Influence of Fines Content on Cyclic Resistance through the Critical State Framework

Edouardine-Pascale Ingabire
University of Toronto, Toronto, Ontario, Canada
Lawrence Clelland, Alex Sy
Klohn Crippen Berger, Canada
Mason Ghafghazi
University of Toronto, Toronto, Ontario, Canada

ABSTRACT

The critical state approach proposes a theoretically-sound mechanistic framework which can be applied to liquefaction assessment in cohesionless soils. Because the value of the state parameter (ψ) controls soil behaviour, it is the preferred index to estimate CRR. In this paper, available data from the literature are compiled and re-analyzed to derive soil-specific cyclic resistance curves. The database includes laboratory standard sands, natural clean to silty sands, and silty mine tailings. The objective of this work is to assess the influence of fines content on CRR.

RÉSUMÉ

L'approche *critical state* offre une méthodologie théoriquement solide pour évaluer le potentiel de liquéfaction de matériaux granulaires. Le *state parameter* (ψ) est l'indice privilégié pour estimer CRR, puisque la valeur de ψ contrôle le comportement d'un matériau. Dans la présente étude, des résultats de laboratoire publiés dans la littérature sont utilisés afin d'estimer CRR en fonction de ψ. La base de données inclut des sables normalisés, des sables à sables silteux naturels ainsi que des résidus miniers. L'objectif principal est d'évaluer l'influence du pourcentage de fines sur CRR.

1 INTRODUCTION

Liquefaction is the phenomenon in which a soil mass loses its shear strength following generation of excess pore water pressure. The 1964 earthquakes in Japan and Alaska put liquefaction and its consequences on geotechnical engineers' radar. Following these events, intensive research was undertaken to improve the knowledge relating to soil cyclic behaviour. Traditionally, emphasis was put on natural clean sands containing less than 5% fines, where fines are particles with a grain size diameter smaller than 0.075 mm. Field observations during the seismic events in Japan (1995) and Turkey (1999) led to various studies on cyclic behaviour of silty soils (e.g. Ishihara et al., 1998; Bray and Sancio, 2006).

Because of the difficulties in obtaining high-quality samples of non-plastic and low-plasticity silty soils, engineers rely on in-situ penetration tests such as the Cone Penetration Test (CPT) to assess their cyclic resistance in a process similar to that of sands. The CPT has become the predominant tool for site investigations as it offers continuous profiling at a relatively low cost and high speed. The CPT is particularly useful in silty soils as other tests such as the Standard Penetration Test (SPT) tend to be much less reliable in soft and fine-grained soils.

The industry-standard simplified method (Seed and Idriss, 1971; Boulanger and Idriss, 2014) proposes a liquefaction triggering curve which is a boundary between cases where evidence of liquefaction was or was not observed during past earthquakes. This empirical approach correlates the CPT tip resistance (q_c) to the cyclic resistance of cohesionless soils expressed as the Cyclic Resistance Ratio (CRR).

Since the introduction of the case-history based approach by Seed and Idriss (1971), nearly fifty years ago, the database has been successively expanded and the correlations refined with the contribution of many researchers. Regardless of this effort, the framework remains a 'simplified procedure' with the most major simplistic assumption being that the entire range of soil properties can be reduced to a single index, i.e. fines content correction. The measured in-situ q_c is corrected with an equivalent clean sand adjustment (Δq_{c1N}) to account for the effect of fines on q_c and CRR.

The simplified procedure is largely based on documented case histories and laboratory data on natural clean sands and non-plastic silty sands with less than 35% fines content (FC). The literature presents conflicting views on the influence of fines content on cyclic resistance. Furthermore, compressibility and drainage conditions which control penetration resistance among other parameters, are ignored in the empirical framework. Experimental data on unconventional soils are needed to understand the factors governing cyclic behaviour.

The critical state approach, which uses a theoretically sound mechanistic framework, can be applied for liquefaction assessment of silty deposits. This framework requires an estimation of the in-situ density and the density index of choice is the state parameter (ψ), because it controls soil behaviour (Been and Jefferies, 1985).

In the current study, a database of laboratory tests from the literature was compiled, scrutinized, and re-analyzed to assess the influence of FC on CRR. The database includes monotonic triaxial compression tests, as well as cyclic triaxial and cyclic direct simple shear tests for six materials ranging from sands to silty sands. Following a background on liquefaction assessment and critical state mechanics, the paper describes the materials studied and the process of data compilation. A comparison of the cyclic resistance curves derived for each soil demonstrates a greater influence of compressibility, rather than FC, on CRR.

2 BACKGROUND

2.1 Liquefaction assessment

During an earthquake, soil is subjected to an irregular series of cyclic shear stresses. Such conditions are approximately reproduced in laboratory by performing cyclic triaxial (CTx) and cyclic direct simple shear tests (CDSS). In both devices, a soil element is consolidated and then subjected to multiple cycles of uniform loads.

The magnitude of the cyclic loading is quantified with the cyclic stress ratio (CSR), which is the ratio of the uniform shear stress divided by the initial effective confining stress. A typical soil response under cyclic loading is characterized by cumulative increase in the excess pore water pressure, causing an overall reduction in the confining effective stress, a reduction of sample stiffness, and accumulation of strains. The number of cycles at which liquefaction will occur (N_{liq}) depends on density, confinement and CSR. Soil cyclic resistance is commonly reported by the cyclic resistance ratio (CRR_{15}), which is defined as the CSR required to reach liquefaction in 15 cycles.

