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Effects of gravel content on liquefaction resistance and its assessment considering deformation characteristics in gravel-mixed sand

Hirofumi Toyota¹ and Susumu Takada²

Abstract: Many reports describe overestimation of liquefaction resistance based on sounding data related to ground materials containing coarse particles such as gravel and cobbles. Better methods of liquefaction potential estimation must be developed using investigation data other than those from sounding. Gathering perfect and undisturbed samples is difficult, but using seismic methods such as PS logging might be effective for assessing liquefaction potential. For this study, Bender element (BE) tests and local small strain (LSS) tests were conducted, respectively, to measure the dynamic and static shear moduli of gravel-mixed sand specimens. Subsequently, relations between liquefaction strength and secant shear moduli were examined to provide reliable estimation of liquefaction in gravel-mixed sand.

Although the liquefaction resistance increased considerably with over-consolidation, the initial shear modulus exhibited only a slight change with the same over-consolidation. The experimentally obtained results elucidated that the important shear strain level, for which secant

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shear modulus has a strong relation with liquefaction strength, was not a linear elastic region of
0.001%: it was about 0.01%.

Keywords: Bender element test, Cyclic triaxial test, Deformation characteristic, Gravel content,
Liquefaction, Over-consolidation, Sand

Introduction

Measurements of soil properties in the ground must be taken to estimate the safety factor of
liquefaction, as defined by the ratio between the cyclic resistance ratio (CRR) against
liquefaction and the cyclic stress ratio (CSR) applied in the ground. In Japan, CRR is commonly
inferred indirectly using the standard penetration test (SPT). Matsuo (2004) reported that
gravelly ground liquefied during the 1995 Kobe earthquake in Japan. His report described that
the liquefaction resistance inferred for gravelly ground using SPT data had low reliability
because the blow count increased when the boring rod contacted gravel or stone in the ground.
Furthermore, liquefaction of ground containing gravel was reported in the aftermath of the 1948
Fukui earthquake in Japan (Ishihara 1985), the 1995 Kobe earthquake (Hatanaka et al. 1997;
Matsuo 2004), the 2008 Wenchuan earthquake in China (Zhou et al. 2009; Cao et al. 2010), the
2011 Tohoku earthquake in Japan (Toyota et al. 2012), and the 2016 Kaikoura earthquake
(Cubrinovski et al. 2017). Those events underscored the liquefaction phenomenon of gravelly
ground and the need for high-accuracy prediction.

The effects of gravel in the soil matrix were assessed using monotonic loading (consolidated-
drained triaxial) tests (Fragaszy et al. 1992) and cyclic loading (undrained triaxial) tests (Evans
and Zhou 1995). Those studies were conducted to elucidate the shear behavior of gravelly soils
based on the shear behavior of a high-density soil matrix.

Field seismic techniques allow an observer to ascertain the soil stiffness of spacious ground under in-situ stress conditions and in an undisturbed state. Surface wave methods, which have recently become popular and widely used, can reveal the distribution of shear wave velocity in a cross section of the ground at low cost (Nazarian and Stokoe 1984; Addo and Robertson 1992). Therefore, the use of shear wave velocity presents benefits for assessing liquefaction potential.

In an earlier report of a study using the shear wave velocity to assess liquefaction resistance, Tokimatsu and Uchida (1990) introduced a normalizing factor for the shear modulus. Use of that factor can equilibrate soil density effects in different soils. Andrus and Stokoe (2000) used case history data from 26 earthquakes and more than 70 sites to develop their procedures using liquefaction resistance based on the shear wave velocity. Since that time, many researchers (Chen et al. 2005; Baxter et al. 2008; Chen et al. 2008; Dabiri et al. 2011; Ahmadi and Paydar 2014) have revised the model proposed by Andrus and Stokoe (2000) considering factors in soils of diverse types. Kiyota and Wu (2017) demonstrated high correlation between normalized CRR and normalized shear wave velocity using reconstituted and undisturbed samples with low cementation.

