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A compendium of soil liquefaction potential assessment methods

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ABSTRACT

Since soil improvement techniques are not always economically feasible in large extents, accurate assessment of soil liquefaction is necessary for safe design of foundation engineering as well as post-earthquake emergency evacuation in liquefaction prone areas. Prediction of soil liquefaction potential is a very complex engineering problem because of the heterogeneous nature of soils and involvement of many varieties of factors. This problem has prompted researchers to carry out extensive research on reliable prediction of soil liquefaction. The outcomes have been discovered to be scattered in many journals, conference proceedings, and reports. This paper aims to collate the scattered information, provide a framework for the identification of suitable methods of assessment of liquefaction potential based on technical aspects, and produce a multicriteria reference table.

1 BACKGROUND

Significant collateral damages, loss of lives, and loss of lifelines have been reported in several earthquake-induced soil liquefaction events, including the Canterbury Earthquake Sequence (CES) which occurred on September 4, 2010 (M_w 7.1) and February 17, 2011 (M_w 6.2), (Cubrinovski *et al.*, 2018; NASEM, 2016; MBIE, 2016). A review of current literature shows that material characterization, in-situ state characterization, and system response studies are required to produce comprehensive assessment of soil liquefaction potential.

Generally, “soil liquefaction occurs due to rapid cyclic loading during seismic events where sufficient time is not available for dissipating the excess pore-water pressures generated through natural drainage” (Dixit *et* al, 2012, p. 2759). Consequently, the soil loses its shear strength which usually causes damage to the built environment. Loose cohesionless saturated soils that are subjected to strong shakings from earthquakes are most likely to liquefy (Towhata *et al.*, 2016). The major factors determining extent or severity of soil

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liquefaction are soil density, nearness to groundwater table, magnitude of earthquake vibrations, distance from source of the earthquake, duration of ground motion, site conditions, ground acceleration, thickness and type of soil deposits, fines content, grain size distribution, plasticity of fines, degree of saturation, confining pressure, permeability, shear modulus degradation, and reduction of effective stress (Dixit *et al*., 2012; Seed *et al*., 2003; Youd and Idriss, 2001).

The applicability and limitations of field tests (SPT, CPT, V_s and BPT) are well summarized by Youd and Idriss (2001). Assessment of liquefaction potential based on soil classification characteristics may include soil gradation (particle size analysis), fine contents (FC), plasticity index (PI), and soil index (I_C). The in-situ state parameters include several correlations such as q_{c1N} in CPT, $(N_1)_{60}$ in SPT, (V_s) in shear vane test, relative density (D_r) which is suitable for only clean sands, void ratio ($e_{max} - e_{mim}$) versus D_{50} and ($e_{max} - e_{mim}$) versus FC. The system response effects include studies of soil layer interactions, seismic history, soil-structure interactions and mechanisms that intensify or mitigate liquefaction effects and manifestations (Cubrinovski *et al*., 2018). Several studies over the years have indicated that assessment of soil liquefaction potential can be inferred often from empirical correlations of soil properties obtained from both field and laboratory tests such as cyclic resistance ratio (CRR)versus D_r, q_{c1N} versus D_r, CRR versus q_{c1N}, void ratio (e) versus fine content (FC), PGA, C_D versus ($e_{max} - e_{mim}$), and N-value versus D_r (Cubrinovski and Ishihara, 1999).

In this paper, the authors attempt to produce a compendium of assessment techniques for soil liquefaction potential as well as a framework for identification of relevant assessment techniques for liquefaction potential based on the literature database.

The first step required to design program for mitigating soil liquefaction is to nearly and accurately predict its potential in in-situ soils prior to occurrence of major earthquake event. Over the years, several approaches have been proposed to evaluate soil liquefaction potentials which are generally categorized by the NASEM (2016) as "simplified stress-based, cyclic strain-based, energy-based methods, laboratory tests, physical models, computational mechanics-based techniques, performance-based evaluation, and design methods". The above is currently the most comprehensive study carried out on soil liquefaction assessment, yet deficient in some recent advancements on discovered empirical correlations, system response analysis, and numerical methods. This paper offers a multi-criteria reference table and a summarized compendium of assessment techniques in liquefaction potential which are mostly applied in current practice.

