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Ministry of
Transportation and Infrastructure

MEMORANDUM

EGBC Permit To Practice No.: 1003429

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Sr. Structural Design Engineer
South Coast Region

October 18, 2023

MoTI Project No.: 12712-0000

Re: Highway #3 Snass Creek Bridge Replacement (Structure Number: 01214)
GEOTECHNICAL RECOMMENDATIONS

1.0 INTRODUCTION

This memorandum provides a summary of geotechnical engineering analyses completed for the proposed Highway #3 Snass Creek Bridge replacement project and includes geotechnical recommendations for design and construction of the project.

The existing Snass Creek Bridge (SN 01214) is located at approximately 37.8 km to the east of Hope on Hwy #3 in British Columbia. The existing bridge, originally constructed in 1949 with additional widening in 1963, is approaching its service life and needs to be replaced.

Based on the Structural drawings prepared by the Ministry of Transportation and Infrastructure (MoTI), the proposed new bridge will be a single-span structure (span width of about 24.8m) located at the same location as the existing bridge. The proposed new bridge will be designed to carry three travel lanes and the width of the proposed new bridge will be adequate for future expansion to four (4) travel lanes. The alignment of the existing highway (approximately 400m on each side of the bridge) will be slightly shifted to match the new wider bridge.

The new bridge will comprise reinforced concrete girders supported on seven (7) 610mm diameter open-ended steel pipe piles at each abutment.

2.0 DESIGN STANDARD

The proposed new Highway #3 Snass Creek Bridge is designed in accordance with the following listed standards:

- Canadian Highway Bridge Design Code (CHBDC S06-19); and,
- BC MoTI Bridge Standards and Procedures Supplement to CHBDC S06-19.

3.0 GEOTECHNICAL EXPLORATION AND SUBSURFACE CONDITIONS

3.1 GEOTECHNICAL EXPLORATION

In order to assess the subsoil and groundwater conditions, the Ministry completed a field geotechnical investigation program between September 20 and 23, 2018. A total of ten (10) test holes were completed during the site investigation, including two (2) deep mud rotary boreholes and eight (8) shallow solid stem auger holes. The test hole locations are shown in the geotechnical factual report attached in Appendix B.

All test holes were completed using a subcontracted truck mounted drill rig supplied and operated by Sea-To-Sky Drilling Ltd. of Burnaby. The mud rotary boreholes were designated as BH18-01 to BH18-02, and solid stem auger holes were designated as AH18-01 to AH18-10.

BH18-01 (east abutment) and BH18-02 (west abutment) were drilled to depths of 30.33m (99.5 ft.) and 29.26m (96 ft.), respectively. Standard Penetration Tests (SPTs) were conducted at regular intervals of 1.5m (5 ft.) within the mud rotary bore holes. Where obstacles (cobbles or boulders) were encountered, a Tricone drill bit was used to drill through making it necessary to skip SPTs at some depths. Once the target depths were achieved, the bore holes were terminated and backfilled using bentonite chips and sand in accordance with the groundwater protection regulation of British Columbia.

All eight (8) shallow solid stem auger holes were drilled on the paved shoulder close to the fog line of the highway. The auger holes were drilled to auger refusal between depths of 1.07m (3.5 ft.) and 3.66m (12 ft.). The auger holes were backfilled using the drilling spoils with cold asphalt patching of the road surface.

A summary of the completed test holes is provided in Table 1 below. The details of the test holes are provided in the Factual Geotechnical Report attached to Appendix B. Where cobbles and boulders were encountered, a 3-7/8" Tricone drill bit or HD coring barrel were used to advance the test holes. The depths of using Tricone and HD coring barrel are indicated on the test hole logs attached in Appendix B.