Chang et al. (2014) and Chang (2016) reported particular investigations of the applicability of the model to gravelly soil. They demonstrated that although the shear wave velocity increases linearly with gravel content, the gravel content exerts only a minor effect on liquefaction resistance. Despite that progress, knowledge of stress history effects induced by pre-shearing or OCR remained insufficient because of the strong effect of stress history on liquefaction, which cannot be explained by the effects of soil density, as demonstrated by Toyota and Takada (2017). Therefore, the relation between liquefaction resistance and the shear wave velocity in
gravelly soils is still under examination while devoting attention to the stress history. Field investigations have indicated that most liquefied gravelly soils comprise both sand and gravel (Wang 1984; Ishihara 1985; Evans and Zhou 1995). Therefore, sand and gravel were artificially mixed for the experiments described in this report. Moreover, the soil samples tested in this study were created by mixing sand and gravel with a gap-graded distribution to classify the soil matrix from the sample, although natural soils have well-graded distributions in many cases. Using this distinct method, this study elucidates effects of gravel under the same soil matrix. This report presents discussion of the limitations of the use of the shear wave velocity for assessing liquefaction resistance.

Experiments

Apparatus

Triaxial tests were conducted to gather data related to soil stiffness from an extremely small to small strain level (0.001%–1% axial strain). For this objective, a special triaxial apparatus was used to measure the local small strain of the specimen precisely. Figure 1 presents a schematic diagram of the triaxial apparatus for local small strain measurement by proximity transducers (Eddy-current type) with resolution of 0.2 μm and capacity of 2 mm. Preliminary tests confirmed that no slippage occurs between the membrane and specimen when comparing results obtained using a target that was merely glued on the membrane and a target with a spine stuck into the specimen. The proximity transducers and targets are mounted in attachment devices to measure the vertical and horizontal displacements. The transducer for horizontal displacement was located at the center of the specimen height as a representative position.
Although all targets and transducers are installed in the triaxial cell, the positions of the targets for vertical displacement and the transducer for horizontal displacement can be changed from outside of the cell. The attachment devices were glued on the membrane encasing the specimen, which stands itself by application of -30 kPa vacuum, after saturation processing of the specimen. Local axial strain is estimated from the average of vertical displacements. Local radial strain is estimated from the horizontal displacement measured at the attachment device, which holds the radial direction of the specimen. The attachment devices were only mounted when conducting local small strain (LSS) tests. They were unmounted when conducting liquefaction tests. A pair of bender elements (BE) with 4.6 mm length, 12 mm width, and 1 mm thickness is installed in the top cap and the pedestal to measure shear wave velocity $V_s$. 

**Soil and specimen preparation**

Toyoura sand was used as a soil matrix to investigate shear and liquefaction characteristics of gravel sand mixture. Toyoura sand is a Japanese standard sand with mineralogy almost entirely comprising quartz. Its physical properties and the grain size distribution are presented in Fig. 2. Although gravel is defined by the unified soil classification system (ASTM D 2487) as particles retained in a 4.75 mm sieve, particles retained on a 2 mm sieve were defined as gravel for the present study using JIS A 1204. Gravel particles added to the sample are of 2–9.5 mm diameter. Nevertheless, 4.75–9.5 mm particles account for most of the weight (Fig. 2). Gravel content $G_c$ is defined by the mass ratio between gravel and Toyoura sand in the study. The grain size distribution of the sample of $G_c = 30\%$ is also portrayed in the figure. The distribution is gap-graded because the two particle sizes of soils were mixed.

The specimen size was $d = 100$ mm diameter, with $h = 200$ mm height. A membrane is
stretched tautly to the inside wall of a split mold and is set up on a base pedestal. Dry Toyoura sand and gravel mixture are placed carefully using a spoon to create a uniform specimen in the mold. The loosest state of Toyoura sand produced using this method was $D_r = 35\%$. A thin 2-mm-diameter bar was stuck in the part of Toyoura sand many times to disturb sand particle directions and to make dense Toyoura sand ($D_r = 75\%$). Sand of $D_r = 75\%$ is regarded as only slightly liquefied during earthquakes in the real ground. However, experiments were conducted in both extremes of loose liquefiable and dense non-liquefiable state to collect the fundamental wide range of data. The bottom and top 5-mm layer consisted only of Toyoura sand to obtain a good contact between soil and bender elements. When $G_c$ was 40% or more, unevenness sometimes appeared on the specimen surface. Gravel particles are placed inside a 92-mm-diameter brass ring (Fig. 3). After the exterior of the ring was filled with Toyoura sand of the equal density to that of the other sand matrix for creating smooth surface specimens, the ring was carefully removed. Therefore membrane compliance has not been considered in this study (Evans et al. 1992). The necessary weight of Toyoura sand was estimated using a formula reported by Walker and Holtz (1951). The vacuum of -30 kPa was applied to the specimen to maintain its shape until cell pressure was applied to the specimen.

Testing methods

The test procedure is outlined as described below.

