Liquefaction Resistance of Sand Improved with Enzymatically Induced Calcite Precipitation based on Laboratory Investigation

Minson Simatupang
Department of Civil Engineering, Halu Oleo University, Kendari 93232, Indonesia

Corresponding Author: minson.simatupang@uho.ac.id

Abstract. Liquefaction possibility assessment of soil in field is a prominent aspect on geotechnical engineering practice. Varieties of evaluations have been developed including penetration-based methods and shear wave velocity methods over the years since Seed & Idriss (1971) introduced the simplified method based on SPT data. With the focus on natural soil, those methods have been well evolved and have become the most frequent techniques used for evaluating liquefaction possibility of soil in situ. However, the capability of those empirical relations for evaluating liquefaction possibility of ground improved with EICP approach needs authentication. This research presents the capability of the available liquefaction evaluation charts for evaluating liquefaction possibility of ground improved with EICP approach. Investigations were conducted using cyclic triaxial apparatus integrated with bender element to evaluate liquefaction resistance, \( R_c \), of EICP-improved sand as a liquefaction countermeasure. Parameters used were confining pressures, degrees of saturation during curing, calcite contents, and particles size of the sand. The test results were compared with the field performance charts of natural soils introduced by investigators. The results show that slight amount of the precipitated calcite can improve notably the maximum shear modulus of soils directly associating to \( V_s \) improvement. The correlation between corrected resistance to liquefaction \( CRR \) (cyclic resistance ratio) and adjusted shear wave velocity \( V_s \), of EICP-improved sands, \( CRR-V_s \) correlation, tends to represent different tendency with that of unimproved sands. It appears that available liquefaction prediction curves are not capable for evaluating liquefaction possibility of EICP-improved ground.

Keywords: Liquefaction resistance; maximum shear modulus; shear wave velocity; EICP-improved ground.

1. Introduction
Enzymatically induced calcite precipitation (EICP) is a new grouting approach for liquefaction mitigation. As a novel technique, the performance of this approach for reducing possible damage due to earthquake needs evaluation. For soils in nature, many approaching using penetration data [1–4] and shear wave velocity (\( V_s \)) [2–6] in the form of liquefaction charts have been developed. Those charts are commonly applied for predicting liquefaction possibility of soil in field. However, the efficacy of the available charts for assessing liquefaction possibility of ground improved with EICP needs authentication.
The objectives of this research are: 1) to create a connection between liquefaction resistance and shear wave velocity of ground improved with EICP approach based on data taken from the experiment in the laboratory, 2) to match the data obtained with the field performance chart of liquefaction resistance of native soils introduced by researchers. Fulfilling those objectives, the data prepared by Simatupang and Okamura (2017) [7] and Simatupang et al. (2018) [8] are benefitted, those data, liquefaction resistance $R_L$ and maximum shear modulus $G_0$ of EICP-improved sands, are used for creating a chart showing a correlation between corrected resistance to liquefaction and shear wave velocity named cyclic resistance ratio (CRR) and $V_s$ respectively. The comparison between the existing charts of natural soils and the created points of EICP-improved sands are shown in the same graph to confirm the applicability of those existing charts for predicting liquefaction possibility of EICP-improved ground. For those goals, a series of cyclic triaxial shear tests under undrained condition were performed on the EICP-improved sands under five parameters of testing: particle size of the sand, confining pressure ($CP$), calcite content ($CC$), and degree of saturation during curing ($S_r$). The influences of those factors on the relationship between resistance to liquefaction and shear modulus are revealed.

2. Data and method
The sands used in this research were Keisha No. 4 (Kitanohon Sangyo Co. Ltd.) and Toyoura sand [9]. All data of those sands, physical properties and distributions of grain-size, are presented in Table 1 and Fig. 1 respectively.

| Table 1 Physical properties of sand used [8] |
|---------------------------------------------|
| Keisha No.4 | Toyoura sand |
| Specific Gravity | $G_s$ | 2.65 | 2.64 |
| Max. void ratio | $e_{\text{max}}$ | 0.804 | 0.973 |
| Min. void ratio | $e_{\text{min}}$ | 0.605 | 0.609 |
| Mean grain size | $D_{50}$ | 0.825 | 0.17 |

Figure 1. Distribution of grain-size of sand used [8].