2 LIQUEFACTION POTENTIAL ASSESSMENT METHODS

2.1 The simplified stress-based method

The simplified stress-based approach which is often referred to as the "simplified method" (or the "Seed-Idriss simplified method") is the most common applied method to evaluate liquefaction triggering in geotechnical engineering practice (MBIE, 2016; NASEM, 2016). This method uses the factor of safety (FOS) defined as ratio of the "cyclic resistance ratio" (CRR) to "cyclic stress ratio" (CSR), where CRR is a measure of the soil resistance (i.e. cyclic stress ratio required to allow liquefaction take place) and CSR quantifies earthquake loading (i.e. cyclic shear stress) (Seed and Idriss, 1971; Seed et al., 2003). Equation 1 expresses the FOS as:

$$FOS = \frac{CRR}{CSR} \tag{1}$$

Other relevant equations in the simplified stress-based approach are listed as Equation (2) to (6) where, PGA = peak ground acceleration (horizontal component); g = acceleration due to gravity; r_d = shear stress reduction factor σ_v = total overburden stress at depth z; σ'_{v0} = effective overburden stress depth z; MSF = magnitude scaling factor (obtainable in an earthquake magnitude chart) (e.g.: Seed and Idriss, 1971; Seed *et* *al*., 2003; Youd and Idriss, 2001); M_w = magnitude of earthquake moment; where V_{S1} is stress corrected shear wave velocity; V_{S1}^{*} = limiting upper value of V_{S1} required for cyclic liquefaction to happen and is usually varies between 200 and 215m/s; P_a = reference stress (i.e.100kPa). Seed *et al*. (2003) provided an updated liquefaction triggering curves based on SPT - $(N_1)_{60}$ values as shown in Figure 1. These curves were produced from case-history test points (more than 1971 version) and are therefore, more comprehensive for soil samples containing either "clean sand" and with adjustment for (FC). Similar correlation have been developed and modified over the years for q_{c1N} in CPTs', V_{s1} in shear vane tests, and BPT (e.g. Youd and Idriss, 2001).

$$CSR = 0.65 \times \frac{PGA}{g} \times \frac{\sigma_v}{\sigma'_{v0}} \times r_d \tag{2}$$