1. A specimen set on the triaxial apparatus was saturated using the vacuum saturation procedure (Rad and Clough 1984). Then back-pressure of 200 kPa was applied to produce a B-value greater than 0.97.

2. The specimen was consolidated isotropically under effective confining pressure $\sigma_0'$ of 100
kPa. Subsequently, over-consolidation histories for which OCR = 2 and 4 were applied to the specimen as necessary. The normally consolidated specimen is designated in the study as OCR = 1.

3. The shear wave velocity was measured using a pair of bender elements installed in the top cap and the pedestal.

4. Shearing of two modes was applied to each specimen. One shearing mode is cyclic undrained triaxial tests performed under constant cell pressure. The axial rod was moved cyclically with axial strain rates of 1%/min. The reversal was performed when the axial stress reached a certain half amplitude of cyclic axial stress $\sigma_0$. The test was terminated after double the amplitude of the cyclic axial strain (D.A.) reached 2% because the progress of shear strain after liquefaction is extremely limited in the case of high gravel content, as portrayed in Fig. 4. Moreover, necking was observed in the top of the specimen. Therefore, the cyclic undrained tests were not able to continue up to D.A. of 5% with uniform strain conditions. Another shearing mode is monotonic loading to estimate local small strain precisely. Constant axial strain rates of 0.1%/min were applied to the specimen under constant cell pressure ($\sigma'_0 = 100$ kPa).

Table 1 presents representative results related to liquefaction resistance, shear wave velocity, initial shear modulus, and testing conditions. To obtain the value of CRR, liquefaction tests were conducted at three CSRs, although they are not shown in the table. The relative density $D_r$ is the averaged approximate value after consolidation (just before shearing). A very small volume change (max. $\Delta D_r = 2.5\%$) was observed during isotropic consolidation. Gravel particles of two types were used in the experiment: “pebbles” having round grain shape and “rubble” particles having angular grain shape.
Test results

Representative cyclic loading behavior

Figures 4(a) to 4(c) present representative stress–strain relations and stress paths in Toyoura sand of $D_r = 75\%$ and different gravel content ($G_c = 0–30\%$). Radial strain $\varepsilon_r$ is calculated from volumetric strain $\varepsilon_v$ and axial strain $\varepsilon_a$ using the relation of $\varepsilon_r = (\varepsilon_v - \varepsilon_a)/2$ without an LSS device in undrained cyclic tests. Although the stress paths show no great difference, the number of cycles to reduce $p' = (\sigma_a' + 2\sigma_r')/3$ increases concomitantly with increased gravel contents. In the stress–strain relations, the progress of cyclic shear strain after reaching cyclic mobility diminishes with increased gravel content. These results demonstrate that liquefaction behavior tends to show similar behavior in denser sand matrix as the gravel contents in the specimen increased under the same density (for $D_r=75\%$) of the sand matrix.

Gravel content effects and OCR

Figure 5 displays the relation between the cyclic stress ratio and the number of cycles to liquefaction, which is defined when the double amplitude of axial strain (D.A.) reached 2\%. The liquefaction strength curves in Fig. 5 show almost no change when the gravel content of the specimen is less than 20\%. This result describes the liquefaction strength as based mainly on the sand matrix: Toyoura sand. When the gravel content of the specimen exceeds about 30\%, the liquefaction strength becomes greater than that of Toyoura sand, probably because the contact force between the gravel particles during cyclic loading is formed easily through specimen deformation (e.g. A greater internal friction angle has been provided for gravel than that for sand in the design code.).
The liquefaction resistance (cyclic resistance ratio) of each specimen is defined as the cyclic stress ratio to reach a specified deformation, say 20 cycles, which is the number that is commonly used in Japan. Figures 6(a) and 6(b) respectively portray the relation between the liquefaction resistance and the gravel content, and the relation between the liquefaction resistance and OCR. The liquefaction resistance increases only slightly when the gravel content is less than 20%, but it increases considerably when the gravel content exceeds 30%. This tendency is identical to that described by Chang (2016) and by Chang et al. (2014) for the case of a small amount of gravel, where skeletons of gravel particles are not formed to withstand the applied force. Furthermore, the liquefaction resistance increases concomitantly with OCR, which is extremely sensitive: The liquefaction resistance for the specimen of $G_c = 20\%$ with stress history of OCR = 4 is equal to that of the specimen of $G_c = 50\%$ with OCR = 1.

We used BE tests to estimate the shear wave velocities of the specimens. Figure 7 shows the representative input and received signals, including arrival time measurements, in BE tests. The input signal shown in the figure was a single sinusoidal wave with half-amplitude of 10 V and frequencies of 10 kHz. The travel distance was evaluated as the tip-to-tip distance. The traveling time was assessed using the start-to-start method.

