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Ministry of Transportation and Infrastructure

TECHNICAL MEMORANDUM

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CC Bill Rose, P.Eng.
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FROM Eric Constantinescu, P. Eng.
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Re: Geotechnical Memorandum 2 – Geotechnical Design Parameters, Highway 16 Mile 28 Crossing

This memorandum outlines the design parameters and methods used for the geotechnical design of the embankment approaches, retaining walls and overhead structures that are part of the Highway 16 Mile 28 Crossing project. The project site is located on Highway 16, approximately 45 km west of Terrace BC.

Geotechnical analysis was based on the results of the site investigation and laboratory testing works outlined in Geotechnical Memorandum 1, issued by the BC Ministry of Transportation and Infrastructure (MoTI), and dated June 29, 2016.

1.0 GEOTECHNICAL DESIGN AND ANALYSIS

Geotechnical analyses have been carried out at the 50% design level, and are underway to a 70% design level. The 50% design was submitted and the Value Engineering (VE) review was completed the week of June 13, 2016. The VE draft report was provided June 29, and the 50% design review was carried out on June 30. The 70% design is projected to be completed in the third week of August.

Geotechnical roles and responsibilities for the project are as follows:

- Geotechnical design of the bridge approaches and pile foundations is being conducted by Golder Associates Ltd., Maureen A. Kelly, P.Eng. is the Engineer of Record.
- Geotechnical design of the permanent rock slopes and rockfall catchment is being conducted by Golder Associates Ltd., Charlie Harrison, P.Eng. is the Engineer of Record.
- Geotechnical design of the pavement structure, and embankment slopes outside of the overpass and review of the geotechnical design by Golder is being completed by the MoTI, Eric Constantinescu, P.Eng. is the Engineer of Record.
- Mr. Donald Gillespie, P.Eng. of TetraTech Engineering Ltd. was the geotechnical VE reviewer.

2.0 50 PERCENT DESIGN AND ANALYSIS

Geotechnical analyses carried out for the 50% design included settlement estimates for the overpass approach, static slope stability analysis for the embankment slopes, and seismic induced liquefaction assessment of the foundation soils at the east approach and abutment.

Analysis was carried out in accordance the Canadian Highway Bridge Design Code (S6-14), using the 2015 National Building Code of Canada (NBCC) seismic ratings. As directed, the structure was rated by the MoTI as “Other” and as such, the design was required to maintain Life Safety under the 1:2,475 year design earthquake (2 percent chance of exceedance in 50 years). The structure, including approaches and walls, was required to meet a 75 year design life.

Numerical methods to determine post-earthquake deformation was not required under the design code for this site, and was not carried out for this project.

2.1 Settlement Estimates for 50 Percent Design

Settlement of the deep clay layer locally present was estimated using Settle3D software for locations along both the highway and the CN Rail embankment extending east from the east abutment of the crossing.

Post construction settlement analysis considered both the elastic settlement of granular soils, as well as one-dimensional consolidation of the clay soils, the latter being based on the Compression Index (Cc) values obtained from laboratory consolidation testing.

Measurements of porewater pressures were obtained from vibrating wire piezometers installed at the site.

Settlement analysis was conducted using the following parameters:

Table 1 – 50 Percent Design Parameters for Settlement Analysis

Material	Unit Weight (kN/m^3)	Elastic Modulus of Settlement	Compression Index (C_c)	Initial Void Ratio (e_0)
Embankment Fill	19	-	-	-
Sand	19	10500	-	-
Gravel	20	120750	-	-
Clay	18	-	0.15	0.818

2.2 Slope Stability for 50 Percent Design

Two-dimensional slope stability analysis was carried out using SlopeW software using the General Limit Equilibrium (GLE) method, and was conducted for both the current “as-is” site conditions, as well as under the proposed embankment and approach configurations at the abutment (Sta. 1057+70) and east approach (1058+00).

Slope stability analysis was carried out under both static and earthquake loading conditions. Analysis for the static condition assumed undrained conditions for the clay using a tau-sigma ratio based on the OCR values obtained from consolidation testing and in situ testing.

Following 2015 NBCC, and based on shear wave velocities obtained from the site, the site was assessed to be Site Class “D”. An amplification factor of 1.3 was applied to the NBCC accelerations provided for Site Class “C” to obtain Site Class “D” values.

Psuedo-static slope stability conditions were assessed using a ½ PGA surface acceleration for the 1:2,475 earthquake event (2 percent probability of exceedance in 50 years).

Shear strength values were inferred from field investigation and laboratory test results. To account for the upwards gradient in the clay stratum measured in the piezometers, strength values for the clay were based on a shear strength to effective stress (τ-σ’) ratio of 0.13. This is lower than typically anticipated for a normally consolidated clay. Parameters assumed for the slope stability analyses are presented in in Table 2, below.

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Table 2 – 50 Percent Design Parameters for Slope Stability Analysis

Material	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle ϕ (degrees)	Tau-Sigma Ratio
Embankment Fills	19	0	35	-
Non-liquefied Sand	19	0	34	-
Liquefied Sand	19	-	-	0.1
Gravel	20	0	38	-
Clay	18	-	-	0.13
Compaction Piles	20	58	34	-

2.3 Seismic Stability for 50 Percent Design

Potential liquefaction of the loose, saturated sand layer identified underlying the east abutment and approach was initially assessment by MoTI using SPT data following Seed Simplified Methods. This assessment indicated that liquefaction would occur under the design level earthquake, and detailed assessment of the liquefaction hazard was required and subsequently carried out by Golder, using site specific CPT data.

