Appendix 1.0-AB SRK, Tenas Preliminary Liquefaction Potential Assessment

TELKWA COAL
LIMITED

SRK Consulting (Canada) Inc.
1066 West Hastings Street, Suite 2200
Vancouver, BC V6E 3X2
Canada

+1 604 681 4196 office
+1 604 687 5532 fax

vancouver@srk.com
www.srk.com

Memorandum

Tenas' Preliminary Liquefaction Potential Assessment

June 7, 2021

To	Dan Farmer
From	Carmelo Galindo, Santiago Pastine, Calvin Boese
Cc	—
Subject	SPT-based Liquefaction Potential Assessment on Previous Investigations (prior to 2021)
Client	Telkwa Coal Ltd.
Project	2CT025.008

1 Introduction

The Tenas Project is in central British Columbia, approximately 15 km south of Smithers and 2 km southwest of Telkwa. The Project site is located south of the Tenas River and the boundaries of the Project encompass the lower reaches of the Telkwa River, Goathorn Creek, and Pine Creek. The Tenas Project's total mine life is anticipated to be 22 years, including one year of pre-stripping and construction followed by 21 years of production. The mine plan was designed and scheduled in a manner that allows potentially-acid generating (PAG) rock to be deposited into Management Ponds (MP), including the East Management Pond (EMP), West Management Pond (WMP), and North Management Pond (NMP).

Telkwa Coal Ltd. (TCL) has retained SRK Consulting (Canada) Inc. (SRK) to provide engineering services to complete a geotechnical assessment of the Tenas Control Pond (TCP) and MP for the Tenas Project at a feasibility level (SRK, 2021). The scope of work included interpretation of historical field and laboratory campaigns, determination of soil stratigraphy, strength of materials, and a slope stability assessment of the TCP and MP embankments.

As part of the conclusions, SRK recommended that the potential for liquefaction of the soil foundation may be assessed based on the available field and laboratory testing data. The purpose of this memorandum is to provide a preliminary assessment of the potential for liquefaction of the TCP and MP's soil foundation and provide recommendations for the detailed engineering stage.

2 Liquefaction Potential Assessment

2.1 Previous Investigations

Foundation conditions for previous designs of the TCP and MP were studied by SRK in 2018 (SRK, 2018) and 2019 (SRK, 2019a). The previous investigations included 13 geotechnical sonic drill holes in 2018 and 12 geotechnical sonic drill holes in 2019 carried out to investigate material foundation at a maximum depth of 30.8 m and 67.1 m, respectively, including SPT tests at 3.0 m intervals, sampling and logging according to the Unified Soil Classification System (USCS).

The liquefaction potential of the soil foundation was assessed following the (Idriss & Boulanger, 2008) methodology based on the SPT data.

2.2 SPT-based methodology

The assessment of soil liquefaction characteristics proposed by Idriss & Boulanger (2004, 2008) is presented below.

The stress-based approach for evaluating the potential for liquefaction triggering, initiated by Seed & Idriss (1967), compares the earthquake-induced cyclic stress ratios (CSR) with the cyclic resistance ratios (CRR) of the soil. The soil's CRR is usually correlated to an in-situ parameter such as CPT penetration resistance, SPT blow count, or shear wave velocity, V_s.