Laboratory studies on cyclic resistance of sand-fines mixtures have demonstrated a lack of agreement on the influence of FC on cyclic resistance. Amini and Qi (2000), among others, have suggested that an increase in FC increases CRR, while other studies suggest that an increase in FC reduces CRR (e.g., Troncoso and Verdugo, 1995). Ishihara and Koseki (1989) have suggested that FC is poorly related with cyclic strength, and that plasticity of the fines is the important factor.

It has been argued that these contradictions arise for two main reasons: Firstly, the parameter to quantify density has not been consistent in the results reported in the literature, which does not allow for a logical comparison of the results. Compactness has been expressed in terms of relative density, D_r, global void ratio, e, and skeleton void ratio, e_{sk}, or intergranular void ratio, e_g (Carraro et al., 2005). Secondly, it has been argued that it is essential to know the threshold fines content (FC_{th}) of the base sand to predict if the liquefaction resistance will increase or decrease with FC (e.g., Polito and Martin, 2001).

FC_{th} captures how the soil microstructure is affected by the presence of fines. At FC lower than FC_{th}, the fines are not part of the soil force chain formed by active grain contacts (Thevanayagam, 1998). The soil matrix is controlled by the sand particles which are transferring normal stresses and sustaining shear stresses. As FC increases above FC_{th}, the fines dominate in the soil force chain. Thevanayagam et al. (2000) suggested that, for the same initial global void ratio, e, an increase in FC will cause an initial reduction in CRR, until FC exceeds FC_{th}, at which point CRR will decrease with increasing FC.

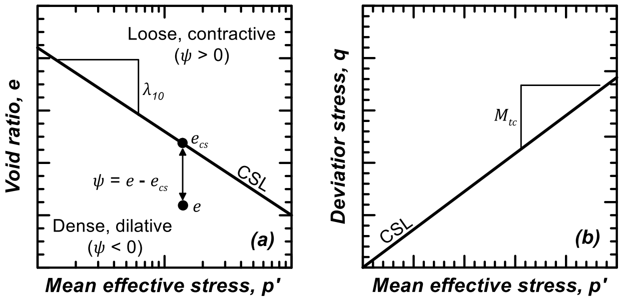

Figure 1: Definition of critical state line (a) in e-$\log p'$ space (b) in q-p' space

In the case-history based approach, after applying the fines content correction, the framework essentially regards all soils as a uniformly graded clean sand. The use of an 'equivalent clean sand' concept suggests that all sands are expected to behave similarly under the same conditions of density and confinement. Castro and Poulos (1977) demonstrated that two different sands may show different behaviour when subjected to cyclic loading, although the two sands appear similar based on index test results.

2.2 Critical state soil mechanics

To understand the controlling factor in seismically-induced liquefaction, it is important to adopt a mechanistic approach. Critical state offers a good framework because it was demonstrated that cyclic response normalizes very well with ψ (e.g., Stamatopoulos, 2010).

The state parameter, ψ, is an alternative to D_r, which captures the effects of both void ratio and confining pressure. The state parameter is defined as the difference between the current void ratio (e) and the void ratio at critical state (e_{cs}), at the same mean effective stress (p') (Been and Jefferies, 1985).

The critical state is the final state of a soil element sheared to very large strains, at which point it will continue to deform at constant void ratio, e_{cs}, constant mean effective stress, p'_{cs}, and constant deviator stress, q_{cs}. These e_{cs} and p'_{cs} values form a locus known as the Critical State Line (CSL) as illustrated in Figure 1. A semi-logarithmic linear representation is often used over p' ranges of 50 kPa to 1000 kPa, typical of most engineering problems.

In the e-$\log p'$ space, the CSL is defined by its intercept at p'_{cs}=1 kPa (Γ_1) and slope (λ_{10}). λ_{10} reflects soil compressibility (Been and Jefferies, 1985). The subscript 10 specifies that it is evaluated on a base 10. M_{tc} is the critical shear stress ratio which is analogous to the critical state friction angle (φ_{cs}), with the subscript 'tc' denoting the triaxial compression condition. M_{tc} also describes the CSL in q-p' space.

