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ASSESSMENT OF SOIL LIQUEFACTION POTENTIAL OF AN EMBAMKMENT DAM IN THE CHARLEVOIX SEISMIC ZONE (CSZ) – A CASE STUDY WITH THE COMPARAISON OF THREE TOTAL STRESS METHODS

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ABSTRACT
This paper aims to compare three total stress methods to evaluate soil liquefaction triggering of an embankment dam located in the CSZ, a highly seismic region of Quebec in eastern Canada. The three approaches are: 1- a preliminary semi-empirical method based on Idriss & Boulanger (2008), 2- a 1D nonlinear Finite Difference Method site response with FLAC and 3- a 2D equivalent linear Finite Element Method analysis with QUAKE/W. All methods estimate or calculate the cyclic shear stress ratio (CSR) and the cyclic resistance ratio (CRR) is estimated from in situ testing. For the fill in place, all three analysis show potential for liquefaction triggering since the CSR is always higher than the CRR. The simplified solution shows good results with the 2D analysis with QUAKE/W while the 1D analysis underestimates the CSR profile. The effect of the 2D geometry is verified by fitting the response spectra between the 2D and 1D dynamic analysis with a factor of 2.

RÉSUMÉ
Cet article met en évidence trois méthodes d'évaluation du potentiel de liquéfaction en contrainte totale pour une digue en terre situé dans la zone sismique de Charlevoix, une région dont le potentiel sismique est très important. Les trois méthodes employées sont: 1- la méthode simplifiée d'Idriss et Boulanger (2008), 2- une modélisation numérique 1D non-linéaire avec FLAC et 3- une analyse numérique 2D en équivalence linéaire avec QUAKE/W de la suite Geo-Studio. Toutes ces méthodes estiment ou calcul la sollicitation cyclique (CSR) et la résistance cyclique (CRR) est estimé par un essai in situ. Le remblai présente un potentiel de liquéfaction puisque le CSR est toujours plus élévé que le CRR. La méthode simplifiée et la modélisation numérique 2D présente un profil CSR similaires tandis que la modélisation en 1D sous-estime le CSR. L'effet de la géométrie sur la réponse spectrale en 1D est ajusté par un facteur d'amplification de 2.

1 INTRODUCTION

Soil liquefaction assessment is of great importance for civil engineering projects and particularly in embankment dams. The complexity of a plain strain 2D model in effective stress with hydro-mechanical coupling is often out of reach for most practitioner as it requires a great deal of skills, time and data. Over time, simpler methods were developed to deal with small projects where such efforts are impossible. These methods were designed to allow a reliable and conservative (sometimes over conservative) soil liquefaction potential assessment.

The scope of this paper is to present a case study where three (3) total stress methods were used to assess soil liquefaction potential on an embankment dam resting on roc. These methods are as follows: 1- the simplified methods firstly developed by the NCEER/NSF workshop participant of 96-98 and published by Youd & al. (2001) and updated by Idriss & Boulanger (2008); 2- a 1D non-linear finite difference model (FDM) using Itasca software FLAC 6.0; 3- a 2D equivalent-linear finite element model (FEM) with QUAKE/W from Geo-Slope international software.

The scope of this paper is not to redefine soil liquefaction theory and does not intent to propose a bullet proof methodology to soil liquefaction for all types of geometry and conditions. It presents tools that are in most cases available for practitioners to compare different analysis of soil liquefaction.

2 SOIL LIQUEFACTION BACKGROUND

2.1 Basic principles

"Soil liquefaction is the phenomenon in which soil loses much of its strength or stiffness for a generally short period but nevertheless long enough to be the cause of many failures" (Jefferies and Been 2008). If a saturated loose soil layer is solicited under cyclic loading from an earthquake it will tend to contract and transfer the stress onto the pore water. When the pore water pressure exceeds the vertical stress the soil layer will liquefy.

Although it is more common to find liquefaction in clean sand, it is now very well-known that loose silty sands and even clays (Boulanger & Idriss 2006 or Bray & Sancio 2006) are also susceptible to soil liquefaction. More recently it was found that soil liquefaction is also possible in gravelly soils depending on the drainage boundary conditions (Cao & al. 2013).

Soil liquefaction assessment is often performed as a preliminary study in a more global earthquake stability analysis. It should not be considered as the only study to define stability. It should be joined with other post-earthquake analysis to assess the global stability of an embankment and probability for lateral spreading and settlement. For further references, the Deep Foundation Institute published an excellent review of the state of practice in liquefaction analysis and an interesting interview with Mike Jefferies from Golder Associates (Siegel 2013)

2.2 Review of guidelines, laws and regulations in Canada for the selection of the Earthquake Design Ground Motion (EDGM)

Soil liquefaction analysis is used to identify the probability of a soil to liquefy under a specific earthquake recurrence and intensity. In Canada, if there are no specific provincial guidelines, the selection of the EDGM for dam safety analysis will follow CDA guidelines (CDA 2007). They use a Risk-Informed Approach which is a criteria based on the dam classification. This is defined with a hydraulic dam break study and the estimated consequence on the civil society and the environment. The recurrence of the earthquake can vary from 1: 100 years for low societal risk to 1: 10 000 years or maximum credible probability for very high and extreme societal risk. It should be noted that the CDA guidelines are more a state of practice recommendations than an actual law and they do not provide hazard values for different locations but rather propose a minimum annual exceedance probability of the natural hazard (recurrence).