Table 1: Geotechnical Site Investigation Summary

Test Hole	Approximate Location* (UTM Zone 10)		Test Hole Depth (m)	Exploration Method	Location Comments
	Northing	Easting			
BH18-01	5454751	641343	30.33	Mud Rotary	Proposed East Abutment
BH18-02	5454752	641323	29.26	Mud Rotary	Proposed West Abutment
AH18-01	5454565	641158	1.98	Solid Stem Auger	
AH18-02	5454627	641183	3.05	Solid Stem Auger	
AH18-03	5454686	641209	3.05	Solid Stem Auger	
AH18-04	5454730	641251	1.07	Solid Stem Auger	
AH18-05	5454731	641411	1.37	Solid Stem Auger	
AH18-06	5454685	641458	2.29	Solid Stem Auger	
AH18-07	5454630	641477	3.66	Solid Stem Auger	
AH18-08	5454564	641486	2.74	Solid Stem Auger	

* Please note, the UTM coordinates are approximate and derived from Google Earth 2018.

3.2 SUBSURFACE CONDITIONS

3.2.1 Surficial Geology

Based on the Geological Survey of Canada Map 41-1989, sheet one, Hope, BC, the subsoil conditions at the site are anticipated to consist of thick colluvium deposits over bedrock (PM_u, Ultramafic Rock, local Gabbro).

3.2.2 Subsurface Soil Conditions

Detailed descriptions of the subsurface conditions encountered during site exploration are presented in the Geotechnical Factual Report prepared by MoTI in Appendix B. In summary, the field exploration confirmed that the site is underlain by about a 4.5 to 5m thickness of sandy gravel fill followed by a natural deposit of sand and gravel to sandy gravel to a depth of beyond 30.33m (99.5 ft.) (the depth of the deepest test hole BH18-01). Cobbles and boulders were encountered in both the fill and the natural sand and gravel to sandy gravel.

At the proposed new bridge abutments, the existing fill located within 5m from the existing ground surface was noted to be compact to dense with Standard Cone Penetration test (SPT) blow counts ranging between 19 to 42 blows per 300mm penetration. The natural deposit of sand and gravel to sandy gravel was noted to be dense to very dense with Standard Cone Penetration test (SPT) blow counts greater than 34 blows per 300mm penetration to a practical refusal.

3.2.3 Groundwater Condition

Due to the nature of the mud rotary drilling method (use of drilling fluid), the elevation of the static groundwater table could not be confirmed during site investigation. Considering the subsurface conditions (the site is underlain by permeable sand and gravel to sandy gravel) and proxinity to Snass Creek, the static groundwater level should be close to the level of Snass Creek.

The groundwater table is expected to be located at a geodetic elevation of approximately +741m in the summer ‘dry’ season and is expected to fluctuate with the water level in Snass Creek. Higher water levels (up to Q200 level of +744.20m geodetic elevation) are expected during the spring “wet” season.

4.0 SEISMIC CONSIDERATION

It is understood that the structure is categorized as “Major Route Bridges”, in accordance with CAN/CSA-S6-19 (S6-1) and BC Supplement to S6-19.

4.1 Firm Ground PGA and Uniform Hazard Response Spectrum

Based on interpolation from the Natural Resources Canada webpage, the “2015 National Building Code Seismic Hazard Calculation” values of peak ground acceleration (PGA), the Uniform Hazard Response Spectrum spectral acceleration values (for 5% damping factor) at the hypothetical “Near-Surface Firm Ground” for the site are given below:

Table 2: Firm Ground PGA and Uniform Hazard Response Spectrum

Seismic Event	PGA (g)	PGV (m/s)	Sa (0.2s)	Sa (0.5s)	Sa (1.0s)	Sa (2.0s)	Sa (5.0s)	Sa (10.0s)
475-year Return Period (10% probability of exceedance in 50 years)	0.074	0.101	0.166	0.142	0.088	0.053	0.016	0.0059
975-year Return Period (5% probability of exceedance in 50 years)	0.106	0.151	0.233	0.197	0.127	0.079	0.028	0.0094
2475-year Return Period (2% probability of exceedance in 50 years)	0.160	0.234	0.345	0.284	0.189	0.122	0.048	0.016