Changes of the shear wave velocity obtained from BE tests to assess the respective effects of the gravel content and OCR are depicted in Figs. 8(a) and 8(b). Results show that the shear wave velocity increases gradually with the gravel content, even with a small amount of gravel, which shows a different trend of liquefaction resistance because the liquefaction resistance is insensitive to a small amount of gravel (Fig. 6(a)). The liquefaction behavior might depend heavily on a weak part of the specimen such as the sand matrix, although the shear wave velocity depends on the density (or void ratio) in the whole specimen. However, the shear wave
velocity shows a slight change with the change of OCR. This tendency differs greatly from the
tendency of liquefaction resistance, where the liquefaction resistance increases with the
increase of OCR, as presented in Fig. 6(b). Verdugo (2016) also presented that although $V_s$ is
insensitive to OCR, the liquefaction resistance increases significantly with OCR. The main
reason for insensitivity of $V_s$ related to OCR is that the void ratio change induced by over-
consolidated loading is extremely small, e.g., only 0.002 in the case of OCR=4 and $G_c=20\%$. A
similar result obtained using Toyoura sand was reported by Toyota and Takada (2017). Toyota
et al. (2018) reported that although the reconstituted Toyoura sand having inherent anisotropy
(particle orientation) exhibited marked anisotropy in shear strength, the difference of $V_s$ was
slight. It would have a reverse correlation with the shear strength in terms of inherent anisotropy.
Therefore, it cannot be said that the shear wave velocity is the most suitable soil property for
assessing liquefaction resistance.

Stress–strain relations of monotonic loading under a cell pressure constant are presented in
Fig. 9. Figure 9(a) presents gravel content effects on stress–strain relations. When the gravel
content becomes greater, deviator stress $q$ also becomes large at the same shear strain $\varepsilon_s$. The
maximum (peak) shear strength depends on the gravel content. However, the values of $q$ tend
to converge at large $\varepsilon_s$ (residual state). Sand friction might dominate the residual strength. The
maximum strength might depend on interlocking of the gravel particles.

Figure 9(b) portrays the OCR effects on stress–strain relations. The behavior by which $q$ is
larger for greater values of OCR at the same shear strain, is observed only for small shear strain
of less than 1%. However, good agreement of stress–strain relations is achieved at shear strain
greater than 2%, irrespective of the value of OCR. Therefore, the shear strength obtained from
monotonic loading, irrespective of the value of OCR, is unsuitable for predicting liquefaction
resistance because the liquefaction resistance increases with OCR (Fig. 6(b)). Stress–strain relations at a low strain level are derived using LSS tests (Fig. 9(c)). Good consistency of the slope of stress–strain relations is achieved at very low strain of less than 0.001%. For this reason, the shear wave velocity changes only slightly with OCR (Fig. 8(b)). In the case of OCR = 4 in Fig. 9(c), the value of $q$ at shear strain larger than 0.001% is greater than those in other OCRs. This fact is expected to engender greater liquefaction resistance without an apparent increase of the shear wave velocity.

Figure 10 depicts the shear strain dependency on secant shear modulus, which is calculated using the relation of $G_{sec}=q / (3\varepsilon_s)$. The secant shear modulus $G_{sec}$ increases concomitantly with increased gravel content (Fig. 10(a)). However, the initial shear modulus $G_0$ is almost independent of OCR. A difference exists by which the initial shear modulus $G_0$ maintains up to larger shear strain as the value of OCR becomes greater (Fig. 10(b)). From this finding, liquefaction resistance is inferred to have closer relations with the value of $G_{sec}$ in shear strain between 0.001% and 0.1% than that of $G_0$ obtained in a linear elastic region.

**Gravel particle size effects on liquefaction**

The gravel used has the grain size distribution presented in Fig. 2. Most of its particles are 4.75–9.5 mm. Here, only small gravel particles of 2–4.75 mm were used for testing by extraction from the same gravel (2–9.5 mm) to examine the effects of gravel particle size. However, the gravel particles were not completely different sizes, such as 2–4.75 mm or 4.75–9.5 mm. Therefore, it is noteworthy that the result obtained from the tests would include effects of particle gradation on liquefaction. Figure 11 presents a summary of the relation between the cyclic stress ratio and the number of cycles to liquefaction. The gravel contents we tested were
20% and 40%. A readily apparent difference of the cyclic stress ratio at 20 cycles does not appear in the case of $G_c = 20\%$. However, in the case of $G_c = 40\%$, CRR of 2–9.5 mm gravel is greater than that of 2–4.75 mm gravel. Therefore, the liquefaction resistance increases slightly even given the same gravel contents when the gravel particles are larger for the same reason a greater internal friction angle is used in larger-grain soils such as gravel.