The National Building Code of Canada (NBCC 2005 & 2010) have published median earthquake hazard values for the main cities in Canada based on the robust approach developed by Adams and Halchuk (2003) (see open file 4459). An important factor to consider between the CDA guidelines and the NBCC values is the use of mean values rather than the median for the EDGM respectively. A hazard calculation tool is available from the GSC's website (www.seismescanada.rncan.gc.ca) for location outside of the main cities in Canada.

In the province of Québec, a dam safety review must satisfy the Dam Safety Act and the Dam Safety Regulation. The act and regulation was passed in 2000 in response to 1996 downpour and severe floods mostly in the Saguenay region (Gouvernement du Québec 2002). To make sure the application of the act and regulation is followed by the dam owners, all dam safety reviews and new dam constructions are verified by the Centre d'Expertise Hydrique du Québec (CEHQ). Article 29 of the Act's regulation forces all dam owners to the same earthquake recurrence of 1: 2475 years but with the median values calculated by the GSC (like the NBCC) rather than the mean values (like the CDA). These values can be obtained directly from the NBCC or the hazard calculation tool

2.3 Total (cyclic stress) vs effective stress approach

The main difference between a total and an effective stress approach resides in the liquefaction triggering calculation. In an effective stress approach, liquefaction is triggered when the cyclic shear stress have caused enough build-up of pore water pressure (PWP) for the soil to lose its strength or stiffness. In a total stress approach the triggering of liquefaction is determined by the ratio of the cyclic resistance ratio (CRR) over the cyclic stress ratio (CSR). When the ratio is below one, liquefaction has been triggered. One of the main difficulty with an effective stress approach, apart from the complex numerical calculations involved, is the selection of the PWP function which will define the soil behavior under cyclic loading. In order to overcome this, a total stress approach can be very useful. This is the most common practice in estimating soil liquefaction triggering (Siegel 2013). The in situ tests will provide the estimation of soil resistance (CRR), while the cyclic shear stress (CSR) can be calculated from different methods.

2.3.1 Simplified total stress – Idriss & Boulanger (2008)

The simplified method presented by Idriss and Boulanger (2008) is a continuation of the well-known method developed during the NCEER 1996 and NCEER/NSF 1998 workshop on soil liquefaction (Youd and al. 2001). The main differences between the two are from the CSR stress correction factors (r_d), the overburden correction factor (kσ) and the fines content (FC) correction factors. More recently, Boulanger & al. (2013) reviewed the liquefaction triggering curves with an updated case history database. They concluded that the Idriss & Boulanger (2008) triggering curves were still consistent with the updated case histories.The earthquake induced CSR, for a specific depth, equals 65 % of the maximum cyclic shear stress ratio as suggested by Seed and Idriss (1971):

$$\mathrm{CSR}_{M,\sigma'_V} = 0.65\frac{\tau_{max}}{\sigma'_V} \quad [1]$$

Where τ_{max} is the maximum earthquake-induced cyclic shear stress and σ'_V is the vertical effective stress at a depth, z. The shear stress can be calculated using numerical modelling considering there is a sufficient number of time histories in order to eliminated discrepancies. It can also be estimated using the simplified procedure from Seed and Idriss (1971) as follows:

$$\mathrm{CSR}_{M,\sigma'_V} = 0.65\frac{\sigma_V}{\sigma'_V}\frac{a_{max}}{g}r_d \quad [2]$$

Where σ_V is the total vertical stress and a_{max}/g = peak horizontal acceleration (or PGA for Peak Ground Acceleration) as a fraction of gravity.

3 CASE STUDY: EARTH EMBANKMENT DAM

3.1 Seismic zone and seismic hazard values

The dam is located in the Charlevoix Seismic Zone (CSZ) also called the Charlevoix-Kamouraska Seismic Zone. It is the most active seismic zone in eastern Canada with over 200 detected earthquakes per year. The activity of the region is coming from the St-Lawrence paleo-rift faults and most earthquake occur in the Canadian shield between the surface and 30 km depth (Natural Resources Canada).

Table 1. NBCC 2010 median seismic hazard value for a class A and C with for a hazard recurrence of 1: 2475 years

Site class	PGA [g]	Sa(0.2) [g]	Sa(0.5) [g]	Sa(1.0) [g]	Sa(2.0) [g]
Class A*	0.62	0.85	0.35	0.14	0.04
Class C	0.86	1.66	0.84	0.37	0.11

* Reference Ground Condition (RGC) for eastern Canada (Adams and Halchuk (2003)

Being located in Quebec, the dam is subjected to the Dam Safety Act and Regulation. It is located in zone 5 on the CEHQ seismicity map with a Peak Rock Acceleration (PRA) of 0.5 equivalent to a site class A (Hard Rock) as defined in the NBCC. The seismic hazard values calculated for the exact location taken from the GSC are presented in Table 1 for a site classification C. When applying the simplified solution for soil liquefaction potential, both sources of peak acceleration need to be adjusted to the site conditions or site classification. This is determined from either the average Standard Penetration Resistance ($\overline{\mathrm{N}}_{60}$), average shear wave velocity ($\overline{\mathrm{V}}_{\mathrm{s}}$) or soil undrained shear strength (S_{u}) over the first 30 m from the surface, including rock.