4.2 Site Class for Seismic Site Response

The SPT blow counts obtained in test holes (BH18-01 and BH18-02) indicated that the average SPT penetration resistance of the natural sand and gravel to sandy gravel is greater than 34 blows per 300mm penetration; therefore, the site may be classified as "Site Class D" as per S6-19 Table 4.1. The site coefficients F(T), F(PGA), and F(PGV) shall be calculated in accordance with Table 4.2 to Table 4.9 presented in CHBDC S06-19, Section 4.4.3.3. for "**Site Class D**".

Based on CAN/CSA-S6-19 (S6-1) Table 4.10, the proposed structure is classified as seismic performance category 2.

4.3 Soil Liquefaction

As discussed above, the static groundwater table is expected to fluctuate between geodetic elevations of approximately +741m and +744.2m. The existing natural deposit of sand and gravel to sandy gravel underlying the site was noted to be dense to very dense with the SPT resistance blow counts all greater than 34 blows per 300mm penetration. The natural deposit of dense to very dense sand and gravel to sandy gravel is not susceptible to liquefaction under design earthquakes.

5.0 PILE DESIGN

Based on the structural design drawings prepared by MoTI, it is understood that the proposed new single-span bridge structures will be supported on two abutments on pile foundations. 610mm (24 inches) diameter steel pipe piles with a wall thickness of 15.9mm (5/8 inch) are proposed to support the abutments.

All piles are expected to be driven open-ended with an impact hammer. The pile cap of the abutments will be partially buried with pile cut-off elevations varying between +743.9m and +744.8m. The inside of the abutment piles will be filled with reinforced concrete for the top portion with a length to be determined as shown on the design drawings. If the soil plug formed inside the pipe pile is not long enough, additional granular fill will be required to fill over the soil plug up to the bottom elevation of the design reinforced concrete to be placed. Based on the structural drawings, the factored Maximum Pile Design Load is **1,800 kN.**

In accordance with Canadian Highway Bridge Design Code (CHBDC S06-19) and BC MoTI Bridge Standards and Procedures Supplement to CHBDC S06-19 the minimum required resistance factors of the deep foundation design with typical degree of understanding are summarized in Table 3 below:

Table 3 Deep Foundation Resistance Factors

Foundation Type	Limit State	Test Method/Model	Resistance Factor, with Typical Degree of Understanding *
Deep Foundation	Compression	Static Analysis	0.4
		Static Loading Test	0.6
		Dynamic Analysis	0.4
		Dynamic Test	0.5
	Tension	Static Analysis	0.3
		Static Test	0.5
	Lateral	Analysis	0.5
		Static Test	0.5

* ULS consequence factor of 1.0 (typical degree of understanding) has been used in accordance with S6-19 Table 6.2.

5.1 Axial Pile Resistance

SPT penetration resistance blow counts were used to estimate pile axial capacity. This is considered to be a Static Analysis and accordingly a Resistance Factor with a typical degree of understanding of 0.4 should be applied. Using a resistance factor of 0.4, the required minimum ultimate axial compressive resistance is **4,500 kN**.

Subsurface soil conditions and laboratory testing outlined in the Factual Geotechnical Report were reviewed to develop the parameters used in estimating the axial capacities and lateral resistance of piles. General soil parameters used for axial pile capacity estimations are summarized in Table 4 below.

Table 4 - Soil Parameters at Abutments, Used for Axial Pile Capacity Estimate

Material	Depth (m)	Unit Weight of Material	Angle of Friction	Cohesion	β Coefficients	N_t Factors
Fill (Sand and Gravel)	0 - 4	20 kN/m^3	38°	-	0.6	90
Sand and Gravel to Sandy Gravel	4 - 11	20 kN/m^3	38°	-	0.8	100
	11 - 17	20 kN/m^3	38°	-	0.8	150
Sandy Gravel	> 17	21 kN/m^3	38°	-	0.8	150

* Q200 Water Table at +744.20m, geodetic.