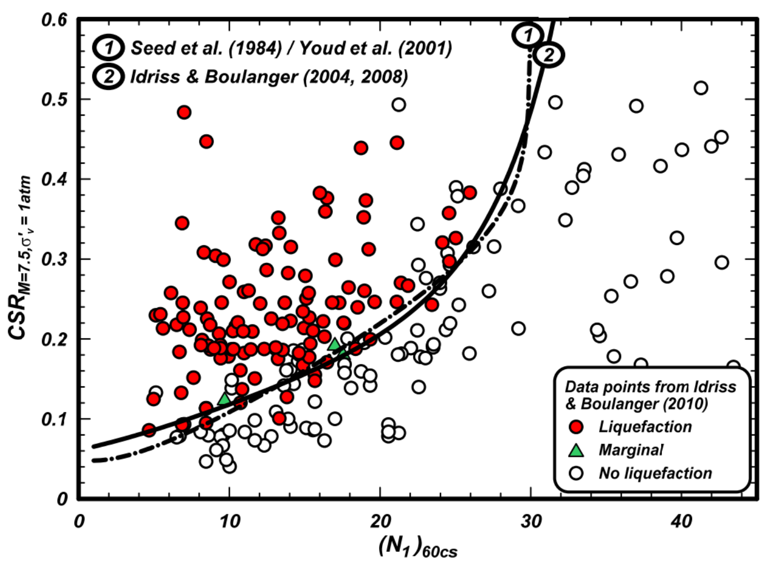


Figure 2-1: Examples of SPT-based liquefaction triggering curves with a database of case histories processed with the (Idriss & Boulanger, 2008) procedure

Earthquake-induced cyclic stress ratio (CSR)

The earthquake-induced CSR, at a given depth z within the soil profile, is usually expressed as a representative value equal to 65% of the maximum CSR:

$$CSR_{M,\sigma_v'}\,[-] = 0.65 \cdot \frac{\tau_{max}}{\sigma_v'}$$

Where:

- $\tau_{max}\,[kPa]$: maximum earthquake induced shear stress;
- $\sigma_v'\,[kPa]$: vertical effective stress.

The value of τ_{max} can be estimated using the equation developed as part of Seed & Idriss Simplified Liquefaction Procedure, i.e.:

$$CSR_{M,\sigma_v'}\,[-] = 0.65 \cdot \frac{\sigma_v}{\sigma_v'} \cdot \frac{PGA}{g} \cdot r_d$$

Where:

- $\sigma_v\,[kPa]$: vertical total stress;
- $PGA/g\,[-]$: maximum horizontal acceleration (as a fraction of gravity) at the ground surface;
- $r_d\,[-]$: shear stress reduction factor.

Cyclic resistance ratio CRR and penetration resistances

The soil's CRR is correlated to SPT penetration resistance after application of procedural and overburden stress corrections. SPT blow counts are corrected for overburden stress effects as:

$$(N_1)_{60}\,[-] = C_N N_{60}$$

Where:

- $(N_1)_{60}\,[-]$: penetration resistance obtained in the same sand at an overburden stress of 1 atm;
- $C_N\,[-]$: overburden correction factor;
- $N_{60}\,[-]$: penetration resistance obtained at an overburden stress of 1 atm.

CRR is dependent on the duration of shaking (which is expressed through an earthquake magnitude scaling factor, MSF) and effective overburden stress (expressed through a K_σ factor), i.e.:

$$CRR_{M,\sigma_v'}\,[-] = CRR_{M=7.5,\sigma_v'=1} \cdot MSF \cdot K_\sigma$$

The correlation of CRR to $(N_1)_{60}$ in cohesionless soils is also affected by the soil's fines content. This correlation can also be expressed in terms of an equivalent clean-sand $(N_1)_{60cs}$ values:

$$(N_1)_{60cs}\,[-] = (N_1)_{60} + \Delta(N_1)_{60}$$

The equation for the SPT-based correlation is as follows:

$$CRR_{\mathrm{M=7.5},\ \sigma'_v=1\mathrm{atm}}\ [-] = \exp\left[\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right]$$

Shear stress reduction coefficient r_d

The shear stress reduction coefficient r_d accounts for the dynamic response of the soil profile, based on the depth below the ground surface z in meters, and is expressed as:

$$r_d\ [-] = \exp[\alpha(z) + \beta(z) \cdot M]$$

Where:

- $\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right)$
- $\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)$
- M $[-]$: Earthquake moment magnitude

Overburden correction factor C_N

C_N is the normalization factor at an effective reference pressure of $100\ kPa$, and is expressed as:

$$C_N = \left(\frac{P_a}{\sigma'_v}\right)^m \leq 1.7$$

Where:

- $m = 0.784 - 0.0768\sqrt{(N_1)_{60cs}}$
- P_a $[-]$: atmospheric pressure.