$$r_d = 1 - 0.00765z, \text{ for } z < 9.2m \tag{3a}$$

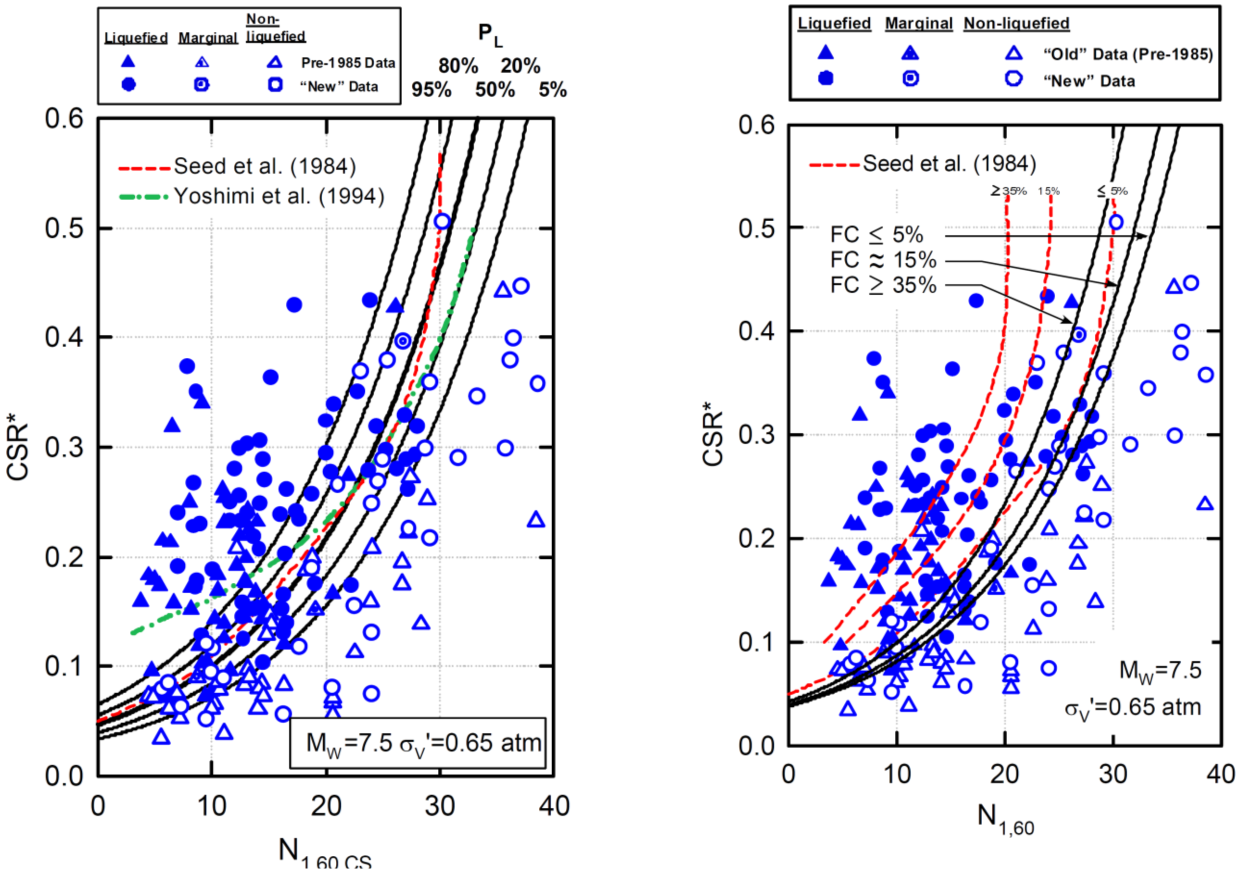
$$r_d = 1.174 - 0.0267z, \text{ for } z \geq 9.2m \tag{3b}$$

$$MSF = \left(\frac{M_w}{7.5}\right)^{-2.56} \tag{4}$$

$$CRR = \left[0.022\left(\frac{V_{S1}}{100}\right) + 2.8\left(\frac{1}{V_{S1}^{*} - V_{S1}} - \frac{1}{V_{S1}^{*}}\right)\right] \times MSF \tag{5}$$

$$V_{S1} = V_S\left(\frac{P_a}{\sigma'_{v0}}\right)^{0.25} \tag{6}$$

In summary, this method states that, liquefaction will occur when $FOS \leq 1$ and will not when $FOS > 1$.



(a) for clean sands

(b) with adjustment for fine content shown

Figure 1: SPT $(N_1)_{60}$-based Liquefaction triggering charts for M_w=7.5 & σ'ᵥ =0.65atm (Seed et al., 2003).

The simplified stress method is suitable for clean sands, has capacity to account for concerned soil layer depth, water table depth, magnitude of seismic shaking, (Seed *et al*., 1983). However, notable limitations of this approach are can be expensive because of required in-situ tests and obtaining undisturbed samples from sites can be difficult. Although, adoption of field performance correlations can make up for the above deficiency.

2.2 The strain-based method

The strain-based approach which was originally proposed by Dobry *et al.* (1982) has been reviewed by several authors (for instance: Baziar *et al*., 2011; Dobry and Addoun, 2015; NASEM, 2016). Figure 2 shows the observed correlation between pore pressure and cyclic shear strain by Dobry and Addoun (2015). This approach uses a small cyclic shear strains (γ_c) (in the order of 1%) to replace the CSR normally used in the simplified stress approach. The strain-based approach as summarized by Dobry *et al*. (1982) p. 23 are as follows:

Step1: Determine γ_c and n where γ_c is cyclic shear strain, magnitude of earthquake M is related to n, n is number of cycles of uniform cyclic stress τ_c (e.g.: Youd & Idriss, 2001). γ_c is obtained from Equation 7:

$$\gamma_c = 0.65 \times \frac{PGA}{g} \times \frac{\sigma_0 r_d}{G_{max}\left(\frac{G}{G_{max}}\right)\gamma_c} \tag{7}$$

Where PGA is peak ground acceleration (horizontal component), g is acceleration due to gravity, r_d is shear stress reduction factor already defined in Equation 3, σ_0 is total overburden pressure at depth z, G_{max} is shear modulus of the soil at very small cyclic strain ($\gamma_c = 10^{-4}$percent), $\left(\frac{G}{G_{max}}\right)\gamma_c$ is effective modulus reduction factor of soil relating to cyclic strain, γ_c .

Step 2: Compare γ_c and threshold strain (γ_t): When $\gamma_c < \gamma_t$, Both pore pressure build-up and liquefaction will not occur.

Step 3: "When $\gamma_c > \gamma_t$, the values of γ_c and n should be applied together with experimental curves to estimate the value of the pore pressure build-up at the end of the earthquake, $\left(\frac{\Delta u}{\sigma_0'}\right)$ where Δu is the change in pore water pressure, and σ_0' is initial effective overburden stress at depth z" *(Dobry et al.*, 1982, p. 23)

Step 4: $\frac{\Delta u}{\sigma_0'}$ calculated in step (3) above is used to analyse when initial liquefaction will be experienced i.e. $\left(\frac{\Delta u}{\sigma_0'} = 1.0\right)$ or not $\left(\frac{\Delta u}{\sigma_0'} < 1.0\right)$.

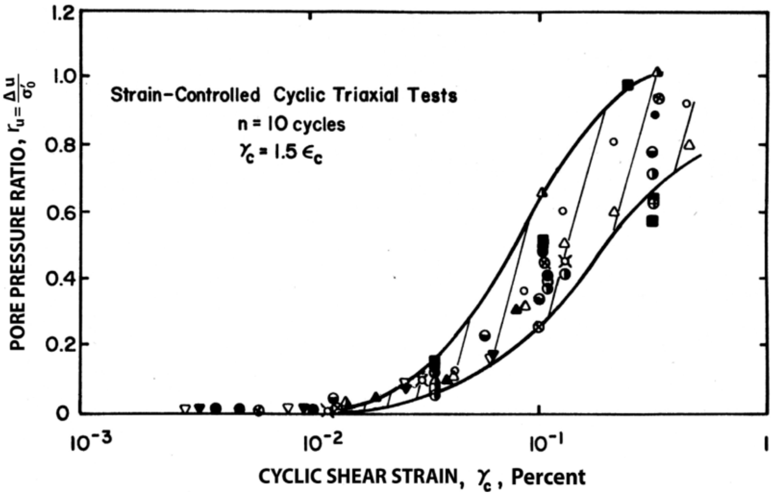


Figure 2: Correlation of pore pressure versus cyclic shear strain γ_c (after Dobry and Addoun, 2015)

The strain-based approach is suitable for clean sands with ($PI = 0$), theoretically reasonable, estimates initial pore pressure, can estimate complicated stress-strain history on build-up of pore pressure. The limitation of this method is greater difficulty of estimating the cyclic strain compared with the cyclic shear stress-approach (Seed *et al*., 1983).

2.3 Energy-based methods

Davis and Berrill (1982) introduced an energy-based approach for liquefaction potential evaluation in which the energy content of an earthquake is compared with the amount of dissipated energy required for soil liquefaction, known as "capacity energy" and this proposed model was further updated in Berrill and Davis (1985). Energy based approaches of assessing liquefaction triggering are numerous and have been well reviewed and studied by several authors, (for instance: Baziar *et al*., 2011; Figueroa *et al*., 1994; Green and Mitchell, 2004; Lasley *et al.*, 2017). The systemic strain energy based method of liquefaction evaluation is well outlined in (Baziar *et al*., 2011). Both the energy demand and capacity were estimated from the stress-strain hysteresis loop. The applicability of energy-based evaluation techniques is suitable for cohesionless sand under cyclic loading. The observed limitation in this method is shared from those involved in both stress and strain methods of liquefaction assessment.