The ultimate axial capacities of the piles were estimated using the effective stress (Beta) method outlined in Canadian Foundation Engineering Manual 4th Edition, 2006. The ultimate pile capacities are estimated using both skin friction and end-bearing and assuming a plug will form at the bottom of pile to develop end bearing.

The results of the axial pile capacity estimates are presented in Appendix A and summarized in Table 5 below:

Table 5 - Pile Capacity and Length Estimate

	Factored Ultimate Pile Compression Capacity with Geotechnical Resistance Factor of 0.4	Minimum Pile Embedment Depth below underside of Pile Cap
Abutment	4,500 kN	20.0 m

Please note that the pile needs to be driven to effective termination/refusal criteria and possibly PDA testing will be necessary to confirm that the required ultimate pile axial capacity is achieved. Termination/Refusal criteria are dependent on many factors such as pile type, pile size, length, wall thickness, soil type, design load, driving equipment, driving energy, and hammer efficiency which are discussed in Section 5.3 below and should be confirmed once the detailed construction information is available.

5.2 Pile Lateral Resistance

The pile lateral resistance analysis was conducted using a commercially available computer software program, LPILE (2019 Version 11.01) by Ensoft Inc. L-Pile input soil parameters are presented in Table 6 below:

Table 6 Snass Creek Bridge Replacement L-Pile Input Parameters – Abutment

Layer No.	Soil Layer	Soil Model	Layer Depth (m)	Effective Unit Weight (kN/m^3)	Friction Angle (deg.)	K (kN/m^3)
1	Compact to dense Sandy Gravel Fill	API Sand	0-1	20	36	24,400
2	Compact to dense Sandy Gravel Fill	API Sand Submerged	1-4	10.2	36	24,400
3	Dense Sand and Gravel	API Sand Submerged	4-11	10.2	36	34,000
4	Dense Sandy Gravel to Sand and Gravel	API Sand Submerged	11-25	10.2	38	34,000

* Q200 Water Table +744.2m, geodetic.

Soil response on the piles is modeled by non-linear elastic springs (“p-y”) attached to the pile elements which are independent of the pile size and length. The P-Y curves are attached in Appendix A and the actual data required for the structural assessment are provided separately.

Piles supporting each abutment will be arranged in a single row transverse to the bridge direction. Closely spaced piles may have a significant group interaction effect. It is recommended to apply an appropriate p-multiplier, as shown in Table 7 below, to the equivalent k-value (soil spring).

It is understood that the pile fixity depths had been checked and reviewed by the structural Engineer of record.

Table 7 Pile Group P-Multiplier

Pile Spacing (c-c Diameters)	P-Multiplier Transverse
2	0.78
3	0.85
4	0.91
≥5	1.00

5.3 Pile Drivability

The pile drivability is assessed using a commercially available computer software program, GRLWEAP Version 2010-1 by GRL Engineers Inc. The preliminary GRLWEAP analysis results are presented in Table 9 below.

Table 9 GRLWEAP Analyses Results

Pile Type	Minimum Embedment Depth (m)	Ultimate Capacity (kN)	Max. Compression Stress (MPa)	Blow Counts blow/25mm	Effective Energy (kN-m)
Φ610mm Abut. Pile	20.0	4,500	248.3	3.5	83.3

In summary, with a minimum effective hammer impact energy of about 83.3 kN-m and a final set of minimum 3.5 blows over the last 25mm (or maximum 7.1mm per blow) at a total pile embedment depth of 20m, the 610mm diameter abutment piles will achieve the required factored geotechnical ultimate pile resistance of 4,500 kN.

It should be noted that the maximum compression stress near the end of initial driving may exceed the steel's yield strength. Depending upon the strength of the steel material, thickening of wall of the pile top may be required to prevent pile top damage due to the hammer impact.