The increases of the shear wave velocity and the liquefaction resistance with gravel content are presented in Fig. 12, while taking account of the gravel size. The shear wave velocities of 2–9.5 mm gravel are markedly higher than those of 2–4.75 mm gravel. Therefore, the shear wave velocity increases clearly with enlargement of the gravel particle size, given the same gravel contents. However, the effect on liquefaction resistance is not as obvious as that of the shear wave velocity.

Gravel shape effects on liquefaction

Gravel with particles having a sub-rounded shape was used for the experiments described above. This gravel is designated as pebbles in the study. Next, crushed stone was prepared to have an almost identical grain size distribution to that of pebbles. This gravel, which has angular shape, is designated as rubble. Figures 13(a) and 13(b) respectively present photographs of pebbles and rubble. Al-Rousan et al. (2007) summarized widely used image analysis methods for characterization of the aggregate shape. In those methods, the aspect ratio and the roundness were chosen to quantify two shapes (angularities) of gravel used in the study. The aspect ratio is defined as the major axis / minor axis in a 2D image projection. The roundness is expressed as
Roundness = \frac{p^2}{4\pi A} \quad (1)

where \( p \) and \( A \) respectively represent the perimeter and area of the 2D projection of aggregate particles. Table 2 presents image analysis results. Pebbles have a slightly greater aspect ratio than that of rubble. However, rubble particles have greater roundness than that of pebbles; the rubble shape is more angular than that of pebbles.

Figure 14 portrays the relation between the cyclic stress ratio and the number of cycles to liquefaction. Each tested specimen includes gravel of either pebble or rubble type, as shown respectively in Figs. 13(a) and 13(b); its gravel contents are 20% or 40%. There might be a readily apparent tendency by which the liquefaction strength is greater in the case of rubble than that in the case of pebbles because rubble has angular grain shape and because its internal friction angle is larger than that of round grains such as pebbles (Terzaghi and Peck 1948). Moreover, the increase of liquefaction strength becomes greater for gravel contents higher than 40% than at smaller gravel contents of 20%. Effects of the gravel shape on liquefaction are greater than those of the gravel size, as presented in Fig. 11.

Increases of the shear wave velocity and the liquefaction resistance with gravel contents are presented in Fig. 15, devoting particular attention to the gravel shape. The shear wave velocities of the pebbles are almost equal to those of the rubble. The main reason for this rough equality can be inferred as follows. The interlocking effect of gravel particles is slight in an extremely small strain range. However, the gravel shape strongly affects the liquefaction strength, probably because of the greater interlocking effects among gravel particles through the specimens’ deformation during liquefaction tests.
Result of loose Toyoura sand

Figure 16 portrays the relation between the cyclic stress ratio and the number of cycles to liquefaction in Toyoura sand of $D_r = 35\%$ with different gravel contents. Figures 16(a) and 16(b) respectively present results for OCR = 1 and OCR = 4. The maximum density change induced by stress history of OCR=4 in Toyoura sand of $D_r = 35\%$ was associated with a void ratio change of $\Delta e = 0.009$ and relative density change of $\Delta D_r = 2.5\%$. Therefore, the density of the OCR=4 specimen is slightly higher than that of the OCR=1 specimen. The liquefaction strength in OCR = 4 is clearly greater than that in OCR = 1. Actually, the liquefaction strength is increasing gradually with gravel contents in both OCR = 1 and OCR = 4.

Liquefaction resistance is defined for this study as a cyclic stress ratio to reach D.A. = 2\% at 20 cycles. Figures 17(a) and 17(b) present the respective relations between the liquefaction resistance (CRR) and gravel contents in OCR = 1 and OCR = 4. The liquefaction resistance is almost equal up to 20\% of gravel contents in the case of OCR = 1. Then, the liquefaction resistance increases with gravel contents greater than 20\%. This tendency is identical to that found in the case of Toyoura sand of $D_r = 75\%$ (Fig. 6(a)). However, detailed discussion would reveal that the difference in cyclic stress ratio appears at fewer cycles than 20 cycles (Fig. 16(a)), even in the cases of Toyoura sand with $G_c = 10\%$ and $G_c = 20\%$. Moreover, the increase of liquefaction resistance with gravel contents (CRR from 0.117 to 0.129) is less in loose Toyoura sand ($D_r = 35\%$) than that (CRR from 0.220 to 0.350) in dense Toyoura sand ($D_r = 75\%$). Therefore, the gravel content effect on liquefaction is greater in a denser sand matrix.