3.2 Dam geometry (dimensions) and soil conditions

The embankment dam have an upstream slope of 3H: 1V and a downstream slope of 2.5H: 1V. A gravel wearing course is present on each sides with maximum thickness of 600 mm on the crest. A riprap protection (50-300 mm) is present on the upstream slope and a drainage toe made of sand is located on the downstream slope. This type of construction is very characteristic in the province of Quebec for small height dams since till material have good permeability properties for water retention and can be found in many regions.

The geotechnical investigation showed the presence of a very dense till foundation and a loose to medium dense till fill keyed in the foundation. The foundation lays over hard rock. The average height of the dam is between 4 to 6.25 m. A typical cut-section of the dam is presented in Figure 1 with a maximum height of 4 m.

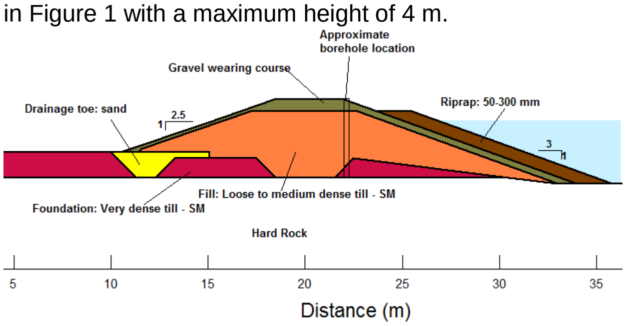

Figure 1. Typical cut-section of the embankment dam located in the CSZ

3.3 Static soil properties

According to the Idriss and Boulanger (2008) a soil will exhibit a clay-like behavior with a plasticity index higher than 7 and a sand-like behavior with a plasticity index below 4. Between these values, the soil is considered at an intermediate behavior or transition. Atterberg limits were performed on one sample of the fill and showed a plasticity index of 6 with a liquid limit of 20. For that reason and without other information available, the till (foundation and fill) is considered to exhibit a sand-like behavior and studied as such. The soil parameters presented in Table 2 are estimated from Hunt (2005).

Table 2. Soil properties for all layers

Material	γ_S kN/m³	φ' °	c' kPa	k m/s
Till (fill)	18	35	0	4.00e-9
Till (foundation)	20	35	0	4.00e-9
Gravel wearing course	19	35	0	1.00e-4
Riprap	20	40	0	1.00e-2
Sand	19	30	0	1.00e-4

Where γ_s is the estimated saturated volumetric weight of each sample, ϕ' is the effective friction angle, c' is the apparent cohesion and k is the permeability.

3.4 Simplified liquefaction analysis

The simplified solution is built for in situ testing like SPT tests. For the embankment dam in question, 1 borehole located on the crest was analysed. This borehole was considered to be a good representation of the dam geometry since it was the deepest (6.25 m) and included sampling of the foundation and fill. The determination of the class for the selection of the appropriate a_{max} (PGA) is done with the harmonic mean of the estimated non normalized shear wave velocities (Vs). The rock below 6.25 m has an estimated shear wave velocity of 1500 m/s. The average shear wave velocity over the first 30 m is 630 m/s which corresponds to a site classification C and a PGA of 0.86 g.

The magnitude scaling factor (MSF) is calculated with the deaggregation files obtained from the GSC for a mean moment magnitude of 6.75 and a mean distance of 24 km. The fine content and median diameter (D_{50}) are measured with soil distribution tests and equal for all till layers. The borehole information is presented in Table 3 and the simplified methods in Table 4.

Table 3. Borehole information for liquefaction assessment

Depth m	Class USCS	FC %	D_{50} mm	γ_S kN/m³	σ'_V kPa	SPT $(N_1)_{60}$	SPT $(N_1)_{60\text{-cs}}$
1.30	SM	44	0.14	18	18.0	39.1	44.7
2.06	SM	44	0.14	18	28.6	11.4	17.0
2.50	SM	44	0.14	18	32.2	11.1	16.7
3.45	SM	44	0.14	18	40.0	13.2	18.8
4.40	SM	44	0.14	18	47.8	12.3	17.9
5.12	SM	44	0.14	18	53.7	10.8	16.4
5.85	SM	44	0.14	20	61.1	67.1	72.7
6.25	SM	44	0.14	20	67.6	55.7	61.3