The liquefaction analysis was carried out using Idriss and Boulanger (2008) and Finn (2014) methods. A FoS of 1.2 was used to assess the liquefaction potential. in accordance with suggested methods (1.0 to 1.3 from 2003 Guidelines for the Seismic Design of Highway Bridges MCEER/ATC-49, Ashford et al. 2011 and Martin et al. 1999). The magnitude used for liquefaction analyses were determined from Magnitude-Distance deaggregation plots for PGA provided by the NBCC. Zones of liquefiable under 1:2,475 seismic event were identified at 1057+00 (9.5 m thick) and at 1058+00 (4 m thick). Liquefaction of the sand at 1058+60 was isolated to localized thin layers and not considered to pose a significant risk for flowslide failure.

Residual strength values were corrected to equivalent clean sand conditions were at 10 percent of the in-situ effective vertical stress for the liquefiable sand layer. Slope stability analyses were then carried out for the post-liquefaction condition with the residual strength in the sand layer.

Vertical settlement due to liquefaction was estimated using Ishihara and Yoshimine (1992) re-presented by Idriss and Boulanger (2008).

2.4 Slope Stability Results and Criteria for 50 Percent Design

Slope stability analyses were conducted for the three conditions described, based on the above input parameters and methods. The following table summarizes the design targets and calculated factors of safety.

Table 3 – Slope Stability Results for 50 Percent Design

Condition	Factor of Safety			Embankment Instability Condition
	Minimum Required	As is Condition	With Fill Proposed	
Static stability	> 1.5	n/a	1.8	Both sides stable (1057+70 and 1058+00)
Seismic stability with pseudo-static loading (no liquefaction)	>1.1	n/a	1.4	Both sides stable (1057+70 and 1058+00)
Stability after seismic event (with liquefaction)	>1.0	0.9	0.6	South unstable (1057+70), both sides unstable at 1058+00
Ground Improved stability after seismic event (with liquefaction)	>1.0	n/a	1.0	Both sides stable (1057+70 and 1058+00)

Both the static and pseudo-static conditions exceed the minimum design level factor of safety; however the strain-softened post-earthquake condition fails to meet the minimum factor of safety of 1. For this condition, a factor of safety below 1 indicates a flowslide condition with anticipated lateral ground motions in excess of 2 m.

In order to achieve the minimum factor of safety of 1, ground improvement would be required. After review of potential methods, ground improvement measures were proposed for the site:

- Infilling of the topographic depression and pond to the north (CN Rail side) of the embankment beside the east approach with rock fill to a minimum elevation in order to buttress and prevent propagation of a rupture surface.

- Create a ground improvement zone along the south (Skeena River side) of the embankment through the installation of timber compaction piles prior to filling.
- Construction of the approach using mechanically stabilized earth walls on both the north and south sides that would maintain internal integrity during lateral displacement in order to meet "Life Safety" conditions.

Based on the input parameters outlined in Table 2, the ground improvement increased the factor of safety to above the minimum 1.0 required for the design. Lateral displacements were then estimated using the Newmark sliding block method for the liquefied strength yield condition of the soil.

3.0 VE and 70 Percent Design Analysis

Following the VE review in June, 2016, several questions and suggestions were raised regarding the geotechnical design and analysis. Specific to the crossing, these included

- Optimization of the conventional fill over retaining wall;
- Increase the buttress fill proposed for the south embankment toe (above the ground improvement zone) for enhanced stability;
- Assessment of the lateral ground movement of the east abutment
- Revision of the FoS for triggering liquefaction from 1.2 to 1.0 as stability analysis are also factored; and
- Review of the compression index used in the settlement calculations.

The revised analyses were carried out as suggested by the VE team, and all sections were analyzed using the increased rock fill buttress zones.

Lateral displacements in the longitudinal direction (perpendicular to the primary direction of lateral spreading) were estimated at 20 percent of the estimated lateral spreading displacements (Tokimatsu and Asaka as per Pacific Earthquake Engineering Research Center PEER 2011/04).

Revising the Factor of Safety for triggering liquefaction to 1.0 resulted in a reduction of the extent of liquefiable soils, and corresponding reduction in the extent of both the retaining wall required on the south approach, and the ground improvement zone. Lateral displacement and loading estimates were similarly reduced.

Consolidation settlements were estimated using a range of C_c values between 0.15 and 0.23 which based on both the consolidation laboratory test results, as well as correlations of liquid limit values.

4.0 CLOSURE

A review of AREMA requirements is underway to verify that CN Rail conditions will be met for the crossing structure.

Pile design, including lateral pile analyses, and internal stability of the retaining walls on site are currently underway, and will be addressed in future memoranda.

We trust that this summary of analysis methods provides the information you require at this time. Should you have any further questions or require any additional information on the assessment works carried out to date, please do not hesitate to contact the undersigned.

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