$$e_{cs} = \Gamma - \lambda_{10}\log(p'_{cs}) \qquad [1]$$

Table 1: Description of studied materials

	Grain characteristics		Gradation			
Soil name	Mineralogy	Angularity	D_{50} (μm)	$\frac{D_{60}}{D_{10}}$	FC (%)	Reference
Monterey Sand	Quartz (70% - 80%), feldspar, mica	Sub-rounded to sub-angular, fairly smooth surface	390	1.9	0	Tringale, 1983
Fraser River Sand	Metamorphic rock (35%), quartz (25%), feldspar, granite	Angular to sub-angular, low to medium sphericity	260	1.9	0	Ghafghazi, 2011
EG Sand						Stamatopoulos, 2010
Sand fraction: natural Egyptian deposit	Quartz	Rounded	350	2.4	0	
Fines: crushed from natural Greek sandy deposit	Quartz	-	-	-	100	
Ottawa Sand						Carraro, 2004
Sand fraction	Quartz	Rounded to sub-rounded	310	1.9	0	
Fines: crushed from sand fraction	Quartz	-	-	-	100	
Christchurch CBD Sand						Taylor, 2015
Sand fraction: extracted from natural sand and silty sand deposit	Quartz (60% - 70%), albite, kaolinite, muscovite	Sub-angular to sub-rounded	115	2.7	< 5	
Fines: extracted from natural silty sand deposit	Quartz (60% - 70%), albite, kaolinite, muscovite	Angular, elongated, platy	-	-	100	
Molybdenum Tailings						Been, 2016
TCS Sand	-	-	180	5.0	22	
TCB Silt	-	-	70	15.4	51	

Table 2: Summary of monotonic triaxial compression tests used to assess critical state parameters of studied materials

Material	Test conditions (number of tests)[1]	Preparation method[2]	Range of e_0	Range of p'_0 (kPa)	Reference
Monterey Sand	CID (22)	MT (?)	0.53 – 0.80[5]	30 – 294	Jefferies and Been, 2016
Fraser River Sand	CID (9), CIU (7)	MT	0.63 – 1.05[5]	50 – 515	Ghafghazi, 2011
EG Sand					Stamatopoulos, 2010
0% FC	CID (3), CIU (8)	MT	0.66 – 0.74[3]	50 – 200	
15% FC	CID (3), CIU (6)	MT	0.54 – 0.67[3]	30 – 200	
25% FC	CID (2), CIU (5)	MT	0.37 – 0.58[3]	50 – 200	
Ottawa Sand					Murthy, 2006
0% FC	CID (3), CIU (24)	MT, SD, WP	0.58 – 0.73[4]	150 – 655	
5% FC	CID (7), CIU (17)	MT, SD	0.56 – 0.72[4]	295 – 650	
10% FC	CID (3), CIU (9)	MT, SD	0.54 – 0.62[4]	305 – 650	
15% FC	CID (2), CIU (8)	MT, SD	0.48 – 0.58[4]	340 – 650	
Christchurch CBD Sand					Taylor, 2015
< 5% FC	CID (8), CIU (8)	MT, U: gel-push samples	0.69 – 0.97[4]	30 – 140	
17% FC	CID (6), CIU (10)	MT, U: gel-push samples	0.66 – 1.03[4]	30 – 200	
40% FC	CID (5), CIU (10)	MT, U: gel-push samples	0.72 – 1.05[4]	30 – 300	
Molybdenum Tailings					Been, 2016
TCS Sand	CID (5), CIU (5)	U: Shelby tubes	0.52 – 0.70[5]	35 – 3000	
TCB Silt	CID (7), CIU (6)	U: Shelby tubes	0.45 – 0.61[5]	35 – 3000	

[1] CID: consolidated isotropically drained, CIU: consolidated isotropically undrained
[2] MT: moist tamping, SD: slurry deposition, WP: water pluviation, U: undisturbed
[3] Forward void ratio measurement from sample initial dimensions
[4] Backward void ratio measurement from end of test water content as proposed by Verdugo and Ishihara (1996)
[5] Backward void ratio measurement from water content using sample freezing as proposed by Sladen and Handford (1987)

Table 3: Summary of cyclic tests used to assess liquefaction resistance of studied materials

Material	Apparatus[1] (number of tests)	Preparation method[2]	Range of D_{r0} (%)	Range of σ'_{v0} or p'_0 (kPa)	Liquefaction criteria[3]	Reference
Monterey Sand	CSS (72)	WP	35 – 50	40 – 180	γ = 6% D.A.	Wu, 2002
Fraser River Sand	CSS (61)	WP	31 – 72	50 – 400	γ = 3.75% S.A.	Sivathalayan, 1994
	CSS (14)	AP	38 – 44	50 – 200	γ = 3.75% S.A.	Sriskandakumar, 2004
	CSS (35)	AP	27 – 79	50 – 200	γ = 3.75% S.A.	Manmathrarajan, 2011
	CSS (19)	U: ground freezing	16 – 73	105 – 170	γ = 3.75% S.A.	Robertson and Wride, 1999
EG Sand						Stamatopoulos, 2010
0% FC	CTx (36)	MT	0 – 90	50 – 200	γ = 5% D.A.	
15% FC	CTx (44)	MT	0 – 100	50 – 250	γ = 5% D.A.	
25% FC	CTx (38)	MT	0 – 100	50 – 150	γ = 5% D.A.	
Ottawa Sand						Carraro, 2004
0% FC	CTx (25)	SD	40 – 91	100	γ = 5% D.A.	
5% FC	CTx (12)	SD	31 – 80	100	γ = 5% D.A.	
10% FC	CTx (12)	SD	41 – 78	100	γ = 5% D.A.	
15% FC	CTx (14)	SD	45 – 80	100	γ = 5% D.A.	
Christchurch CBD Sand						Taylor, 2015
< 5% FC	CTx (7)	MT, U: gel push	60	130-140	γ = 5% D.A.	
17% FC	CTx (4)	MT	72	90	γ = 5% D.A.	
40% FC	CTx (4)	MT	72	60	γ = 5% D.A.	
Molybdenum Tailings						Been, 2016
TCS Sand	CSS (4)	U: Shelby	-	500	γ = 3.75% S.A	
TCB Silt	CSS (4)	U: Shelby	-	500	γ = 3.75% S.A	