Table 4. Simplified procedure for liquefaction assessment

Depth #	Vs_1 m/s	MSF ---	kσ ---	rd ---	CSR ---	CRR ---	$CRR_{k\sigma\text{-}MSF}$ ---
1.30	234.7	1.22	1.10	0.99	0.55	1.00	1.00
2.06	172.5	1.22	1.10	0.98	0.61	0.17	***0.23***
2.50	171.5	1.22	1.10	0.98	0.67	0.17	***0.23***
3.45	179.0	1.22	1.10	0.96	0.76	0.19	***0.26***
4.40	175.7	1.22	1.08	0.95	0.82	0.18	***0.24***
5.12	170.1	1.22	1.06	0.94	0.84	0.17	***0.22***
5.85	268.7	1.22	1.10	0.92	0.85	1.00	1.00
6.25	256.5	1.22	1.10	0.92	0.86	1.00	1.00

Where $(N_1)_{60\text{-}cs}$ is the normalized penetration resistance for confinement (subscript 1), 60 % energy ratio (subscript 60) and clean sand (subscript cs) and $CRR_{k\sigma\text{-}MSF}$ is the normalized resistance for a magnitude different than 7.5 (MSF) and the overburden correction factor (kσ). The static shear correction factor (kα) is not calculated in the analysis since for $(N_1)_{60}$ values close to 12, the factor is close to 1 (Idriss & Boulanger, 2008).The CRR is estimated with the normalized penetration resistance as displayed in equation 3. For normalized penetration resistance higher than 32, the CRR is stopped at 1.00 for calculation purposes. The last two soil samples was taken into the till foundation.

$$\mathrm{CRR} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \quad [3]$$

By comparing the $CRR_{k\sigma\text{-}MSF}$ with the CSR, it is clear the soil samples between #2 to #6 (submerged) have a potential for liquefaction because the resistance is lower than the cyclic stress estimated. The next steps of the analysis will be to verify this result using FLAC 1D analysis which represents the same soil profile as the one used for the simplified solution.

3.5 One (1) dimension dynamic analysis with FLAC

3.5.1 Dynamic soil properties and 1D geometry

A plane strain model of 1 m width is used for the analysis. Free-field boundaries were applied on each sides of the model. The soil parameters are defined for a linear model where the bulk modulus and shear modulus are needed to define the elastic behavior. The maximum shear modulus (Gmax) values are calculated with equation 4 and with the normalized shear wave velocity (Estimated from Karray & Hussein (2014), Submitted to the Canadian Geotechnical Journal for possible publication) of each layer.

A value of 300 m/s is considered for the very dense gravel wearing course on top. The bulk modulus for all submerged layers is set to 4.5e9 Pa to allow for an incompressible behavior like water which gives a corresponding poisson ratio of 0.5. The bulk modulus (kmax) above the water table is equal to 3 times the maximum shear modulus. The water table is set at a depth of 1.48 m. The soil profile for the numerical analysis is the same as the one taken from the SPT tests and it is presented in Figure 2.

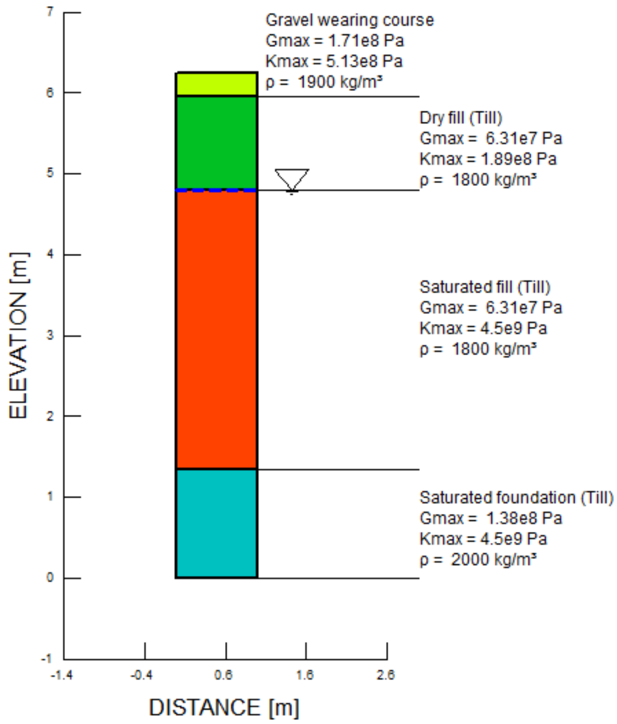

Figure 2. Soil profile for the dynamic analysis with FLAC

$$G_{max} = \rho Vs_1^2 \quad [4]$$

For the propagation of the seismic wave through the soil, FLAC uses a fully non-linear hysteretic damping which is already implemented in the dynamic module. The modulus reduction curve is defined using Sig4 (sigmoidal models) which is also implemented in the software and only needs to be defined. Since there is no available data for the Gmax degradation curve for this till foundation of fill, the Sig4 function is fitted with the upper range data from Seed and Idriss (1970). For a bigger project, it is more appropriate to use the right degradation curve for the right layer.