Both sands data obtained in the laboratory were used in this research. Other information relating to liquefaction resistance, $R_L$, and maximum shear modulus, $G_0$, are based on the test results prepared by
Simatupang and Okamura (2017) [7] and Simatupang et al. (2018) [8]. For obtaining \( R_L \) and \( G_0 \), the specimens were prepared and tested carefully, as follows. The dry sand and solution were mingled evenly in a plastic bag with a specific amount, as presented in Table 2. Tamping on five layers was then conducted directly on pedestal to shape a sample with the target relative density of 50%. The time consuming till tamping process was completed was less than 30 minutes. This was performed for reducing bonding detriment during sample preparation.

### Table 2. The amount of material used

| \( S_r \) (%) | Dry sand | Solution |
|--------------|----------|-----------|
|              | Name     | Amount (g) | volume (ml) |
| 30           | Keisha No.4 | 315       | 25         |
|              | Toyoura   | 300       | 27         |
| 97           | Keisha No.4 | 315       | 84         |

After tamping, the specimen was loaded isotropically during curing of around six hours, the sufficient time for 100% efficiency of calcite precipitation, in the triaxial cell. Up this stage, \( S_r \) was maintained around either 30% or 97% by encasing the specimen with membrane. The specimen was then tested directly after fully saturated condition was achieved by flowing de-aired tap water from the bottom part of the pedestal. De-aired water was streamed slowly through the specimen to the exit hole in the top part. Investigation was started using bender element just prior to liquefaction test for \( G_0 \) and \( R_L \) measurement respectively. Hereinafter, the weight of the precipitated calcite in the sample was measured by the acid washing method.

The cementation process is started by hydrolyzing urea \([\text{CO(NH}_2\text{)}_2]\) became carbonate ion \((\text{CO}_3^{2-})\). With the present of calcium ion \((\text{Ca}^{2+})\) from calcium chloride \((\text{CaCl}_2)\), their combination produces the precipitated calcium carbonate \((\text{CaCO}_3)\), as expressed in Equation (1) to (3). It bonds soil particles and restricts their movement.

\[
\text{CO(NH}_2\text{)}_2+2\text{H}_2\text{O} \rightarrow 2\text{NH}_4^+ + \text{CO}_3^{2-} \quad (1)
\]
\[
\text{CaCl}_2 \rightarrow \text{Ca}^{2+} + 2\text{Cl}^- \quad (2)
\]
\[
\text{Ca}^{2+} + \text{CO}_3^{2-} \rightarrow \text{CaCO}_3\text{�}(\text{precipitation}) \quad (3)
\]

### 3. Result and discussion

#### 3.1 Test results

The test results of \( R_L \) and \( G_0 \) are shown in Fig. 2 in the form of relationship between \( R_L \) and \( G_0 \) in any cases of confining pressures and calcite content.
Those graphs at Fig.2 clearly show that $R_L - G_0$ relationship move right with CP and increase with CC. It seems that improvement ratio on $R_L$ is more significant at low CP and slow down at higher CP for both sands used, as presented at Fig.2(a). Vice versa, $G_0$ shows a good trend on improvement at higher CP and it weakens at low CP. This is due to the behavior of sands always maintaining its volume under undrained cyclic shear loading. Sand tends to experience dilate (volume increase) at low CP, consequently a decrease in pore pressure. At high CP however, sand performs a tendency to contract (volume decrease) and pore pressure increases simultaneously leading to liquefaction. In this case, the build up excess pore pressure would break calcite bonding and the occurrence of sample liquefy could not be avoided. In the small strain level where $G_0$ is determined, the soil skeleton is more compact and stable at higher CP due to the higher pressure receiving by particles. In this position, the build up excess pore pressure is very low which does not interfere the bond among soil particles.