Liquefaction resistance starts to increase from a small gravel content amount in the case of OCR = 4 (Fig. 17(b)). The void ratio of soil strongly affects the initial shear modulus $G_0$ (Iwasaki et al. 1978).
Relative to that point, Fig. 18(a) presents $G_0$ obtained from BE tests as the vertical axis with a logarithmic scale and shows the void ratio as the horizontal axis with a normal scale. Linearly decreasing relations are obtained in Toyoura sand of $D_t = 75\%$ and $D_t = 35\%$. Therefore, the gravel content effect on $G_0$ can be expressed by the change of the void ratio. However, its effect depends on the relative density of the sand matrix (Toyoura sand) because of the different inclination of the relation between $G_0$ and void ratio. Figure 18(b) depicts $G_0$ obtained from LSS tests and BE tests. Here, $G_0$ of LSS tests was defined at shear strain of 0.001%. For gravel contents that are not greater than 20%, good mutual agreement is achieved between tests. When the gravel contents are higher than 20%, $G_0$ obtained from BE tests is slightly greater than that obtained from LSS tests. A similar result, which was discussed using mean grain size $D_{50}$, was reported by Tanaka et al. (2000).

**Discussion of liquefaction resistance evaluation method**

Figure 19 presents the relation between liquefaction resistance and shear wave velocity obtained from BE tests using all data in the study. The data are scattered even for regulated gravel added to Toyoura sand. The main reasons are the following: (1) Although the gravel content effects on liquefaction resistance are nominal for small amounts of gravel contents (Figs. 6(a), 12, 15, and 17(a)), their effects on shear wave velocity are not negligible (Figs. 8(a), 12, 15, and 18). (2) Although a marked increase of liquefaction resistance appears with OCR (Fig. 6(b)), the shear wave velocity is insensitive to OCR in Toyoura sand (Fig. 8(b)). Therefore, choosing the shear wave velocity as a key property is not the best path to direct prediction of the liquefaction resistance. Liquefaction behavior has a closer relation to deformation characteristics at a certain strain level than at extremely low levels of strain at which soils exhibit linear elastic behavior.
Therefore, the relation between the liquefaction resistance and the secant shear modulus in shear strain of 0.001%–1% was examined to find soil properties that are closely related to liquefaction resistance. Because the cyclic stress ratio of $\sigma_d/(2\sigma'_0)$ is used for liquefaction resistance, the secant shear modulus should also be regarded as removing confining stress effects. The secant shear modulus is well known (Iwasaki et al. 1978) to be roughly proportional to the square root of confining stress in a small strain range (about smaller than 0.001%). Therefore, the secant shear modulus was corrected by the square root of confining effective stress in this study, as shown in equation (2).

$$G_{sec1} = G_{sec}\left(\frac{P_a}{\sigma'_0}\right)^0.5$$

In that equation, $G_{sec1}$ is the confining stress-corrected secant shear modulus; $P_a$ is the reference stress of 100 kPa. All $G_{sec1}$ agreed with $G_{sec}$ because the experiments were conducted under $\sigma'_0'=100$ kPa.

Figure 20 presents the relation between the liquefaction resistance and the secant shear modulus corrected by the square root of confining stress at shear strain of 0.001% (Fig. 19(a)) and 0.01% (Fig. 20(b)) under a dense sand matrix (Toyoura sand of $D_r = 75\%$). Plots of OCR = 2 and OCR = 4 are definitely outside of the trend obtained from plots of OCR = 1 in the secant shear modulus at shear strain of 0.001%, $G_{0.001\%}$ (Fig. 20(a)). Moreover, the relation between the liquefaction resistance and the corrected $G_{0.001\%}$ is bilinear with an increase of gravel contents. However, a certain trend, which is linear, not bilinear, appears in all plots including those of the results of OCR = 1, 2, and 4 in the use of the secant shear modulus at shear strain of 0.01%, $G_{0.01\%}$ (Fig. 20(b)). Results show that the use of $G_{0.01\%}$ is more appropriate than that
of $G_{0.001\%}$ to predict liquefaction resistance that includes the effects of OCR and gravel content. Liquefaction behavior includes large deformation of the specimen. Therefore, $G_{0.01\%}$ is regarded as a stronger indicator of liquefaction resistance than $G_{0.001\%}$, which is a property in the linear elastic region.