[1] CTx: cyclic triaxial, CSS: cyclic direct simple shear [2] WP: water pluviation, MT: moist tamping, SD: slurry deposition, U: undisturbed
[3] S.A. : single-amplitude, D.A. : double-amplitude

Table 4: Critical state parameters for 6 natural sands to silty sands and tailings

Material	FC (%)	Γ_1	λ_{10}	M_{tc}	ϕ_{cs} (°)	Reference
Monterey Sand	0	0.878	0.029	1.29[2]	32	Jefferies and Been, 2016
Fraser River Sand	0	1.220	0.138	1.45[2]	36	Ghafghazi, 2011
EG Sand						Stamatopoulos, 2010
EG-0	0	0.890	0.058	1.30[1]	32	
EG-15	15	0.705	0.050	1.30[1,3]	32	
EG-25	25	0.615	0.048	1.30[1,3]	32	
Ottawa Sand						Murthy, 2006
OS-0	0	0.787	0.044	1.21[1]	30	
OS-5	5	0.789	0.055	1.23[1]	31	
OS-10	10	0.690	0.043	1.28[1]	33	
OS-15	15	0.692	0.074	1.39[1]	34	
Christchurch CBD Sand						Taylor, 2015
CBD-0	< 5	1.045	0.067	1.43[1]	35	
CBD-17	17	1.040	0.090	1.43[1]	35	
CBD-40	40	1.205	0.157	1.44[1]	36	
Molybdenum Tailings						Been, 2016
TCS Sand	22	0.914	0.115	1.45[2]	36	
TCB Silt	51	0.713	0.086	1.44[2]	36	

[1] Evaluated from end of test method. [2] Evaluated from the stress-dilatancy behaviour (Bishop, 1966).
[3] Assumed equal to M_{tc} for clean sand gradation.

$$M_{tc} = \frac{q_{cs}}{p'_{cs}} \qquad [2]$$

$$M_{tc} = \frac{6 \sin \phi_{cs}}{3 - \sin \phi_{cs}} \qquad [3]$$

3 LABORATORY DATABASE

Six non-plastic soils are studied in the current paper for which reliable monotonic triaxial and CTx or CDSS test results are available in literature. The soils include two laboratory standard sands: Monterey sand and Ottawa sand; natural sands to silty sands: Fraser River sand, EG sand, and Christchurch CBD sand; and one mine tailings: Molybdenum tailings. For four of these reference soils, sand-fines mixtures with varying FC were studied. For the

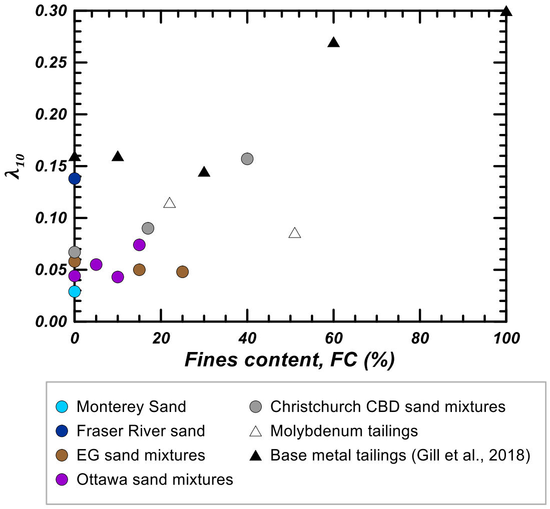

Figure 2: Influence of fines content on λ_{10}

mixtures created using Ottawa sand and EG sand, the fines were produced from crushing. Christchurch CBD sand and Molybdenum tailings exist at different FC in the field. Table 1 summarizes grain characteristics and gradation for each material. For sand-fines mixtures, Table 1 presents a description of the sand and fines fractions.

208 monotonic triaxial compression tests are included in the laboratory database. Isotropic drained (CID) and undrained (CIU) tests conducted on normally consolidated samples (OCR = 1) were considered. The triaxial tests were completed primarily on reconstituted specimens. Table 2 presents a summary of the triaxial database used for the interpretation of critical state parameters.

405 cyclic tests are included in the laboratory database. Cyclic tests completed using the CTx and CDSS devices were considered. All tests were conducted on normally consolidated samples (OCR = 1). For CSS tests, no initial static shear stress was applied (α = 0). The CTx tests were completed on CIU samples. The cyclic tests were conducted primarily on reconstituted specimens prepared at a wide range of initial density and confining effective stress. Table 3 presents a summary of the cyclic test database used to assess liquefaction resistance.

For some soils, the monotonic tests were completed by different original investigators than the cyclic tests. It was considered that the same soil was used in each study because the gradations reported are reasonably similar.

