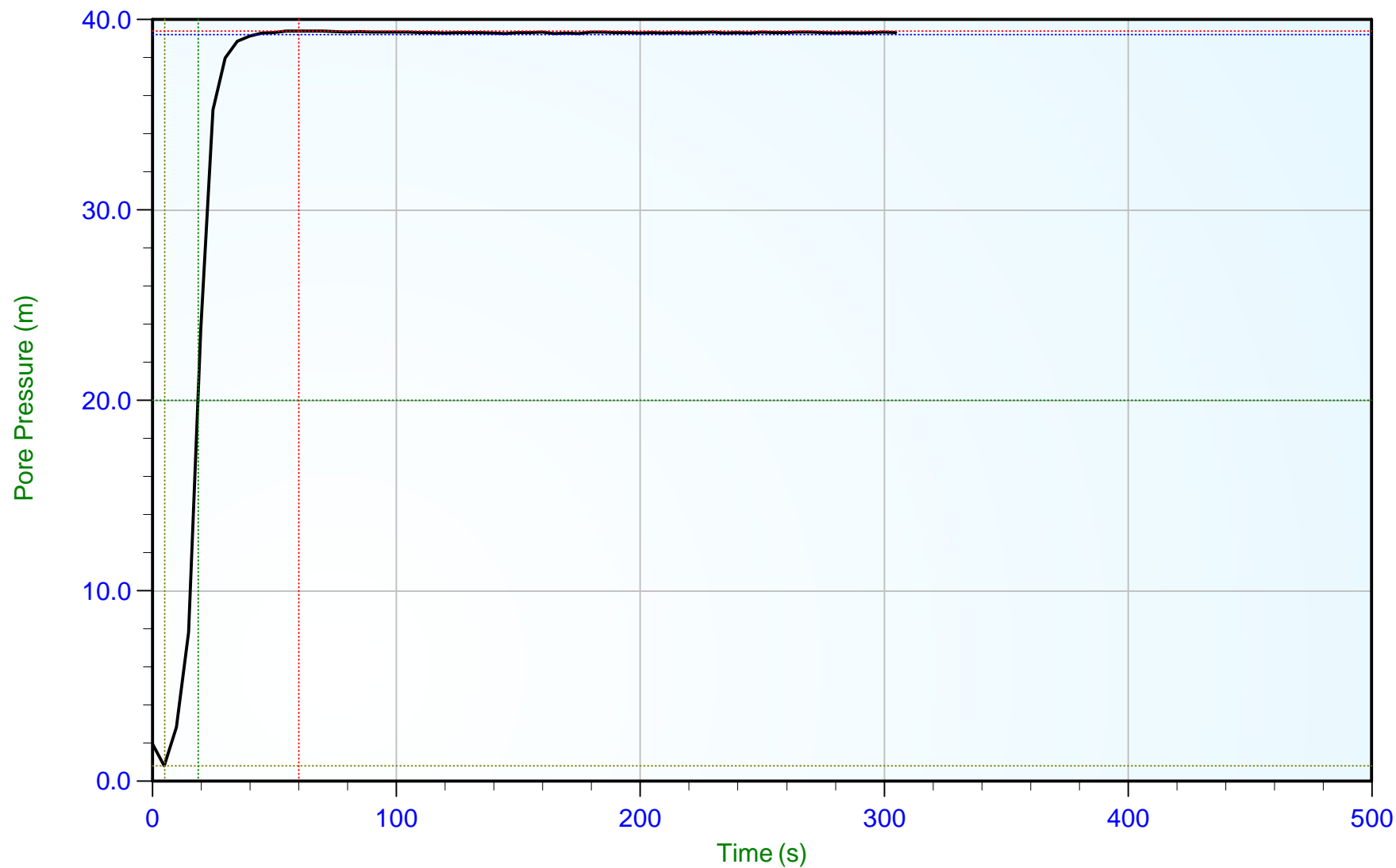
Date: 18-Feb-2015 10:08:56

Site: Hwy 417 & Ramsayville Rd

Sounding: SCPT15-01

Cone: AD361

Cone Area: 15 sq cm



Trace Summary: Filename: 15-05005\_SP01.PPD U Min: 0.8 m WT: 0.676 m / 2.217 ft  
Depth: 39.900 m / 130.904 ft U Max: 39.4 m Ueq: 39.2 m  
Duration: 305.0 s