2.4 Laboratory and in-situ testing method

Previous studies (for instance: Bray *et al*., 2004; Cubrinovski & Ishihara, 1999; MBIE, 2016; Seed *et al.*, 2003; Youd and Idriss, 2001) have shown that index properties (liquid limit, plasticity index); maximum and minimum density (or unit weight); stress-strain studies from consolidated undrained cyclic triaxial tests and void ratio can be used to determine the liquefaction susceptibility of soils. A summary of liquefaction assessment based on plasticity chart as found in Seed *et al.* (2003) is shown in Figure 3. In this plasticity chart, soils within zone A and B should be considered liquefiable while soils outside zones A and B (i.e. zone C) are considered non-liquefiable soils (although they should be checked for sensitivity). Also, the modified Chinese criteria have been used to assess liquefaction potential of clayey soils in past decades and is summarized under section 4(c) in Table 1. The graphical representation of modified Chinese criteria as found in Bray *et al*. (2004) is shown in Figure 4. The applicability of the Chinese criteria is limited to clayey soils and not suitable for sands.

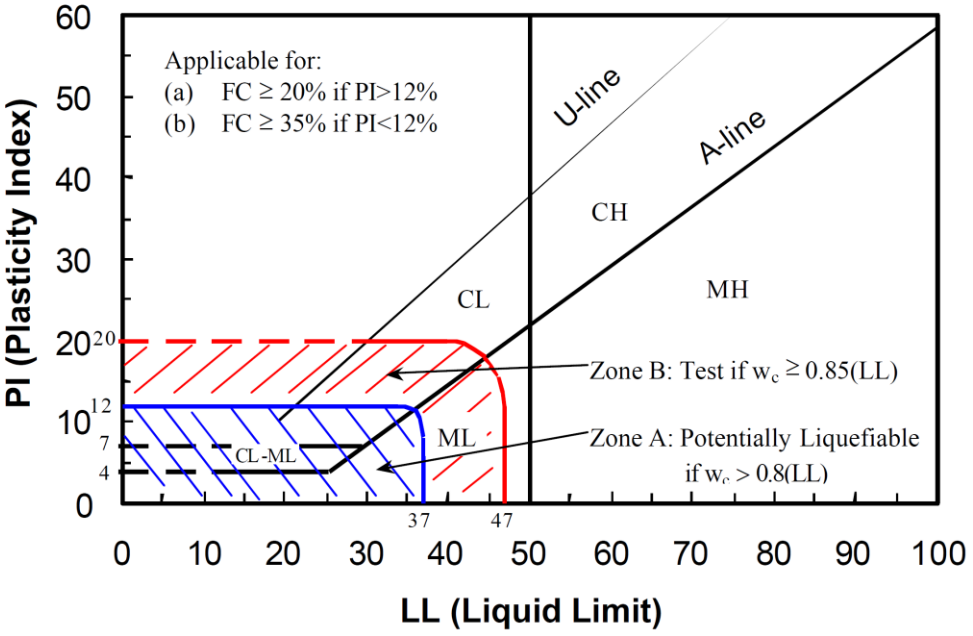
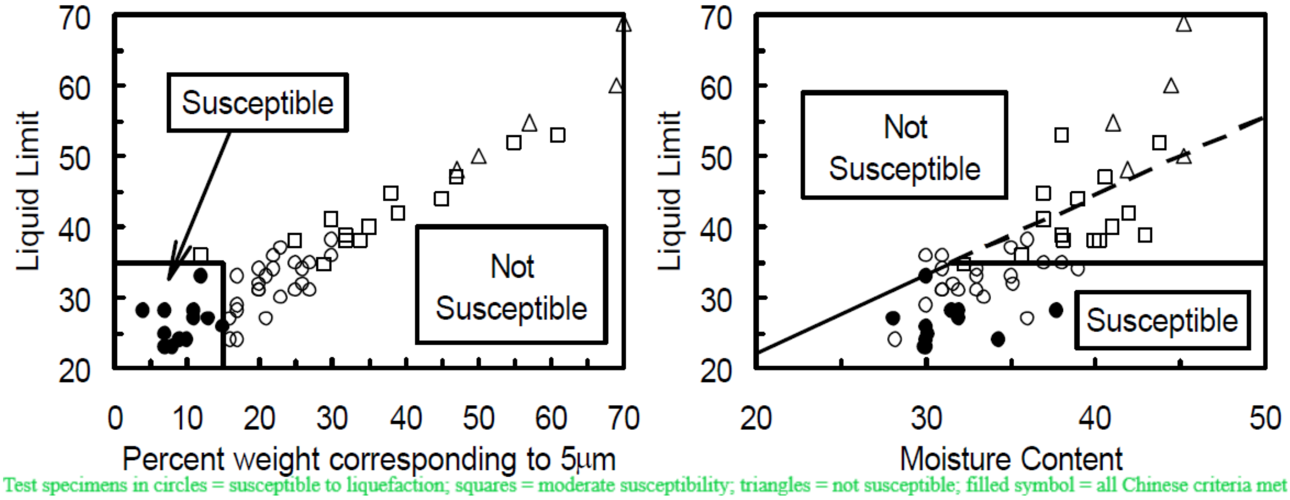