Next, the relations between the liquefaction resistance and the $G_{sec1}$ were examined in all experimental data where $G_{0.001\%}$ and $G_{0.01\%}$ were chosen as $G_{sec}$ for comparison. The results are presented respectively in Figs. 21(a) and 21(b). Although data using $G_{0.001\%}$ are widely scattered, a tendency to converge to a certain trend (line in the figure) appears in data using $G_{0.01\%}$, which indicates that the liquefaction resistance has a closer relation with the soil property of $G_{0.01\%}$ than with $G_{0.001\%}$. The following relation was obtained between liquefaction resistance (CRR) and $G_{0.01\%}$.

$$CRR = 0.045G_{0.01\%} - 0.079 \quad (3)$$

The coefficient of determination $R^2$ between liquefaction resistance and $G_{sec}$ is presented in Fig. 22 with respect to the shear strain to estimate $G_{sec}$. The peak, where $R^2$ becomes closest to 1, appears at around shear strain of 0.01%. This fact indicates that liquefaction resistance has the closest relation with $G_{0.01\%}$ in all ranges of shear strain tested in this study.

The initial shear modulus $G_0$ is readily obtainable from data of seismic methods such as the PS logging or the surface wave survey (Addo and Robertson 1992; Nazarian and Stokoe 1984). However, an important difficulty is obtaining data of $G_{0.01\%}$ in real ground, which is more suitable for liquefaction assessment, as explained above. One might obtain undisturbed samples and conduct experiments to ascertain their deformation properties. However, extracting
undisturbed samples including gravel from the ground is difficult and expensive. Therefore, the reduction rate of the shear modulus with shear strain was examined to infer $G_{0.01\%}$ from $G_{0.001\%}$.

Figure 23 portrays the shear strain dependence of secant shear modulus normalized by the secant shear modulus at shear strain of 0.001% in gravel-mixed sand. The normalized secant shear modulus scatters between 0.5 and 0.8 at 0.01% shear strain in the OCR = 1 condition (Fig. 23(a)), and between 0.7 and 0.9 at 0.01% shear strain in the OCR = 4 condition. A readily apparent tendency exists by which the normalized $G_{sec}$ in the OCR = 4 has a larger value than that in OCR = 1 for shear strain of 0.01%.

Then, reduction rate $G_{0.01\%/0.001\%}$ is calculated. $G_{0.001\%}$ closely approximates $G_0$ obtained from BE test as presented in Fig. 18(b). The relation between the reduction rate and OCR is presented in Fig. 24. Results show that $G_{0.01\%}$ can be estimated from the following equation.

$$G_{0.01\%} = (0.06\text{OCR} + 0.55)G_{0.001\%}$$  \hspace{1cm} (4)

From this relation, the average reduction of the initial shear modulus can be estimated for different values of OCR. Estimating the stress history, such as OCR, in real ground is difficult, but if the ground is improved artificially such as through preloading, we can estimate the OCR of the ground. In such a case, this relation is useful to predict the liquefaction resistance induced by ground improvement.

Although the data scatter even in the same OCR, the reduction rate includes all cases considering the gravel content effects and the effects of the sand matrix density. The results presented above were obtained from ideal specimens prepared using gravel and sand with uniform particle size. Fine contents or ageing effects in the real ground are ignored in this study.
However, the results exhibit fundamental soil characteristics related to liquefaction resistance and deformation modulus. Those important factors must be considered for the precise prediction of liquefaction. A method must be established in future research for obtaining the shear modulus at shear strain of about 0.01% by collecting data related to widely various gravelly soils. Future work must also be undertaken to ascertain the relation between $G_{0.01\%}$ and in-situ properties such as STP-$N$ or CPT-$q_c$.

**Conclusions**

The reliability of liquefaction potential estimates obtained using sounding data declines along with the contents of gravel in sandy ground. It is urgently necessary to increase the accuracy of estimation of the liquefaction potential in real ground that contains gravel because liquefaction damage has also been reported in gravelly ground. For precise investigation of gravel effects on liquefaction, ideal specimens with no aging effect or sampling disturbance were produced artificially in a laboratory using sand and gravel. Then factors that strongly related to the occurrence of liquefaction were measured: liquefaction resistance, shear wave velocity, and the secant shear modulus. Finally, the relations among those factors were examined.

The main results of the study are summarized as presented below.