3.5.2 Natural period

The natural period of the soil profile can be estimated based on average shear wave velocity in the soil layer and equation 5. For a maximum height (H) of 6.25 m and an average shear wave velocity of the 141.1 m/s, the natural period (T_0) if 0.18 sec.

$$T_0 = 4H/\overline{Vs} = 4 \times 6.25/141.1 = 0.18\mathrm{sec} \quad [5]$$

With the 1D column, it is also possible to check for the natural period with a sine wave propagation at the base. The seismic response at the surface will show the fundamental modes of the specific soil column (flac dynamic manual). Figure 3 shows the response spectra for the sine wave analysis. The fundamental mode is found at 0.18 sec which confirms the first estimation using equation 5.

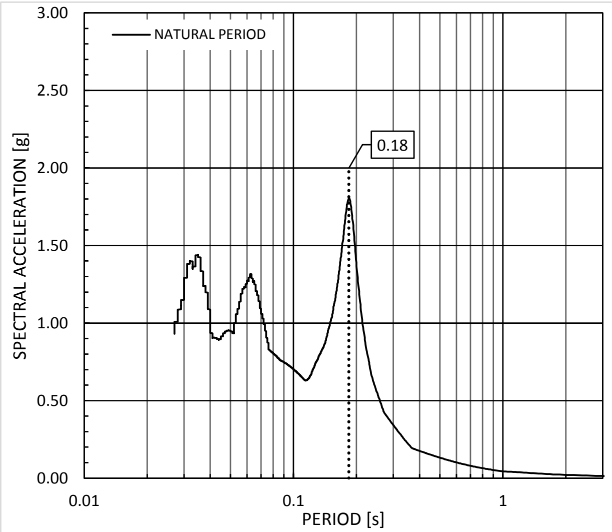

Figure 3. Seismic site response using a sine wave for the natural period verification

3.5.3 Time histories

Seven (7) time histories were selected for the dynamic analysis with FLAC to eliminate discrepancies. Some were simulated ground motions obtained from http://www.seismotoolbox.ca/ for eastern Canada. Some were recorded ground motion in eastern Canada: the Saguenay earthquake in 1988 with a moment magnitude (Mw) of 5.9 recorded in La Malbaie and Baie-St-Paul and the Val des bois earthquake in 2010 with a Mw of 5.0; and some from the Northwest Territory with the Nahani earthquake with a Mw of 6.9.

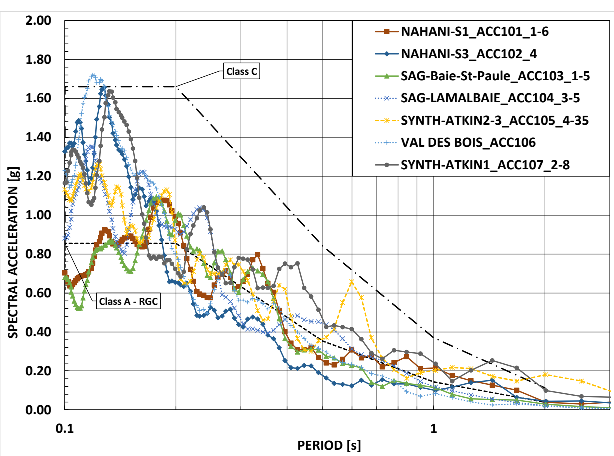

Figure 4. Adjusted time histories to the Class A – RGC spectrum

FLAC does not intrinsically need a baseline correction to function correctly. Still, a linear baseline correction was applied for all signals to eliminate the displacement drift at the end of the time history (Itasca 2008). All time histories were amplified to fit a Class A spectrum for the site location based on the RGC factors with a fitting emphasis close to the natural period of 0.18 sec. Figure 4 presents all amplified acceleration. The spectral content of the selected acceleration profiles are strong in high frequencies (small periods) which is representative of the earthquakes in eastern Canada.

3.5.4 FLAC 1D seismic site-response

Every time history presented in section 3.5.3 were applied separately at the base of the numerical model at the hard rock elevation. The surface site response of each time history is showed on Figure 6 to Figure 12 in color with the hard rock input motion in black. The response spectra of all time histories are presented in Figure 5. There is an evident resonance (amplification) effect in high frequencies. This was expected because of the natural period of the dam being quite low and the earthquake signals in eastern Canada having a strong content in low periods.

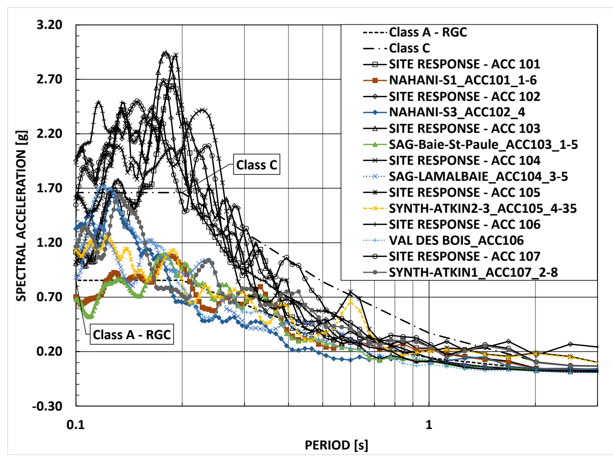

Figure 5. FLAC 1D site response spectra for all time histories

The CSR is calculated at each node element in the model following the relation of Seed and Idriss (1971) presented in equation 1 (see Table 5).