4 EVALUATION OF CRITICAL STATE PARAMETERS

The original triaxial datasets were examined to obtain a semi-logarithmic CSL in the $e - \log p'$ space. This was done to ensure that the CSL position is consistent with our judgement based on whether the samples are contracting, dilating or at critical state at the end of shearing. For the EG sand mixtures, only the end points of the triaxial tests were available, such that the CSL is approximated as a best-fit line through the end points. Furthermore, no details of the void ratio measurement were specified. It was assumed that the void ratio at consolidation (e_0) was estimated from the specimen dimensions before the application of a confining stress, which can cause a misinterpretation in the position of the CSL. For all the other materials, e_0 was calculated from backward calculation based on the sample water content at the end of test. This can be achieved following the method by Verdugo and Ishihara (1996) or by freezing the specimen at the end of shearing as proposed by Sladen and Handford (1987). Furthermore, unless noted otherwise in the original study, it was assumed that M_{tc} is evaluated by the end of test method, in which M_{tc} is taken as the value of the stress ratio (η) at the end of shearing. M_{tc} is more accurately estimated by analyzing the stress-dilatancy behaviour (Bishop, 1966). The critical state parameters Γ_1, λ_{10} and M_{tc} are presented in Table 4. For each material, the CSL was interpreted over an approximative stress range of 50 kPa to 600 kPa.

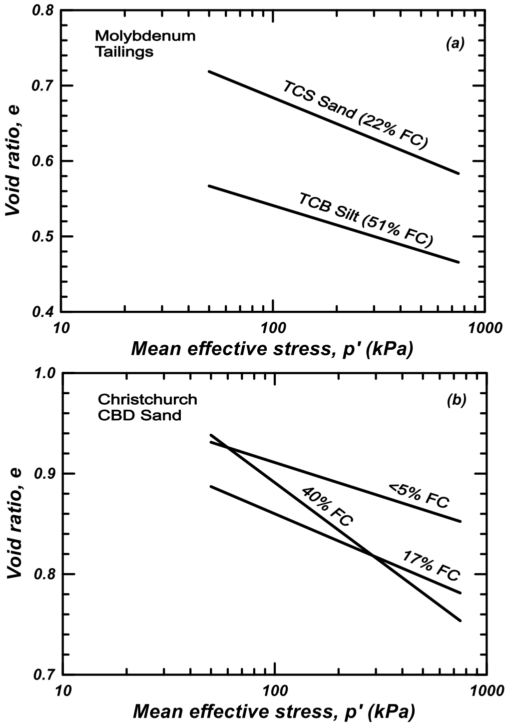

Figure 3: Critical state lines of sand-fine mixtures (a) Molybdenum tailings (b) Christchurch CBD sand

In Table 4, the value of λ_{10} does not necessarily increase for soils with higher FC, which suggests that FC may not be a direct proxy for compressibility. This is illustrated in Figure 2, where no direct trend between increasing FC and increasing λ_{10} is noticed, similarly to the observations of Olson and Stark (2003).

However, for sand-fines mixtures, the CSL does become significantly steeper once a threshold fines content is exceeded. This is readily apparent in Figure 3 from the CSLs of the Christchurch CBD sand and Molybdenum tailings mixtures. For Molybdenum tailings, for FC between 22% and 51%, there is a downward shift in the CSL but no significant change in λ_{10}. Γ_1 decreases with increasing FC and λ_{10} is practically constant. The same trend is observed for the Christchurch CBD sand with FC between 5% and