Overburden correction factor K_σ

The overburden correction factor K_σ recommended to consider the effective pressure σ'_v is:

$$K_\sigma\ [-] = 1 - C_\sigma \ln\left(\frac{\sigma'_v}{p_a}\right) \leq 1.1$$

Where:

- $C_\sigma\ [-] = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60cs}}} \leq 0.3$

The coefficient C_σ can be limited to its maximum value of 0.3 by restricting $(N_1)_{60cs} \leq 37$ for use in these expressions.

Magnitude scaling factor MSF

The magnitude scaling factor (MSF) is used to account for duration effects (i.e., number and relative amplitudes of loading cycles) on the triggering of liquefaction. The MSF for sands used by Idriss & Boulanger (2008) was developed by Idriss (1999) who derived the following relationship:

$$MSF\ [-] = 6.9 \cdot \exp - \frac{M}{4} - 0.058 \leq 1.8$$

An upper limit for the MSF is assigned to very-small-magnitude earthquakes for which a single peak stress can dominate the entire time series.

Equivalent clean sand adjustment for fines content

The liquefaction case histories suggest that the liquefaction triggering correlations shift to the left as the fines content (FC) increases. The equivalent clean sand adjustment, $\Delta(N_1)_{60}$, is thus empirically derived from the liquefaction case history data, and account for the effects that fines content have on both the CRR and the SPT penetration resistances. The expression for SPT is as follows:

$$\Delta(N_1)_{60}\ [-] = \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.77}{FC + 0.01}\right)^2\right]$$

2.3 Input data

2.3.1 Field and Laboratory Tests

All the borehole data from the 2018 and 2019 field and laboratory tests were used in the analysis of the potential for liquefaction triggering of soils.

The overburden material is composed by a very stiff to hard sandy lean clay (CL), with low to medium plasticity, with intervals of fat clay (CH), clayey silty sand (SC or SM) or gravel (GC or GM) at various depths. SPT values typically range from 20 to 40 for lean clay (CL) and fat clay (CH) for depths from 0 m to 20 m, as shown in Figure 2-22-2.

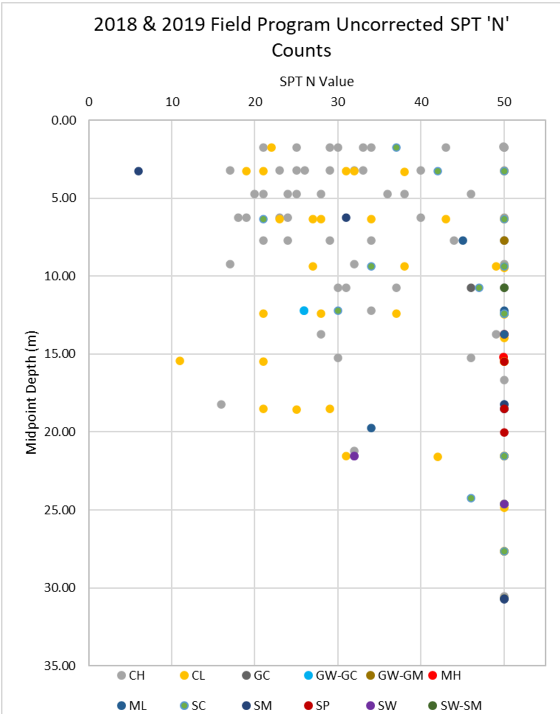


Figure 2-2: 2018 and 2019 SRK Field Campaign. SPT Values vs. Depth.

The results of Particle Size Distribution (PSD) tests are presented in Figure 2-32-3 and Figure 2-4. These charts show that the samples have a fines content of 10% - 95% with a typical sand content of 20% - 40% and a gravel content of 0% - 20%. PSDs obtained in 2019 were consistent with the ones obtained during 2018.