Table 5. CSR calculation results from a 1D dynamic analysis with FLAC

z m	CSR 101	CSR 102	CSR 103	CSR 104	CSR 105	**CSR 106**	CSR 107
0.15	0.33	0.33	0.28	0.32	0.35	0.37	0.35
0.48	0.32	0.32	0.28	0.32	0.34	0.36	0.34
1.05	0.32	0.31	0.27	0.31	0.33	0.35	0.33
2.10	0.37	0.36	0.32	0.36	0.39	0.41	0.39
3.19	0.42	0.39	0.36	0.40	0.45	0.47	0.41
4.26	0.43	0.40	0.37	0.40	0.46	0.48	0.43
5.14	0.39	0.40	0.36	0.40	0.41	0.45	0.42

5.81	0.39	0.42	0.35	0.41	0.39	0.47	0.43

The CRR is estimated the same way as with the simplified solution. With values of normalized CRR in the range of 0.24, the entire fill layer has a potential for liquefaction for all 7 time history tested. The average CSR with FLAC 1D is equal to 0.38. In comparison with the estimated CSR from the simplified method with an average of 0.75, the computed CSR is close to two (2) times lower than the estimated cyclic stress ratio. On the other hand, this analysis does not take into account the 2D geometry effect of the embankment dam and may underestimated the CSR. The next step in the analysis is a 2D dynamic numerical analysis with QUAKE/W to verify the amplification effect of the 2D geometry.