Figure 3: Proposed plasticity chart for assessment liquefaction-prone soils (after Seed et al., 2003)

The D_r-approach of liquefaction assessment is only suitable for clean sands; index properties are useful for clayey soils; the void ratio approach provides insights on sands-FC (silty sand); undrained triaxial test has capabilities to simulate liquefaction on small scale. Generally, limitations of laboratory tests include failure to account for void redistribution during “undrained” in field; sample reconsolidation which is always higher than in-situ state; and requires large correction to account for sampling and densification before shearing takes place in the field situations. The above issues can be solved by executing soil laboratory tests at fields’/in-situ effective stress level (Seed *et al*., 2003).



Test specimens in circles = susceptible to liquefaction; squares = moderate susceptibility; triangles = not susceptible; filled symbol = all Chinese criteria met

Figure 4: The modified Chinese criteria graphical representation (after Bray et al., 2004)

The most common in-situ soil assessment method for soil liquefaction in current practice are SPT, CPT, V_{s1}, and BPT. The soil behaviour index (I_C) originally proposed by Robertson and Wride (1998) which has some modified versions was recommended by Youd and Idriss (2001) to evaluate liquefaction susceptibility of soils. I_C is evaluated by Equation 8, where Q_t is normalized cone tip resistance, and F is normalized friction ratio.

$$I_C = \sqrt{3.47 - \log_{10} Q_t)^2 + (\log_{10} F + 1.22)^2} \tag{8}$$

The detailed I_C-criteria for liquefaction evaluation as found in MBIE (2016) are summarized in Section 5 0f Table 1. The applicability and advantage of the SPT approach over CPT is that soil samples can be retrieved for evaluation of FC, soil gradation, and index properties while the soil properties can only be inferred by the cone tip and sleeve frictional resistance. CPT offers cost efficiency and result consistency over the SPT method (Seed *et al*., 2003).

2.5 Physical modelling testing method

The use of physical model testing for liquefaction assessment usually involve either a “1-g” shake table testing or a dynamic centrifuge model testing and is well outlined in NASEM (2016). The application of shake tables and geotechnical centrifuge in evaluating liquefaction potential have been extensively studied (e.g. Elgamal *et al*., 1996; Iwasaki *et al*., 1984; Yasuda *et al*., 1992). Physical modelling tests are normally used to obtain soil parameters required for coupling constitutive numerical models in the validation and verification process of liquefaction potential assessment. However, associated limitations of this approach are like those described above for soil laboratory testing.