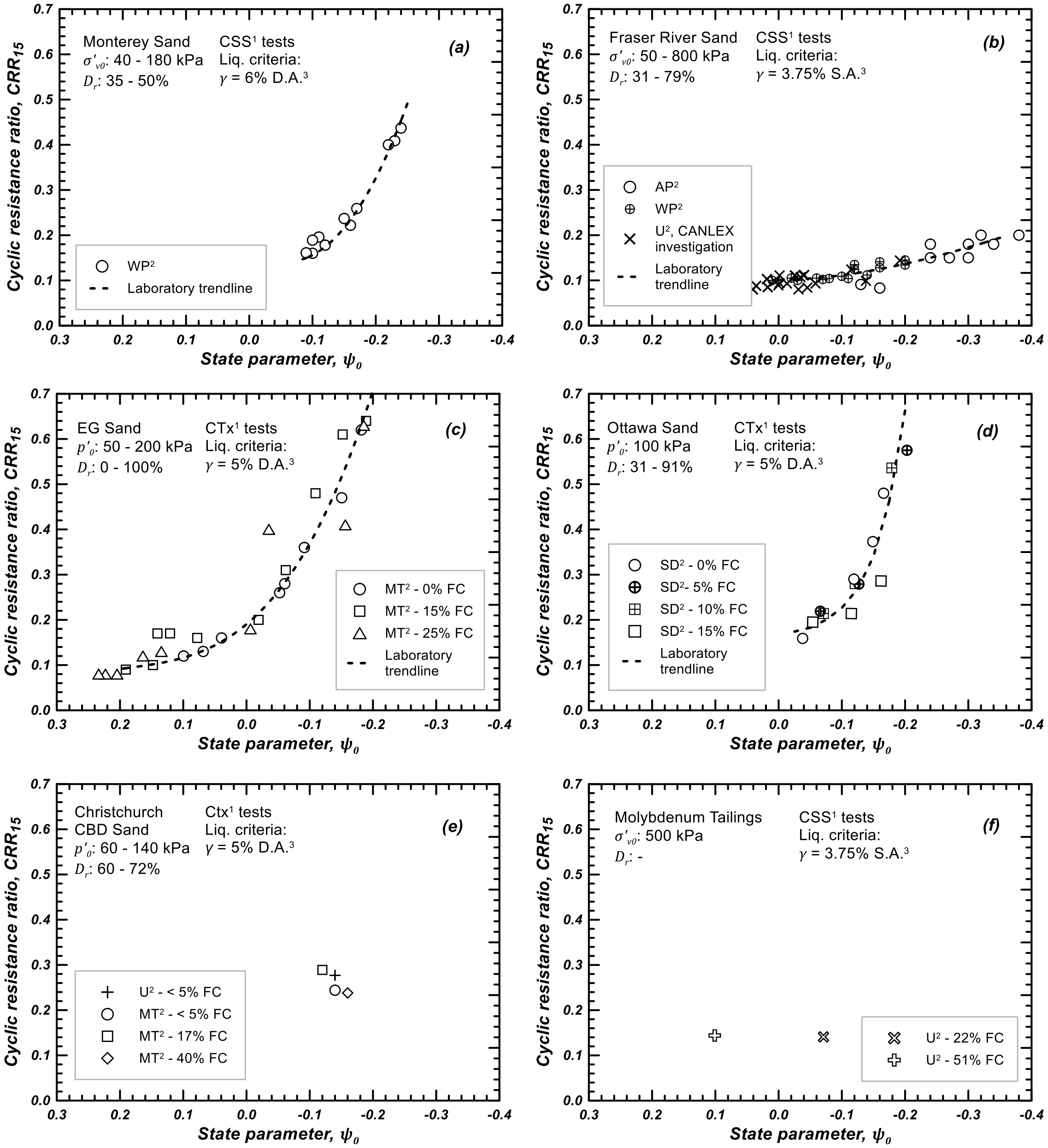

Figure 4: Cyclic resistance ratio at 15 cycles as a function of state parameter for studied materials (a) Monterey sand (b) Fraser River sand (c) EG sand (d) Ottawa sand (e) Christchurch CBD sand (f) Molybdenum tailings

[1] CTx: cyclic triaxial, CSS: cyclic direct simple shear [2] WP: water pluviation, MT: moist tamping, SD: slurry deposition, U: undisturbed

[3] S.A. : single-amplitude, D.A. : double-amplitude

17%. At 40% FC, the CSL becomes significantly steeper and shifts upwards. The λ_{10} value for the Christchurch CBD silty sand is almost twice that of the Christchurch CBD clean sand, indicating a greater compressibility.

FC_{th} is usually between 25% and 45%, with the exact threshold being material specific (Polito and Martin, 2001).This transition happens for FC over 51% for the Molybdenum tailings, and between 17% and 40% for the Christchurch CBD sand.

For the sand-fines mixtures in Table 4, M_{tc} is largely unaffected by particle size distribution. The variation in M_{tc} is within the accuracy ranges identified by Ghafghazi and Shuttle (2009), except for the Ottawa sand mixtures. This can partly be due to inaccuracies in the end of test method.

Gill et al. (2018) assessed the influence of FC on critical state parameters for five gradations of one tailings material with FC between 0% and 100%. The study yielded the same conclusions regarding the position of the CSL in the e-$\log p'$ space and the relatively constant value of M_{tc}. The λ_{10} values for these tailings are also included in Figure 2, which shows that FC_{th} is between 30% and 60%.

5 INFLUENCE OF FINES CONTENT ON CYCLIC RESISTANCE

Figure 4 presents a summary of the cyclic test results conducted on all the materials studied. The testing apparatus, sample preparation methods, the range of D_{r0} and σ'_{v0} or p'_0 are also presented in Figure 4. The liquefaction criteria are 5% D.A. for CTx tests or 3.75% S.A. for CDSS tests except for Monterey Sand. For each material, a strong correlation between CRR_{15} and ψ_0 exists in which CRR_{15} decreases with increasing ψ_0.

For Monterey sand, Fraser River sand, EG sand and Ottawa sand, the cyclic tests were completed at different initial conditions of stress and density, as detailed in Table 3. The variation of D_{r0} is around 30%, while the confining stress generally covers the range of 50 kPa to 200 kPa. A unique CRR_{15}-ψ_0 relation is established for each material. Unlike the simplified method, state-parameter based cyclic resistance curves are unaffected by initial density and consolidation stress, which suggests that ψ normalizes density and stress effects on CRR_{15}.

For the EG sand and Ottawa sand mixtures, the CRR_{15}-ψ_0 relation is independent of FC. In all these mixtures, FC is presumably below FC_{th} where sand particles dominate the soil matrix.

For Fraser River Sand, the increase in cyclic resistance of reconstituted specimens is smaller than all other materials for ψ_0 between -0.1 to -0.35. The Fraser River sand data included cyclic resistance evaluated for samples reconstituted with various preparation techniques and for frozen samples. The effect of fabric appears negligible, although specimen preparation is generally expected to affect cyclic resistance (e.g., Mullilis et al., 1977).