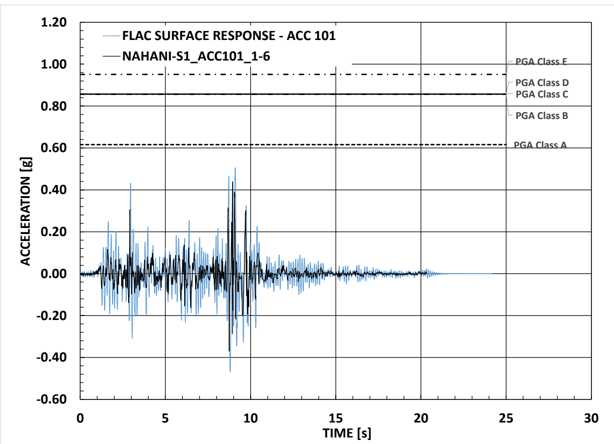

Figure 6. Acceleration 101 – Nahani-S1

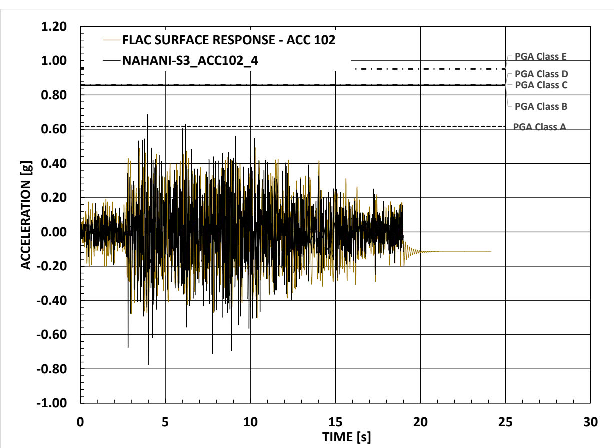

Figure 7. Acceleration 102 – Nahani-S3

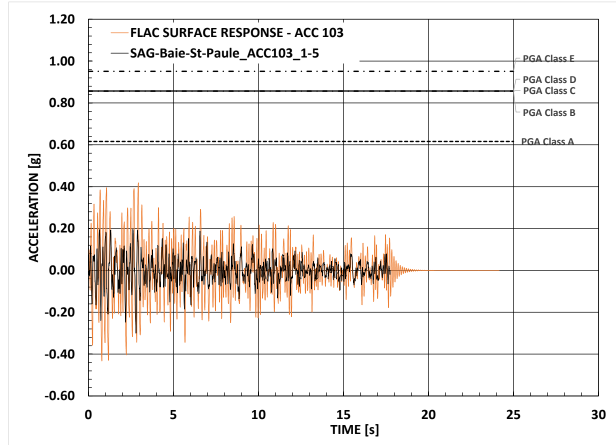

Figure 8. Acceleration 103 – Saguenay: Baie St-Paul

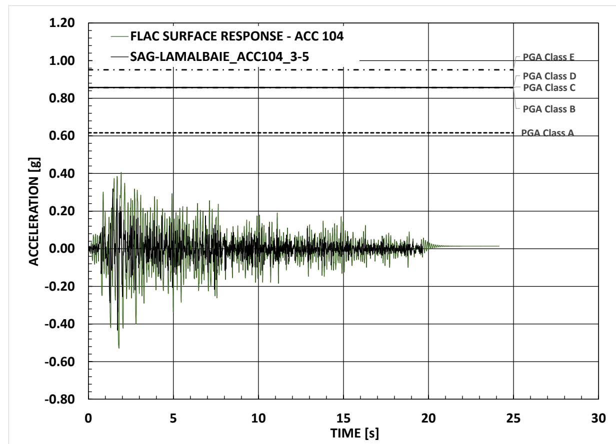

Figure 9. Acceleration 104 – Saguenay: La Malbaie

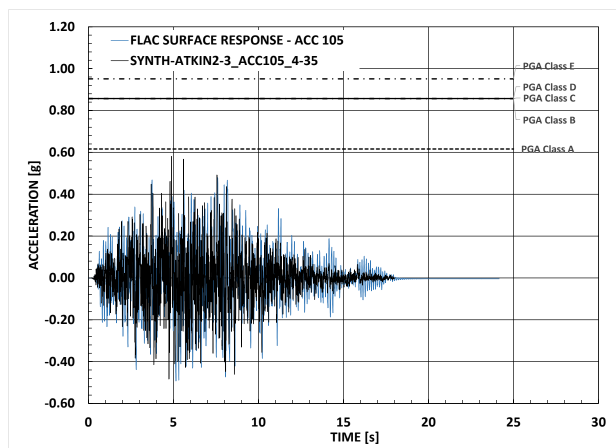

Figure 10. Acceleration 105 – Synthetic Atkinson 2-3

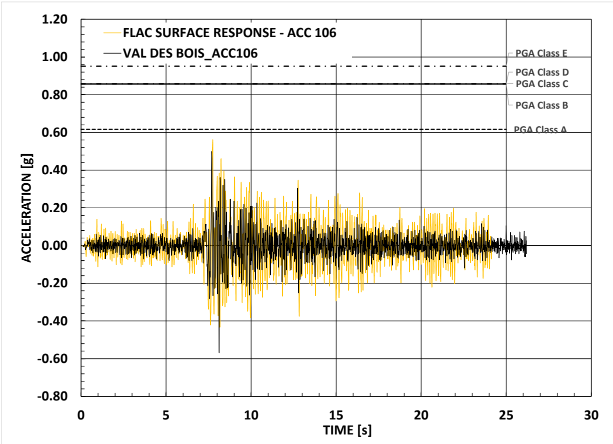

Figure 11. Acceleration 106 – Val des bois

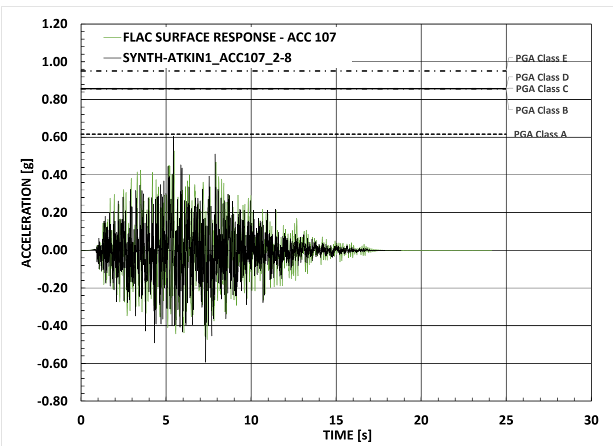

Figure 12. Acceleration 107 – Synthetic Atkinson 1

3.6 Dynamic analysis with Quake 2D in equivalent-linear

3.6.1 Dynamic soil parameters, geometry and time history

The dynamic analysis using QUAKE/W in 2D requires the definition of: the Gmax reduction function, the damping ratio function, the pore water pressure (PWP) function, the cyclic number function, the Kα function (static shear stress correction factor) and the Kσ function (overburden correction factor). QUAKE/W calculates the CSR the same way as detailed in equation 1 except that it also divides the CSR by the Kα and the Kσ. As stated before, the Kα for is above 1 for values of $(N_1)_{60} > 12$. Also, the Kσ is already included in the CRR calculation in the simplified method. For the purpose of comparing the CSR between the different analyses, the Kα and Kσ are equal to 0 during the shaking in QUAKE/W. A quadratic and triangular mesh is used with a total of 1473 nodes and 1318 elements and with a global element size of 0.25 m.

Furthermore, the cyclic number function and the PWP function will not influence the calculation of the CSR (Geostudio 2013) but they are needed for the calculation to be lunched. For that reason, both functions are given arbitrary values. The Gmax functions illustrates the variation of the Gmax modulus with depth. This value is estimated with the shear wave velocity (Vs) profile and the vertical effective stress.

In order to reduce calculation time, the QUAKE/W analysis was performed with only one time history which is the maximum earthquake response from the 1D numerical analysis with FLAC; the time history of Val des Bois (ACC106) was applied at the base of the model.

3.6.2 2D Dynamic analysis

To verify the QUAKE/W model response and the right parameters, a 1D numerical analysis was performed with the dynamic parameters as explained in section 3.6.1. The site response spectra was analysed at the surface and compared with the results from the FLAC analysis. The response spectra is presented in Figure 13.