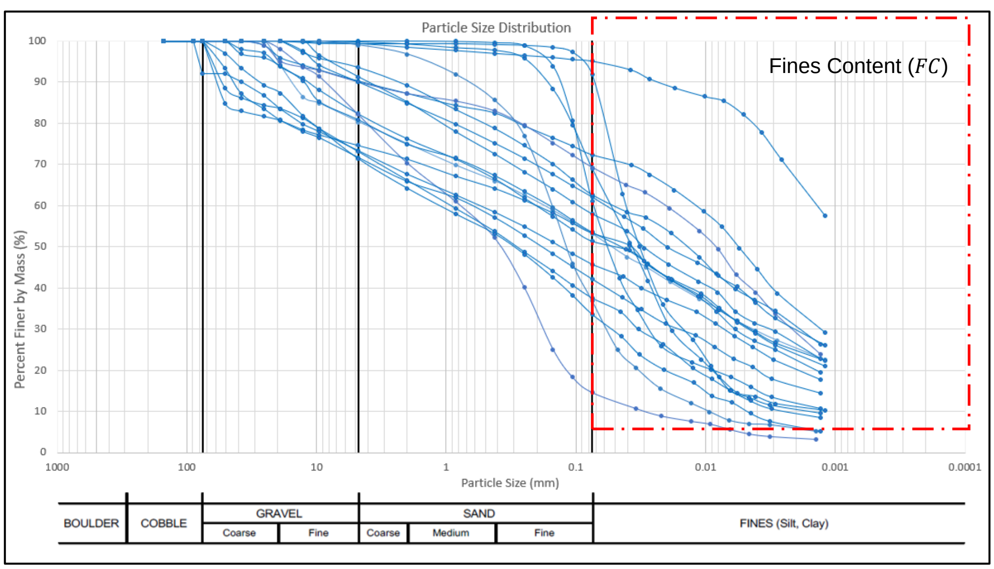


Figure 2-3: 2018 SRK Field Campaign. PSD Curves (SRK, 2019a).

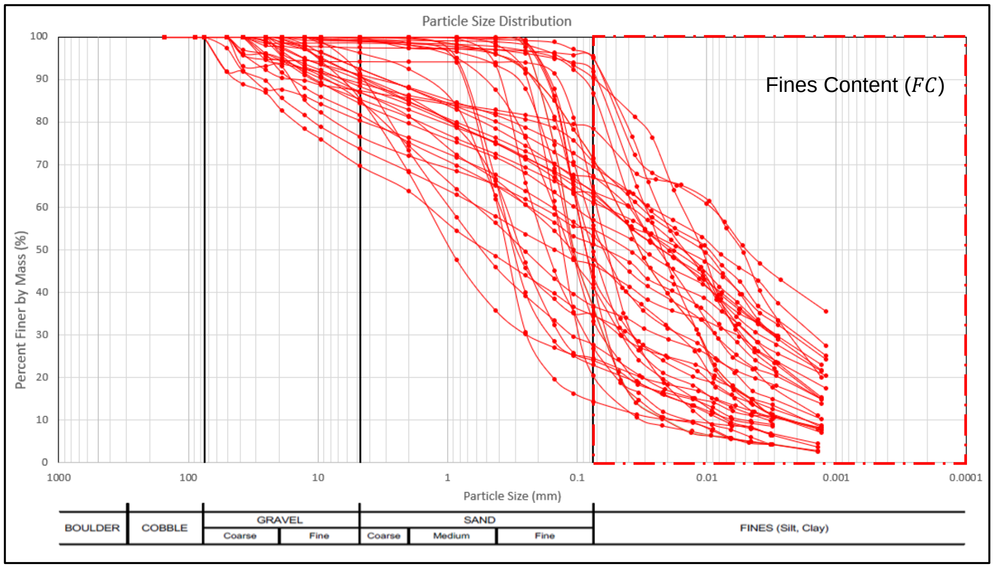


Figure 2-4: 2019 SRK Field Campaign. PSD Curves (SRK, 2019a).

2.3.2 Geotechnical Parameters

Geotechnical parameters were obtained from SRK (2019b).

Table 2-1: Material Parameters.