At higher $S_{nc}$, $R_L - G_0$ relationship of $CC_E = 0.8$ is almost coincided with that of $CC_E = 0.4$ at lower $S_{nc}$. This fact illustrates that by lowering $S_{nc}$ the amount of the expected calcite precipitated can be reduced significantly for achieving a specific $R_L - G_0$ value.

3.2. Relationship between $R_L$ and $G_0$

To investigate the possibility of a good correlation between $R_L$ and $G_0$, their relationship at various CP of both sands tested, Keisha No.4 and Toyoura sand, have been examined as shown in Fig.3.
That $R_L$ against $G_0$ moves to the right side depicted at Fig.3 concomitant with the escalate in $CP$ shows the stress level dependency. The correlation shown at that figure gives indication that relationship between $R_L$ and $G_0$ is differentiated based on $CP$. Different sands depicted at that figure also denotes the same trend. What is displayed at Fig.3 agrees with the investigation results on untreated Toyoura and Niigata sand prepared by Nishi et al. (1988) [10] and Tokimatsu and Uchida (1990) [11] respectively. The results of their test on Niigata sand is provided in Fig. 4. Specimens relative density ranged from 48% to 100% were prepared and tested at confining pressures of 0.5, 1, and 2 kgf/cm². Fig.4 proves clearly that $R_L - G_0$ relationship is differentiated based on $CP$, as aforementioned.

![Figure 4. Effect of $CP$ on the relationship between $R_L$ and $G_0$ [10,11]](image)

Like CP, $R_L - G_0$ relationship is also affected by type or grain size of the sand. As shown at Fig.5, different type or grain size of the sand performed different trend (non-unique correlation), though the investigations were performed in the same $CP$. It seems that $R_L - G_0$ relationship of Toyoura sand which has grain size five times smaller is sharper than that of Keisha No.4. This fact is clearer on specimen with lower $CP$. The same thing was also reported by Tokimatsu and co-workers [11,12], as shown in Fig.6. They conducted investigation on untreated Toyoura and Niigata sand with relative densities of the samples were in the range of 48–100%. Based on achievement of their investigation, one interesting point have been concluded that different densities and sample preparations prepared in the same soil perform a good $R_L - G_0$ correlation. Those facts indicate that liquefaction prediction based on maximum shear modulus can be stated as a prospective clue. Even though, neither different confining pressures nor densities shows a specific $R_L - G_0$ correlation.
Figure 5. $R_L$ against $G_0$ at different kind of samples

On those challenges, Tokimatsu and co-workers adopted the concept of normalization. They try to normalize $G_0$ regarding effective CP and void ratios [11,13] in their work for creating a unique $R_L$ - $G_0$ relationship of unimproved sands based on investigation in the laboratory. This proposed idea was adopted in this study.

Figure 6. Effect of different soils on $R_L$ - $G_0$ relationship [12]

3.3. Relationship between $R_L$ and $G_0$ based on laboratory investigation

For evaluating liquefaction possibility of natural soils in situ many empirical relations connecting the $R_L$ and $V_s$ based on both field performance data [5,11,14,15] and laboratory $V_s$ measurement [11–13,16–21] have been emerged and applied in practice. However, the capability of those relations for predicting liquefaction potential of EICP-improved ground must be revealed. It is started by transforming laboratory test data $R_L$ at isotropic condition to $CRR$ value figuring field anisotropic condition by using Equation (4).

$$CRR = \left( \frac{1+2K_0}{3} \right) R_L$$  \hspace{1cm} (4)

The laboratory measured shear-wave velocity in the isotopic condition is estimated as,

$$V_s^2 = \frac{G_0}{\rho_{sat}}$$  \hspace{1cm} (5)
By adjusting \( V_s \) value obtained in the laboratory to the field condition \( V_{sl} \) using Equation (6), the CRR - \( V_{sl} \) relationship can then be plotted to the existing empirical relation.

\[
V_{s1} = V_s \left( \frac{1+2K_0}{3} \right)^{0.25} \left( \frac{100}{\sqrt{\sigma_{v0}}} \right)^{0.25}
\]  

(6)

where: \( q_{sat} \) is the saturated mass density of the soil
\( K_0 \) is the coefficient of the horizontal earth pressure at rest
\( \sigma_{v0} \) is the effective vertical stress in kPa.