1. The liquefaction resistance is insensitive for gravel contents of less than 20%. Under such a condition, the liquefaction properties of the sand matrix (Toyoura sand in this study) dominate the liquefaction resistance. The liquefaction resistance increases concomitantly with gravel contents greater than 20%. However, the initial shear modulus and the shear wave velocity constantly increase with gravel contents, even for small amounts of gravel contents.
2. The liquefaction resistance increases considerably with the increase of OCR. However, the initial shear modulus and the shear wave velocity are difficult to change with OCR. Local small strain tests revealed that the stress history such as OCR expanded the strain range exhibiting a linear elastic behavior without an increase in the initial shear modulus.

3. When the gravel contents become high, both the gravel grain size and shape affect the liquefaction strength. Greater liquefaction resistance and shear wave velocity are obtained when the gravel particles are larger. Regarding the gravel shape, greater liquefaction resistance is obtained from rubble than from pebbles, but the gravel shape is insensitive to the shear wave velocity.

4. Liquefaction resistance of sand including gravel has a close relation with the secant shear modulus at shear strain of about 0.01%.

5. The average reduction rate of $G_{sec}$ at 0.01% shear strain, $G_{0.01%}/G_0$, was found to be about 0.6 in the OCR = 1 specimen and 0.8 in the OCR = 4 specimen.

Acknowledgments

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**Table captions**

TABLE 1—*Summary of results*

TABLE 2—*Image analysis of gravel particles*

**Figure captions**

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Fig. 22. Effect of shear strain on calculation of $G_{sec}$ on the coefficient of determination.

Fig. 23. Reduction of normalized shear moduli with shear strain: (a) OCR = 1 and (b) OCR = 4.

Fig. 24. Relation between reduction of $G_{sec}$ at $\varepsilon_s = 0.01\%$ and OCR.
### TABLE 1—Summary of results

<table>
<thead>
<tr>
<th>$G_\varepsilon$ (%)</th>
<th>$D_\varepsilon$ (%) of sand</th>
<th>OCR</th>
<th>$CRR$ at 20 cycles</th>
<th>$V_s$ (m/s) from BE</th>
<th>$G_0$ (MPa) from LSS</th>
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### TABLE 2—Image analysis of gravel particles

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Fig. 1. Local small strain measurement in a triaxial cell.

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(a) Toyoura sand + $G_c=0\%$
(b) Toyoura sand + $G_c=20\%$
(c) Toyoura sand + $G_c=30\%$

Fig. 4. Representative undrained cyclic behavior: (a) $G_c = 0\%$, (b) $G_c = 20\%$, and (c) $G_c = 30\%$. 
Toyoura sand ($D_r = 75\%$) + Gravel
Dry sticking method, D.A. = 2\%
Cyclic undrained triaxial test ($\sigma'_0 = 100$ kPa)

Cyclic stress ratio, $\sigma_d / (2 \sigma'_0)$

Toyoura sand

Number of cycles to liquefaction, $N$

**Fig. 5.** Liquefaction strength curves ($D_t = 75\%$, pebbles).

Liquefaction resistance, $\sigma_d / (2 \sigma'_0)$

(a) Toyoura sand ($D_t = 75\%$) + Gravel
Liquefaction resistance: D.A. = 2\%, 20 cycles $\sigma'_0 = 100$ kPa

Gravel content, $G_c$ (%)

(b) Toyoura sand ($D_t = 75\%$) + Gravel 20\%
Liquefaction resistance: D.A. = 2\%, 20 cycles $\sigma'_0 = 100$ kPa

Over-consolidation ratio (OCR)

**Fig. 6.** Change of liquefaction resistance: (a) gravel content and (b) over-consolidation ratio.
Fig. 7. Representative input and received waves in BE tests: (a) $G_c=20\%$, (b) $G_c=30\%$, and (c) $G_c=40\%$. 

$G_c$=40%.
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Cyclic undrained triaxial test (σ<sub>0</sub>' = 100 kPa)

Shear wave velocity, \( V_s \) (m/s)
Gravel content, \( G_c \) (%)

Liquefaction resistance, \( \sigma_d / (2\sigma_0') \)

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Liquefaction resistance: D.A. = 2\%, 20 cycles

$\sigma_0' = 100$ kPa, $G_{sec}$ at $\varepsilon_s = 0.001\%$
Fig. 21. Relations between liquefaction resistance and normalized secant shear moduli corrected by the square root of confining stress (all data): (a) $\varepsilon_s = 0.001\%$ and (b) $\varepsilon_s = 0.01\%$.

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