If the final set cannot be achieved at the minimum required pile embedment depths, or the final set is achieved at a shallower pile depth, pile driving should be terminated. High strain dynamic testing, such as Pile Dynamic Analyzer (PDA) test, is recommended to confirm the final axial pile resistances. The dynamic test can allow for the use of a larger geotechnical resistance factor of 0.5. In this circumstance, it is recommended to allow a minimum setup period of 3 days before the dynamic test. The PDA testing results should be submitted to the geotechnical engineer to review and to determine if the design pile capacity has been achieved or if the pile is required to be driven deeper or can be terminated.

Please note, the pile drivability should be reassessed once the detailed information of pile driving hammer, driving assembly and methodology are made available prior to mobilization to site.

5.4 Construction Considerations

As mentioned above, the site is underlain by dense to very dense sand and gravel to sandy gravel to a depth of beyond the anticipated pile tip. Both existing fill and natural soils contain cobbles and boulders which may cause difficulties for pile installation. Driving shoes are recommended to prevent pile tip damage during pile driving. Depending upon the size of boulders, additional drilling or rock splitting may be required to advance the piles during pile driving. The pile installation contractor shall be fully aware of the potential obstacles and have suitable equipment on standby on-site that is capable of advancing the piles through cobbles and boulders.

Driving piles has the potential to cause vibrations and/or localized ground movements. Depending on the sensitivity of the existing structure, it may be necessary to estimate peak particle velocities for the preferred pile configuration and hammer/installation method to assess potential impacts on the existing bridge and identify potential mitigation measures. It is logical to make the contractor responsible for this given they will be in control of the staging details.

6.0 EMBANKMENT DESIGN

All permanent embankment and stability berm fills should be designed with 2H:1V or flatter slope inclination. The under-bridge portion of the abutment slope should be filled using Bridge End Fill (BEF) and can be sloped up to 1.5H:1V if the slope surface is finished using riprap. Where the riverbank slope requires riprap protection it should be sloped at 1.5H:1V or flatter.

In accordance with MoTI's Standard Specifications for Highway Construction 2020, Bridge End Fill (BEF) should be used in the zone behind the bridge abutments. This zone should extend to the bottom of the abutments and horizontally minimum 8 m behind the abutment. The BEF should taper into granular embankment fill at a 1.5H:1V or flatter transition slope.

Placement and compaction of BEF shall meet the requirements outlined in MoTI's Standard Specifications for Highway Construction 2020, SS202. Granular embankment fill should comprise clean, well-graded, sand and gravel with less than 5% fines content (percent passing No. 200 sieve). All granular embankment fill should be placed in maximum 300 mm thick lifts and compacted using a ride-on vibratory roller to a minimum of 95% of the material's Standard Proctor Maximum Dry Density (SPMDD).

It is recommended that the slope design follow the Manual of Control of Erosion and Shallow Slope Movement by MoTI. Runoff from the bridge and road surface should not be discharged onto the earth slopes. This is particularly important at the ends of the bridge where, without collection, high runoff may erode the abutment fill slope surfaces.

Clearing and stripping is required under the new road embankment alignment, defined as the 2H:1V projection from the crest of the embankment. Prior to embankment fill placement, any soft, wet, loose, or other unsuitable materials, including organics, should be removed, and replaced with compacted granular embankment fill. The exposed granular subgrade should be proof rolled with a large smooth drum vibratory roller prior to fill placement.

6.0 PAVEMENT DESIGN

Considering the new pavement structure will be supported on compacted BEF or granular embankment fill, the following minimum pavement structure is recommended at the new alignment:

- 125mm thick – Asphalt
- 300mm thick – 25mm Well Graded Base course (WGB)
- 300mm thick – 75mm Selected Granular Subbase course (SGSB)

Based on the contemplated shift alignment and increase in grades, the existing asphalt can be left in place provided that the full depth of the recommended pavement structure, as discussed below, fits above it. Where the recommended pavement structure intersects the existing structure, the existing asphalt should be removed entirely. Sandwiched asphalt pavement is not permitted.