This is done for knowing the capability of the existing liquefaction prediction curves for evaluating liquefaction possibility of EICP-improved ground.

Plot data CRR - \( V_{sl} \) relationship of both untreated and EICP-treated sands along with existing prediction charts prepared by researchers [5,11,14,15] is presented in Fig. 7. Untreated sands data are positioned around the curve prepared by Andrus and Stokoe (2000) [5], while EICP-improved sands data fall in the area without liquefaction. They took place considerably separate from any prediction curves. It means that slight amount of the precipitated calcite in the specimen can upgrade significantly the value of \( G_0 \). The data points correlating CRR and \( V_{sl} \) of EICP-treated sands shown in Fig.7 reveal that existing liquefaction prediction charts are not applicable for evaluating liquefaction potential of EICP-improved ground.

![Figure 7. Relationship between CRR and \( V_{sl} \)](image)

The data point of EICP-treated sands and their potential trend line shown in Fig.7 illustrate that the CRR performance of EICP-treated ground is very sensitive to the \( V_{sl} \) measurement precision. Up to \( V_{sl} \) value of around 350 m/s, CRR increase gradually to the area of no liquefaction and rocketing thereafter. Smaller grain size and \( S_o \) as well as higher \( CC \) performed a higher capability of EICP-treated ground against liquefaction. Based on the performance illustrated in Fig.7, the usage of EICP-treated ground approach as a liquefaction mitigation can reduce significantly the occurrence possibility of liquefaction in the target zone.

4. Conclusions

Empirical relations based on field performance and laboratory data for assessing liquefaction potential of natural soil in the ground have been well developed and widely used around the world. However, their capability for predicting liquefaction potential of EICP-treated ground have not been known well. Undrained triaxial shear tests were conducted on EICP-treated samples for creating the CRR-\( V_{sl} \) relationship and compared with the existing prediction curves provided by researchers. Test results show that both liquefaction resistance \( R_t \) and small strain shear modulus \( G_0 \) directly relating to \( V_{sl} \) increase
with CC. Unlike \( R_L \), \( G_0 \) shows good improvement due to increase in \( CP \) and slows down at lower \( CP \). The amount of calcite forming materials can be reduced to attain a specific \( R_L - G_0 \) value by reducing \( S_r \). Besides that, \( R_L - G_0 \) relationship is also influenced by soil type or grain size. The \( R_L - G_0 \) relationship of soils with smaller grain size shows a sharper trend line particularly at low \( CP \).

The plot data of untreated sands performed in this study largely coincided with the existing curve prepared by Andrus and Stokoe (2000). While the others of \( EICP \)-treated sands are pointed at the zone of no liquefaction considerably apart from the existing prediction curves. It reveals that small amount of calcite precipitated can ameliorate significantly the capability of the ground against liquefaction. The other mark is that existing liquefaction prediction curves are not applicable for assessing liquefaction potential of \( EICP \)-treated ground.

References

[1] Seed HB, Idriss IM. Simplified Procedure for Evaluating Soil Liquefaction Potential. J Soil Mech Found Devison, Proceddings Am Soc Civ Eng 1971;97:1249–73.

[2] Youd TL, Idriss IM. Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. J Geotech Geoenvironmental Eng ASCE 2001;127:297–313.

[3] Andrus RD, Stokoe KH, Juang CH. Guide for Shear-Wave-Based Liquefaction Potential Evaluation. Earthq Spectra 2004;20:285–308. doi:10.1193/1.1715106.

[4] Idriss IM, Boulanger RW. Semi Empirical Procedures for Evaluating Liquefaction Potential during Earthquakes. Soil Dyn Earthq Eng 2006;26:115–30. doi:10.1016/j.soildyn.2004.11.023.

[5] Andrus RD, Stokoe KH. Liquefaction Resistance of Soils from Shear Wave Velocity. J Geotech Geoenvironmental Eng ASCE 2000;126:1015–25.