Material	Unit weight [kN/m^3]
Dam	20.5
Foundation (CH/CL/Sand)	18

2.3.3 Dam Configuration

The MPs and TCP dam configurations present heights varying from 16 m to 40 m. For the purpose of this assessment, the worst-case scenario was considered. The expected loads induced by the MPs and TCP were represented by an overburden stress given by an average dam height of 40 m, as shown in Figure 2-5, and a dam material unit weight of 20.5 kN/m^3, as shown in Table 2-1.

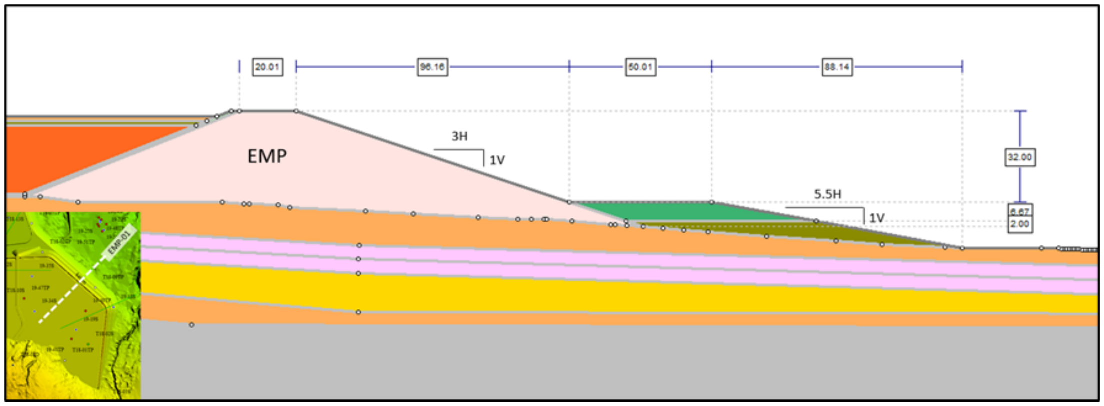


Figure 2-5: EMP-01 Section. Geometric Configuration. Worst-case scenario configuration.

2.3.4 Water Table

A steady state groundwater flow was computed in SRK (2021) to determine the pore pressure distribution along the TCP and MPs, and foundation terrain. Based on the results of this analysis, a water table equal to the foundation level was adopted.

2.3.5 Seismicity

The 2015 National Building Code Seismic Hazard Map (NCR, 2015) indicates that the Telkwa Project site is in a relatively stable seismic area. The peak ground acceleration (PGA) for a two percent probability of exceedance in a 50-year period, i.e. 1 in 2,475 years, is 0.049 times the force of gravity of 9.81 m/s^2.

Since there is no available data on the earthquake moment magnitude M, it is assumed to be 7.5.

2.4 Analysis and Results

The liquefaction assessment results are shown in Figure 2-52-7. Since in all cases $CSR < CRR$, the soils tested surrounding the MPs and TCP are not susceptible to liquefaction for the expected loads.