2.6 Computational mechanics-based method

Beaty and Perlea (2011) reported that two different techniques were available for predicting earthquake-induced pore-water-pressures, including (1) a loose coupled approach based on strains, and (2) a fully coupled approach based on plasticity-encoded constitutive model for predicting both pore pressure response of soil and stress-strain. Models that apply the loose couple approach are not very robust for predicting liquefaction triggering but fully coupled approach is more robust and popular within plasticity concepts to predict the pore-pressure response and stress-strain inside soil fabric (NASEM, 2016).

For instance, constitutive nonlinear models commonly used to simulate liquefaction currently include uncoupled models e.g.: Moore, Dames model (Dawson *et al*., 2001) to the relatively fully coupled models like UBCSand model (Beaty and Byrne, 2011); DM04 model (Dafalias and Manzari, 2004); PM4Sand model (Boulanger and Ziotopoulou, 2015). Current state-of-the-art in liquefaction computational geomechanics already have been well reviewed by Boulanger and Ziotopoulou (2015). Although, results obtained from software can be too generic especially if adequate data are not fully coupled in the computer code, but they currently offer the verification means for most liquefaction studies.

2.7 Performance-based and system response method

The performance-based techniques of liquefaction assessment is mainly based on probabilistic analysis, Bayesian analysis (Kayen *et al*., 2013), deterministic procedures, reliability-based assessments (Kayen *et al*., 2013). This topic is also well documented by NASEM (2016). System response-based liquefaction assessment is well documented by Cubrinovski *et al*. (2018) where the CPT data were used to characterize liquefied (YY) and non-liquefied (NN) soil layers based on existing semi empirical approach. Critical layers were determined for simplified profile of the study site with $q_c, I_c, and\ FOS$ -values from the 2010-2011 CES. Important correlations for analysis includes the normalized cone tip resistance q_{c1N-cs}, (CSR) versus number of cycles (N_c) and the effective pore water (EPWP) ratio (u_E/σ'_{v0}), maximum shear strain (γ_{max}). Notable advantages of the above approach is its capability to evaluate uncertainties, damages, and possible risks of liquefaction. However, the usually required large amount of data for precison evaluation is its major limitation.

3 DISCUSSION

The simplified stress-based procedure originally implemented by Seed and Idriss (1971) was observed to be more commonly applied in recent geotechnical engineering assessment of liquefaction triggering, while the strain-based approach introduced by Dobry *et al.* (1982) is less popular than the former due to cumbersomeness of evaluating the strain amplitude. All the energy-based approaches quantify seismic demand and liquefaction resistance in form of energy. Energy-based methods have potentials of using both strengths of stress and strain concepts and does not require magnitude scaling factor. Laboratory test can be very useful to understand the mechanics and physics of soil liquefaction especially when applied in conjunction with in-situ field tests such as CPT, SPT, Shear vane, and BPT. Several correlations between soil properties and these in-situ tests have been well documented in relevant references. Computational geomechanics is relatively a new concept gaining fast progress in research and development by the implementation of popular liquefaction models in software such as FLAC, OpenSees, PLAXIS, etc. The performance-based methods of evaluating liquefaction mostly applies the probabilistic, deterministic or Bayesian updating methods not only for liquefaction predictions but also to assess the physical damage, severity, uncertainties, and associated risks.