7.0 CLOSURE

This design report has been prepared for this specific site with the specific design objectives conveyed by MoTI.

If any of the assumptions are not deemed acceptable by the structural engineer, it is recommended that the Geotechnical Engineer be consulted.

Prepared by:

2023-10-17

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Enclosures: Appendix A – Pile Design Calculations
Appendix B – Factual Geotechnical Report by MoTI

Appendix A

2015 National Building Code Seismic Hazard Calculation

Pile Design Calculations:

Vertical Axial Pile Capacity

L-Pile P-Y Curve

2015 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

October 10, 2018

Site: 49.2293 N, 121.0587 W User File Reference: Highway

Requested by: ,

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.189	0.278	**0.345**	0.329	**0.284**	**0.189**	**0.122**	**0.048**	**0.016**	**0.160**	**0.234**

Notes. Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. 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 directly 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(0.05)	0.034	0.084	0.122
Sa(0.1)	0.051	0.125	0.181
Sa(0.2)	0.072	0.166	0.233
Sa(0.3)	0.074	0.165	0.228
Sa(0.5)	0.063	0.142	0.197
Sa(1.0)	0.037	0.088	0.127
Sa(2.0)	0.021	0.053	0.079
Sa(5.0)	0.0054	0.016	0.028
Sa(10.0)	0.0021	0.0059	0.0094
PGA	0.030	0.074	0.106
PGV	0.039	0.101	0.151

References

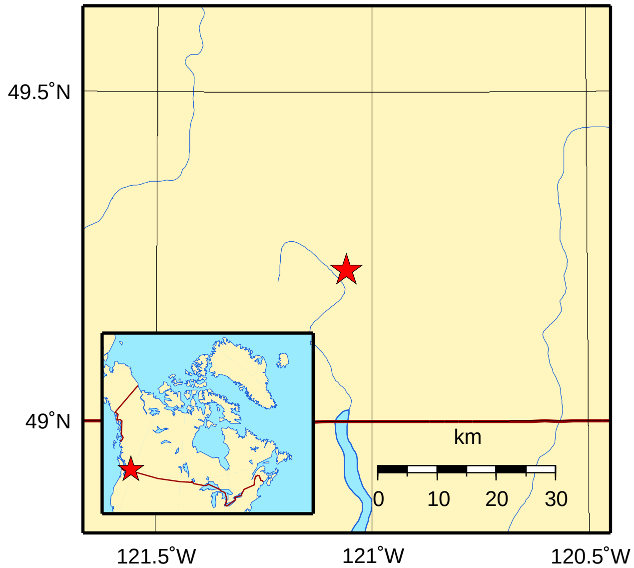
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