Figure 5 presents a comparison of the 'equivalent field' cyclic resistance curves for all the materials studied. The 'equivalent field' curve is obtained from the laboratory-based trendlines interpreted in Figure 4, and considers the effect of K_0 and bi-directional loading (Idriss and Boulanger, 2008).

$$CRR_{field} = 0.9\left(\frac{1+2K_{0(field)}}{3}\right)CRR_{CTx} \qquad [4]$$

$$CRR_{field} = 0.9\left(\frac{1+2K_{0(field)}}{1+2K_{0(CSS)}}\right)CRR_{CSS} = 0.9CRR_{CSS} \qquad [5]$$

for CTx and CDSS tests, respectively. K_0 is approximated using the Jacky (1944) equation ($K_0 = 1 - \sin\varphi$) and φ_{cs} values in Table 4.

In Figure 5, the cyclic resistance curves are soil-specific as expected. At denser initial states, CRR_{15} is lower for soils that are more compressible, i.e. for which the λ_{10} value is higher. The curvature of the CRR_{15}- ψ_0 trendline is more pronounced for materials that are less compressible, or for which the λ_{10} value is smaller. For the practicing geotechnical engineers, the curvature is an important aspect of the CRR_{15}- ψ_0 relation. Indeed, it may influence in the decision of a field soil improvement method if density alone does not have a significant positive impact on the estimated cyclic resistance. However, from a practical point of view, density is usually determined from penetration tests such as the CPT, and the relation between the state parameter and CPT results is the other piece of this puzzle that needs to be investigated.

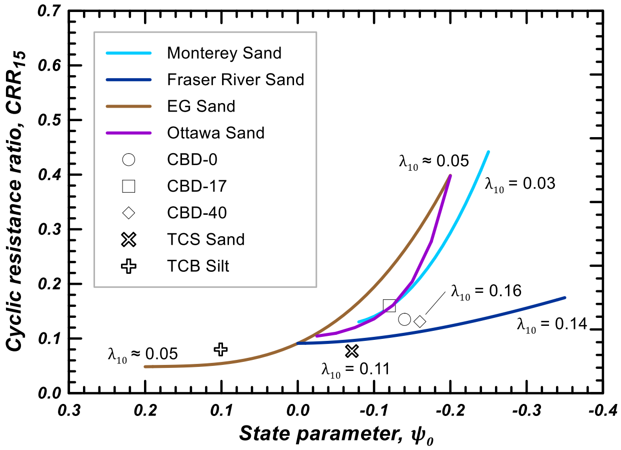

Figure 5: Comparison of equivalent field cyclic resistance curves

6 CONCLUSIONS

Laboratory test results on laboratory standard sands, natural sands to silty sands and silty mine tailings reported in literature were compiled to assess the influence of FC on CRR_{15} using the critical state framework.

There is no direct trend between FC and λ_{10}. For sand-fines mixtures, λ_{10} increases once a fines threshold is exceeded. FC_{th} defines the point at which the fines will dominate in the soil matrix to carry the loads.

The cyclic resistance curves established are soil-specific. When applying the critical state framework, ψ normalizes the effect of initial confinement. For sand-fines mixtures, the relation between CRR_{15} and ψ_0 is independent of FC. For most of the mixtures included in the database, FC is presumably below FC_{th}. Further laboratory investigation is needed to confirm if the CRR_{15}-ψ_0 relation would differ for FC exceeding FC_{th}.

The data demonstrate an influence of compressibility on cyclic resistance. For denser initial states, materials that are more compressible show a lower cyclic resistance. The curvature of the CRR_{15}- ψ_0 relation is also less pronounced for materials that are more compressible. Further testing on materials with high compressibility (λ_{10} values) can better establish the relation between compressibility and improvement in cyclic resistance with density.

7 ACKNOWLEDGEMENTS

The authors greatly appreciate the financial support of Klohn Crippen Berger, Ontario Centres of Excellence (project 27337) and NSERC (EGP 506108-2016).

8 REFERENCES

Amini, F., & Qi, G. Z. (2000). Liquefaction testing of stratified silty sands. *J. of Geotech. and Geoenv. Eng., 125*(3), 208-217.

Been, K. (2016). Characterizing mine tailings for geotechnical design. *Australian Geomechanics, 51*(4), 59-78.

Been, K., & Jefferies, M. (1985). A state parameter for sands. *Géotechnique, 35*(2), 99-112.

Been, K., Romero, S., Obermeyer, J., & Hebeler, G. (2012). Determining in situ state of sand and silt tailings from the CPT. *Tailings and Mine Waste '12,* 14-17 October, Keystone, Colorado. pp. 325-333.

Bishop, A. W. (1966). Sixth rakine lecture: Strength of soils as engineering materials. *Géotechnique, 16*(2), 91-130.

Boulanger, R. W., & Idriss, I. M. (2014). *CPT and SPT based liquefaction triggering procedures* No. UCD/CGM-14/01). University of California, Davis, California:

Bray, J. D., & Sancio, R. B. (2006). Assessment of the liquefaction susceptibility of fine-grained soils. *J. of Geotech. and Geoenv. Eng. ,* (132), 1165-1177.

Carraro, J. A. H. (2004). Mechanical behavior of silty and clayey soils. (PhD, Purdue University).