The site-response spectra for both models are very close to each other in the low periods range (<0.2 sec) and for higher period (>0.6 sec). Between this range, the QUAKE/W 1D shows some amplification in comparison with the FLAC 1D. Overall, both 1D analysis are close and a 2D model was performed with these parameters. This 2D numerical analysis will serve as a way to obtain the amplification caused by the geometry in comparison with the 1D analysis.

The 2D dynamic numerical model geometry is shown on Figure 1. The typical cut-section of the dam at full height is 4 m while in comparison with borehole is up to 6.25 m. The CSR is calculated at the center of the dam crest until the rock layer and the results are shown in Table 6. The average CSR value is 0.54 which is very close from the average CSR estimated from the simplified solution (0.74) but still lower. Once again the QUAKE/W 2D CSR is higher than the average CRR calculated with the simplified solution which is in the range of 0.24. The soil still have a good potential for liquefaction under these conditions.

Table 6. CSR calculation results with a 2D dynamic analysis with QUAKE/W

Depth [m]	0.60	0.85	1.10	1.35	1.60
CSR	0.26	0.30	0.36	0.43	0.49
Depth [m]	1.85	2.10	2.30	2.53	2.76
CSR	0.54	0.58	0.61	0.63	0.64
Depth [m]	3.00	3.25	3.50	3.75	4.00
CSR	0.65	0.65	0.64	0.64	0.64

4 DISCUSSION ON THE DIFFERENT ANALYSIS

The site response spectra for the 2D numerical analysis is much higher than the response spectra of the 1D numerical analysis with FLAC with the Val des Bois earthquake. This tends to indicate an amplification due to geometry. In order to fit the FLAC 1D response spectra with the QUAKE/W 2D, it needs to be amplified by a factor of two (2) (see Figure 13). The same factor is applied to the maximum and minimum CSR profile obtained during the 1D FLAC analysis (see Figure 14).

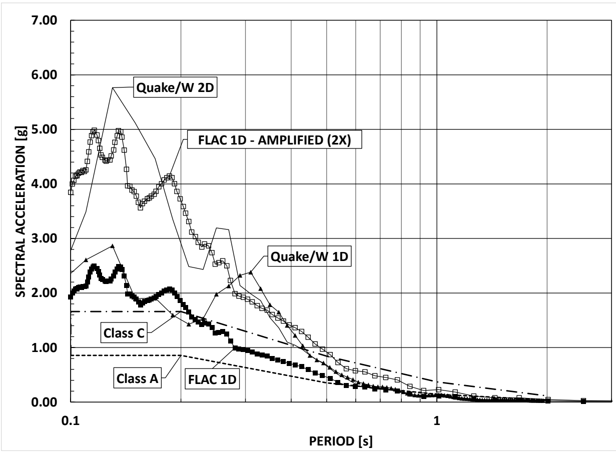

Figure 13. Amplification of the site response spectra between FLAC 1D and QUAKE/W 2D

With this procedure, there is a close fit between the CSR profiles of all three calculation methods. This tends to show that the simplified method may overestimated the cyclic shear stress for cases where a geometry amplification would not be present and the 1D would underestimate the cyclic stress where the same geometry would be important.

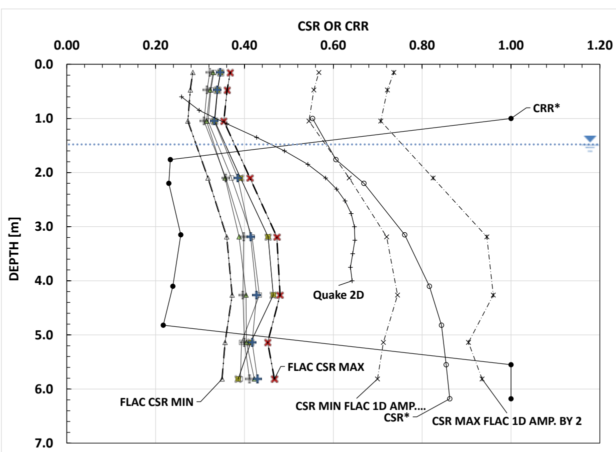

Figure 14. Comparison of the CSR profiles calculated between all three total stress analyses

5 CONCLUSION

Three total stress analysis were performed on an embankment dam to calculate the CSR and determine if liquefaction triggering could happen during a seismic event. The following conclusions can be drawn:

1. For the fill in place, all three analysis show potential for liquefaction triggering since the cyclic stress is always higher than the cyclic resistance;
2. With $(N_1)_{60}$ higher than 32, the foundation, does not present any potential for liquefaction triggering;
3. The geometry of an embankment can have a noticeable impact on the amplification of the earthquake signal;
4. The simplified solution shows good results with the 2D analysis with QUAKE/W while the 1D analysis underestimates the CSR profile;
5. The effect of the 2D geometry is verified by fitting the response spectra between the 2D and 1D dynamic analysis with a factor of 2.

It should be noted that for more complex and important projects, the dynamic soil parameters should be more thoughtfully investigated.

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