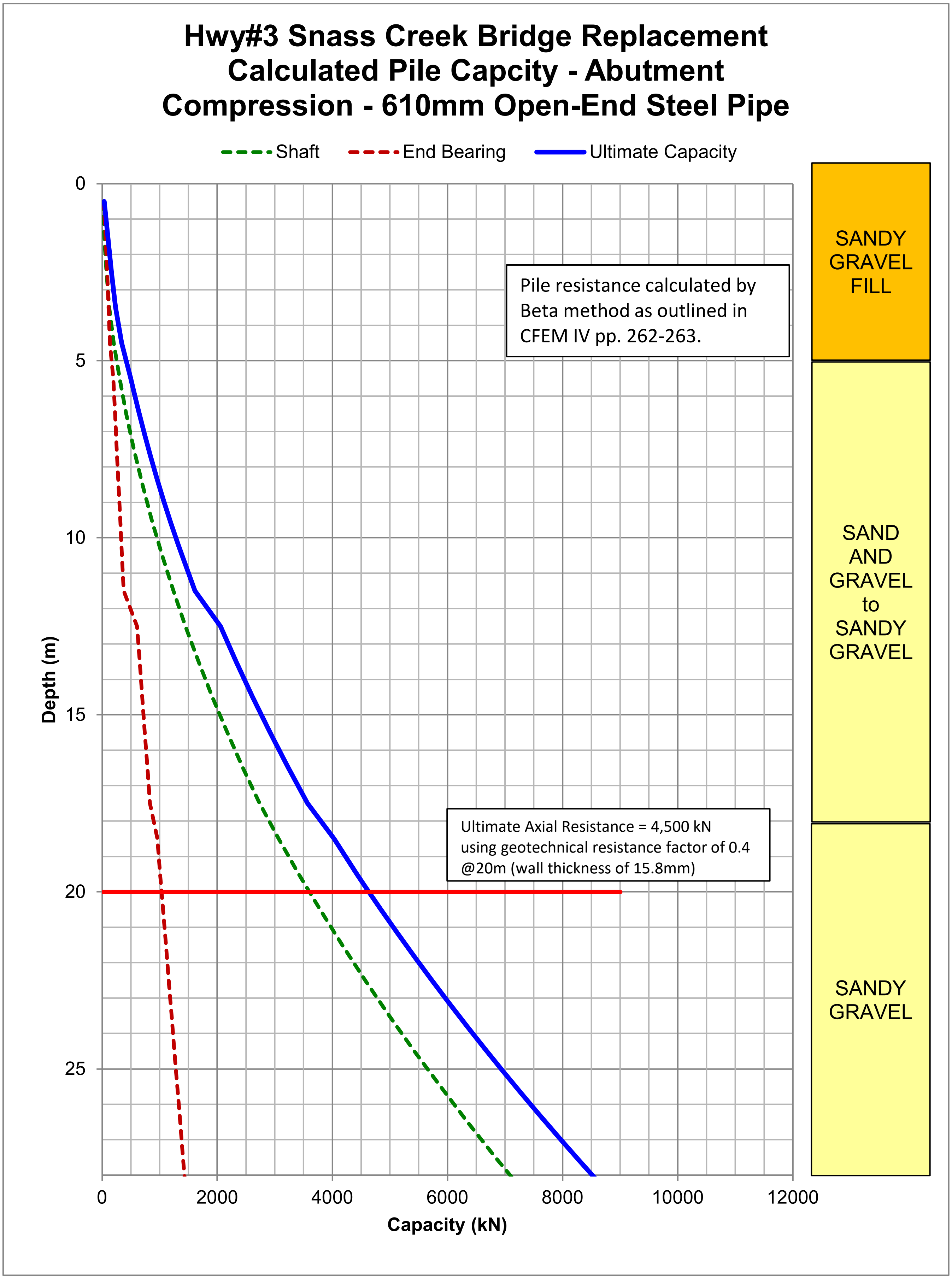
User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