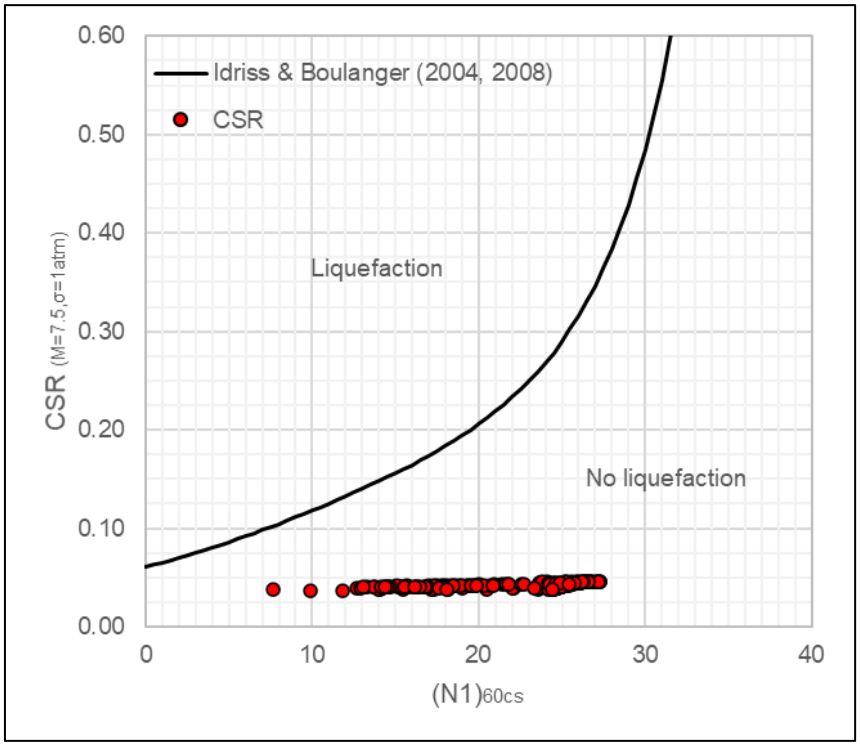


Figure 2-5: Liquefaction assessment results – Idriss & Boulanger SPT-based procedure

3 Conclusions and Recommendations

3.1 Conclusions

A SPT-based correlation procedure to assess the liquefaction potential of soils was undertaken, using the Idriss & Boulanger (2004, 2008) procedure.

Following the simplified SPT-based methodology, the preliminary liquefaction assessment found that historic soil samples gathered (prior to 2021) were not found to be liquefiable for the expected loads.

3.2 Recommendations

For the detailed engineering SRK recommends:

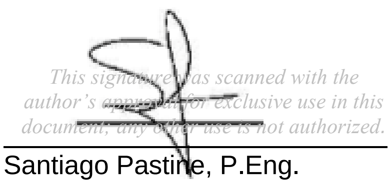
- undertake oedometric consolidation tests on undisturbed samples to determine the pre-consolidation stress;
- undertake isotropically consolidated undrained compression tests on undisturbed sample to determine the drained and undrained shear strength parameters and confirm if undrained behavior is expected or not; and
- Perform a more detailed liquefaction assessment on soil data collected during the 2021 field investigation.

4 Closure

This memorandum, "Tenas' Liquefaction Potential Assessment", was prepared by SRK Consulting (Canada) Inc. to provide recommendations for the Tenas' Management Ponds soil foundations' liquefaction susceptibility.

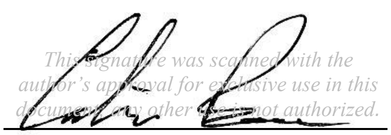
We trust the information included meets your present requirements. If you have any questions regarding the analyses or if you require additional information, please do not hesitate to contact the undersigned.

SRK Consulting (Canada) Inc.



Santiago Pastine, P.Eng.
Consultant

Reviewed by



Calvin Boese, M.Sc., P.Eng.
Principal Consultant

Disclaimer—SRK Consulting (Canada) Inc. has prepared this document for Telkwa Coal Ltd.. Any use or decisions by which a third party makes of this document are the responsibility of such third parties. In no circumstance does SRK accept any consequential liability arising from commercial decisions or actions resulting from the use of this report by a third party.

The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

References

Idriss, I. M. (1999). *An update to the Seed-Idriss simplified procedure for evaluating liquefaction potencial.*

Idriss, I. M., & Boulanger, R. W. (2004). *Evaluating the potential for liquefaction or cyclic failure of silts and clays.*

Idriss, I. M., & Boulanger, R. W. (2008). *Soil liquefaction during earthquakes.*

NCR. (2015). *National Building Code Seismic Hazard Calculation for site: 54.6163 N, 127.152 W: National Resources Canada.*

Seed, H. B., & Idriss, I. M. (1967). *Analysis of soil liquefaction: Niigata earthquake.*

SRK. (2018). *Tenas Project - Factual Data Report for 2018 Geotechnical Investigation.*

SRK. (2019a). *Tenas Project - Factual Data Report for 2019 Field Investigation.*

SRK. (2019b). *Geotechnical Stability Analysis for Management and Control Ponds.*

SRK. (2021). *Tenas Project – Management and Control Ponds Stability Assessment.*