[6] Andrus RD, Stokoe KH. Liquefaction Resistance based on Shear Wave Velocity. Proc. NCEER Work. Eval. & Resist. Soils, 1997, p. 89–128.

[7] Simatupang M, Okamura M. Liquefaction Resistance of Sand Remediated with Carbonate Precipitation at Different Degrees of Saturation during Curing. Soils Found Japanese Geotech Soc 2017;57:619–31. doi:10.1016/j.sandf.2017.04.003.

[8] Simatupang M, Okamura M, Hayashi K, Yasuhara H. Small-strain Shear Modulus and Liquefaction Resistance of Sand with Carbonate Precipitation. Soil Dyn Earthq Eng 2018;115:710–8. doi:10.1016/j.soildyn.2018.09.027.

[9] Kitanihon Sangyo Co. Ltd.: Brochure of Tohoku Keisha, http://www.catvy.ne.jp/~ktsangyo/tohoku.htm n.d.

[10] Nishi K, Komine K, Iijima T. Effects of Confining Pressure and Shear History on the Relation between Liquefaction Strength and Initial Shear Modulus. Proceedings, 23th Annu. Meet. JSSMFE, 1988.

[11] Tokimatsu K, Uchida A. Correlation between Liquefaction Resistance and Shear Wave Velocity. Soils Found Japanese Soc Soil Mech Found Eng 1990;30:33–42.

[12] Tokimatsu K, Yamazaki T, Yoshimi Y. Soil Liquefaction Evaluation by Elastic Shear Moduli. Soils Found Japanese Soc Soil Mech Found Eng 1986;26:25–35.

[13] Tokimatsu K, Kuwayama S, Tamura S. Liquefaction Potential Evaluation based on Rayleigh Wave Investigation and its Comparison with Field Behavior. 2nd Int. Conf. Recent Adv. Geotech. Earthq. Eng. Soil Dyn., 1991, p. 357–64.

[14] Kayen R, Moss RES, Thompson EM, Seed RB, Cetin KO, Der Kiureghian A, et al. Shear-Wave Velocity -Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential. J Geotech Geoenvironmental Eng ASCE 2013;139:407–19. doi:10.1061/(ASCE)GT.1943-5606.0000743.

[15] Robertson PK, Woeller DJ, Finn WDL. Seismic Cone Penetration Test for Evaluating Liquefaction Potential under Cyclic Loading. Can Geotech J 1992;29:686–95.

[16] Wang J-H, Moran K, Baxter CDP. Correlation between Cyclic Resistance Ratios of Intact and
Reconstituted Offshore Saturated Sands and Silts with the Same Shear Wave Velocity. J Geotech Geoenvironmental Eng ASCE 2006;132:1574–80. doi:10.1061/(ASCE)1090-0241(2006)132:12(1574).

[17] Baxter CDP, Bradshaw AS, Green RA, Wang J. Correlation between Cyclic Resistance and Shear-Wave Velocity for Providence Silts. J Geotech Geoenvironmental Eng 2008;134:37–46. doi:10.1061/(ASCE)1090-0241(2008)134:1(37).

[18] Huang Y-T, Huang A-B, Kuo Y-C, Tsai M-D. A Laboratory Study on the Undrained Strength of a Silty Sand from Central Western Taiwan. Soil Dyn Earthq Eng 2004;24:733–43. doi:10.1016/j.soildyn.2004.06.013.

[19] Liu N, Mitchell JK. Influence of Nonplastic Fines on Shear Wave Velocity-Based Assessment of Liquefaction. J Geotech Geoenvironmental Eng 2006;132:1091–7. doi:10.1061/(ASCE)1090-0241(2006)132:8(1091).

[20] Yunmin C, Han K, Ren-peng C. Correlation of Shear Wave Velocity with Liquefaction Resistance based on Laboratory Tests. Soil Dyn Earthq Eng 2005;25:461–9. doi:10.1016/j.soildyn.2005.03.003.

[21] Zhou Y-G, Chen Y-M. Laboratory Investigation on Assessing Liquefaction Resistance of Sandy Soils by Shear Wave Velocity. J Geotech Geoenvironmental Eng ASCE 2007;133:959–72.