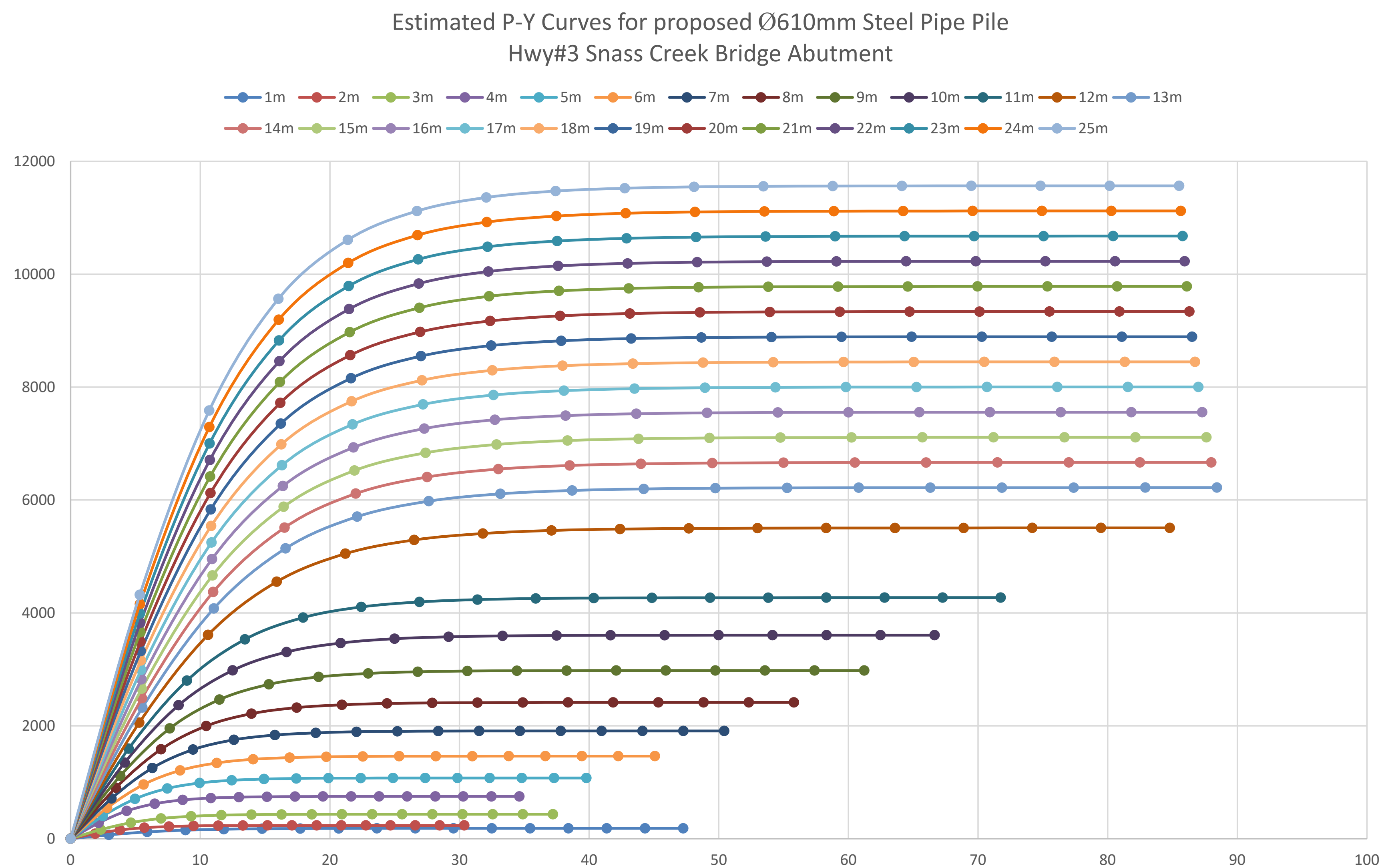
See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français





Hwy#3 Snass Creek Bridge Replacement
Calculated Pile Capcity - Abutment
Compression - 610mm Open-End Steel Pipe
Shaft
End Bearing
Ultimate Capacity
Pile resistance calculated by Beta method as outlined in CFEM IV pp. 262-263.
Ultimate Axial Resistance = 4,500 kN using geotechnical resistance factor of 0.4 @20m (wall thickness of 15.8mm)
Depth (m)
0
5
10
15
20
25
0
2000
4000
6000
8000
10000
12000
Capacity (kN)
SANDY GRAVEL FILL
SAND AND GRAVEL to SANDY GRAVEL
SANDY GRAVEL


Estimated P-Y Curves for proposed Ø610mm Steel Pipe Pile
Hwy#3 Snass Creek Bridge Abutment
1m 2m 3m 4m 5m 6m 7m 8m 9m 10m 11m 12m 13m
14m 15m 16m 17m 18m 19m 20m 21m 22m 23m 24m 25m
12000
10000
8000
6000
4000
2000
0
0 10 20 30 40 50 60 70 80 90 100

Appendix B

Factual Geotechnical Report by MoTI

Factual Geotechnical Report