Table 1: Some Merits and Limitations of Liquefaction Assessment Methods

Section.	Liquefaction Assessment Methods	Evaluation Criteria	References
1.	Simplified stress	• $FOS = \frac{CRR}{CSR}$ • Liquefaction occurs if $FOS \leq 1$ • No liquefaction if $FOS > 1$	(Seed & Idriss, 1971; Youd & Idriss, 2001)
2.	Strain-based	• No liquefaction if $\gamma_c < \gamma_t$ • Initial liquefaction if $\frac{\Delta u}{\sigma_0'} = 1.0$	(Andrus & Stokoe, 2000; Baziar *et al*., 2011; Dobry & Abdoun, 2015; Dobry *et al*., 1982)
3.	Energy-based	• Effective confining stress is proportional to energy per unit volume • Stress-strain hysteresis loop (area) is equivalent to dissipated energy • $\delta w \uparrow with \uparrow D_r$ & $\delta w \uparrow with \uparrow \sigma'_3$	(Davis & Berrill, 1982; Figueroa *et al*., 1994; Green & Mitchell, 2004; Lasley *et al*., 2017)
4.	Laboratory tests (a) Plasticity index (PI)	• $PI < 7$: Susceptible to liquefaction • $7 \leq PI \leq 12$: Potentially susceptible • $PI > 12$: Not susceptible to liquefac.	(MBIE, 2016; Seed *et al*.,, 1983)
	(b) Atterberg limits	• $PI > 12$ & $W_c > 0.85LL$: Susceptible to liquefaction • $12 < PI < 20$ & $W_c > 0.85LL$: More resistance to liquefaction but prone to cyclic mobility	(Bray *et al*., 2004)
	(c) Modified Chinese criteria	• Cohesive soils plotting above A-line can liquefy if 1. % finer than 0.005mm ≤ 15% 2. $LL \leq 35\%$ 3. Water content $w \geq 0.9LL$	(Seed *et al*., 2003)
	(d) Relative density (D_r)	• $q_{c1N-}D_r$ Correlations • $CRR - D_r$ Correlation • $SPT\ N - value\ and\ D_r$ Correlations	(Cubrinovski & Ishihara, 1999)
	(e) Void ratio ($e_{max} - e_{\min}$)	• ($e_{max} - e_{\min}$) versus D_r correlation • ($e_{max} - e_{\min}$) versus FC correlation	(Cubrinovski & Ishihara, 1999, 2002)
5.	In-situ methods CPT, SPT, Vs, BPT, PGA-CSR correlations	• $If\ I_c > 2.6$: too clay-rich to liquefy • $If\ I_c \geq 204$: test soil to confirm or assume liquefiable • $If\ I_c > 2.6$ & $F < 1\%$, soil should be tested or assumed liquefiable • Updated liquefaction triggering charts are available for estimating CRR and CSR.	(MBIE, 2016; Seed *et al*., 1983; Seed *et al*., 2003; Youd & Idriss, 2001)

Note: ↑ means increases

A multicriteria reference table for the frequently applied liquefaction potential assessment methods in current practice is presented in Table 1. The modified SPT-chart shown in Figure 1, according to Seed *et al*. (2003) have been improved on all correction factors, more points than previous versions have been used to develop the charts, and have been improved by *Bayesian* updating to reduce possible associated uncertainties. Also, past studies have derived several empirical correlations between relevant soil properties in order to fully understand the mechanisms and physics of soil liquefaction. In recent years, useful relationships have been developed for plasticity index, Atterberg limits, relative density, void ratios and the Chinese criteria have equally been modified to enable a more accurate liquefaction evaluation for clayey soils. Also, some fair correlation was reported between PGA and CSR at shallow deposits in Christchurch which agrees with the simplified liquefaction evaluation method in Youd and Idriss (2001). Furthermore, it is worthwhile to consider further studies on the proposed "system response analysis of liquefiable deposits" Cubrinovski *et al.* (2018). The applied effective stress analyses used in the above study demonstrated some significant related mechanisms of the studied layers.

4 CONCLUSION/RECOMENDATIONS

This paper summarizes the current state-of-the-art of liquefaction assessment methods in earthquake engineering practice. Details of previous studies have been scattered in several publications and no straightforward collation of liquefaction triggering assessment methods is available in the literature database. Hence, the significance and justification for this paper. Although, the space constraint of this conference paper has not allowed a comprehensive discussion on all liquefaction triggering evaluation methods, more technical details of relevant approach can be obtained from the cited resources in each technique.

Most of the discussed assessment methods require further research to increase their accuracy of predictions for both liquefactions triggering and consequences/severity. Furthermore, it was observed that most studies on liquefaction assessments focused either on "pure sand" or "clay soil" but in real life scenarios, soils may also consist mixture of both soil types. Therefore, it is recommended that studies on mixed soil properties should be carried out for liquefaction triggering assessments. Interdisciplinary research based on sound science and engineering mechanics is encouraged between the geotechnical engineers, engineering geologists, seismologists and physicists to improve the current research trend.

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