Carraro, J. A. H., Bandini, P., & Salgado, R. (2005). Liquefaction resistance of clean and nonplastic silty sands based on cone penetration resistance. *J. of Geotech. and Geoenv. Eng., 129*(11), 965-976

Castro, G., & Poulos, S. J. (1977). Factors affecting liquefaction and cyclic mobility. *J. of Geotech. Eng. Div, 103*(GT6), 501-516.

Ghafghazi, M. (2011). Towards comprehensive interpretation of the state parameter from cone penetration testing in cohesionless soils. (PhD, University of British Columbia).

Ghafghazi, M., & Shuttle, D. (2009). Confidence and accuracy in determination of the critical state friction angle. *Soils and Foundations, 49*(3), 391-395.

Gill, S., Ingabire, E., Vipulanantham, M., Siu, M., Peterson, K., Viles, K., Clelland, L., Sy, A., Ghafghazi, M. (2018). Influence of fines content on interpretation of the CPT tip reistance for liquefaction assessment in mine tailings. *GeoEdmonton 2018: 71St Can. Geotech. Conf.,* Edmonton, Canada, September 23-26, 2018.

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

Ishihara, K., Cubrinovski, M., & Nonaka, T. (1998). Characterization of undrained behaviour of soils in the reclaimed area of kobe. *Soils and Foundations,* , 33-46.

Ishihara, K., & Koseki, J. (1989). Cyclic shear strength of fines-containing sands. *Proc. of Discussion Session on Influence of Local Conditions on Seismic Response: 12th Int. Conf. on Soil Mechanics and Foundation Eng.,* Rio de Janeiro, Brazil, August 13-18, 1989. pp. 101-106.

Jaky, J. (1944). The coefficient of earth pressure at rest, in hungarian "A nyugalmi nyomas tenyezoje". [In Hungarian:] *J. Soc. Hung. Eng. Arch. (Magyar Mernok Es Epitesz-Egylet Kzlonye),*, 355-358.

Jefferies, M., & Been, K. (2016). *Soil liquefaction: A critical state approach* (2nd ed.) CRC Press.

Manmatharajan, V. (2011). Initial stress state and stress history effects on liquefaction susceptibility of sands. (MASc, Carleton University).

Mulilis, J. P., Arulanandan, K., Mitchell, J. K., Chan, C. K., & Seed, H. B. (1977). Effects of sample preparation on sand liquefaction. *J. of Geotech. Eng. Div, 103*(2), 91-108.

Murthy, T. G. S. (2006). Study of the undrained static response of sandy soils in the critical state framework. (PhD, Purdue University).

Olson, S. M., & Stark, T. D. (2003). Use of laboratory data to confirm yield and liquefied strength ratio concepts. *Can. Geotech. J., 40*, 1164-1184.

Polito, C. P., & Martin II, J. R. (2001). Effects of nonplastic fines on the liquefaction resistance of sands. *J. of Geotech. and Geoenv. Eng., 127*(5), 408-415.

Robertson, P. K., & Wride, C. E. F. (1999). *Canadian liquefaction experiment CANLEX, book D: Phase II data review report*. Canada: BiTech Publishers Ltd.

Seed, H. B., & Idriss, I. M. (1971). A simplified procedure for evaluating soil liquefaction potential. *J. of the Soil Mechanics and Foundations Div, 97*(9), 1249-1273.

Sivathayalan, S. (1994). Static, cyclic and post liquefaction simple shear response of sands. (MASc, University of British Columbia).

Sladen, J. A., & Handford, G. (1987). A potential systematic error in laboratory testing of very loose sands. *Can. Geotech. J., 24*(3), 462-466.

Sriskandakumar, S. (2004). Cyclic loading response of fraser river sand for validation of numerical models simulating centrifuge tests. (MASc, University of British Columbia).

Stamatopoulos, C. A. (2010). An experimental study of the liquefaction strength of silty sands in terms of the state parameter. *Soil Dynamics and Earthquake Eng., 30*, 662-678.

Taylor, M. L. (2015). The geotechnical characterisation of christchurch sands for advanced soil modelling. (PhD, University of Canterbury).

Thevanayagam, S. (1998). Effect of fines and confining stress on undrained shear strength of silty sands. *J. of Geotech. and Geoenv.Eng., 124*(6), 479-491.

Thevanayagam, S., Fiorillo, M., & Liang, J. (2000). Effect of non-plastic fines on undrained cyclic strength of silty sands. *Proc. of Session of Geo Denver 2000: Soil Dynamics and Liquefaction,* Denver, USA, August 5-8, 2000. *, GSP 107.* pp. 77-91.

Tringale, P. T. (1983). Soil identification in-situ using an acoustic cone penetrometer. (PhD, University of California, Berkeley).

Troncoso, J. H., & Verdugo, R. (1985). Silt content and dynamic behavior of tailing sands. *11th Int. Conf. on Soil Mechanics and Foundation Eng.,* San Francisco, USA, August 12-16, 1985. pp. 1311-1314.

Verdugo, R., & Ishihara, K. (1996). The steady state of sandy soils. *Soils and Foundations, 36*(2), 81-91.

Wu, J. (2002). Liquefaction triggering and post liquefaction deformations of monterey 0/30 sand under uni-directional cyclic simple shear loading. (PhD, University of California, Berkeley).