Soil Dynamics and Earthquake Engineering 115 (2018) 291–304

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

Monotonic and cyclic simple shear response of gravel-sand mixtures

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ARTICLE INFO

Keywords:
Cyclic Simple Shear
Gravelly Soils
Shear Wave Velocity
Liquefaction
Gravel Sand Mixes

ABSTRACT

Understanding the factors that affect the monotonic and cyclic response of gravelly soils during earthquake events is critical to infrastructure design. In this study a large-size Cyclic Simple Shear (CSS) device was utilized to perform monotonic and cyclic shear tests on mixtures of either subrounded 9 mm Pea Gravel or angular 8 mm Crushed Limestone (CLS8) with subrounded Ottawa C109 sand. Tests were performed in constant volume conditions and shear wave velocity was measured for each specimen. Monotonic and cyclic test results at D_r = 47% show that there is an optimum mixture percentage that results in the greatest shear strength and resistance to liquefaction (40% Sand for Pea Gravel Mixtures and 60% Sand for CLS8 Mixtures). The effects of particle angularity, cyclic stress ratio, and initial vertical stress on monotonic and cyclic response of loose and dense gravel mixtures were investigated and are presented. Comparison of the results from the cyclic simple shear tests with existing liquefaction triggering charts suggests the need for improved charts for gravelly soil liquefaction evaluation.

1. Introduction

Understanding the response of gravelly soils during seismic events is critical to robust performance-based design. Historical (1964 Alaska, USA; 1975 Haicheng, China; 1976 Tangsham, China; 1983, Borah Peak Idaho, USA; 1994 Armenia; 1995 Kobe, Japan) as well as recent earthquakes (2008 Wenchaun, China; 2014 Cephalonia, Greece; 2016 Kaikoura, New Zealand) have demonstrated that gravelly soils are susceptible to liquefaction [1,13–15,24,30]. However, the response of gravelly soils both during and following seismic events is not well understood as there are few well-documented case histories and limited laboratory test data. To properly design and evaluate gravelly soil sites for liquefaction susceptibility, a study of the factors that affect gravelly soil shear response under a variety of conditions is needed.

Laboratory testing of soils allows for the investigation of parameters that affect shear response under controlled conditions and parametric evaluations can be performed for loading scenarios where field case-history data is sparse. Several studies have evaluated the monotonic and cyclic shear response of gravelly soils. Holtz and Gibbs [17] performed consolidated drained triaxial tests on mixtures of sand and gravel with different percent gravel contents and found that the shear strength of gravelly soils increased with increasing the gravel content up to 50–60%. The authors also found that increasing particle angularity increased the shear strength of the gravelly soil. Rashidian [25] performed a study of sand and gravel mixtures prepared very loose (relative density less than approximately 20%) and showed that during undrained monotonic loading specimens with up to 90% gravel content displayed a contractive behavior. Chang and Phantachang [7] performed constant load monotonic simple shear tests on angular crushed aggregates mixed with either poorly-graded or well-graded sand in different percentages. The authors concluded that gravelly soils can be categorized as either sand-like, gravel-like, or in-transition based on gravel content. In both the well-graded and poorly-graded mixtures, increasing gravel content reduced shear resistance. Initial vertical stress was shown not to have an effect on the normalized shear stress ratio (τ/σ_v'), which ranged from 0.40 to 0.60 for the drained simple shear tests.

The cyclic response of gravelly soils has also been investigated in the laboratory. Wong et al. [29] studied the liquefaction response of gravelly soils using large-scale triaxial tests and concluded that uniform gravels exhibit slightly higher resistance to liquefaction than well-graded gravelly soils, but that membrane compliance affected the measured response. Banerjee et al. [3] performed cyclic triaxial tests on dense gravelly soils from Oroville dam and found that the dense gravel exhibited many similarities to dense sand under cyclic loading. Specimen preparation technique was found to have little effect on shear response. Evans and Seed [9] tested Watsonville gravel in triaxial devices, and found that the cyclic stress ratio (CSR) for liquefaction in 10

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https://doi.org/10.1016/j.soildyn.2018.07.016
Received 5 October 2017; Received in revised form 11 June 2018; Accepted 14 July 2018

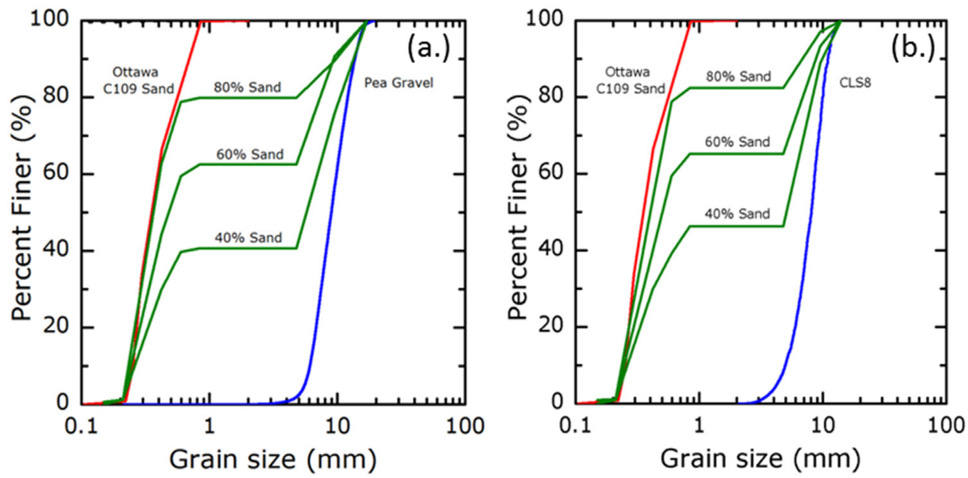

Fig. 1. Grain Size Distributions for (a) Pea Gravel mixtures and (b) CLS8 mixtures.

cycles was only 0.143. Evans et al. [10] showed that membrane compliance in the triaxial apparatus can overestimate liquefaction resistance by as much as 40%. Hatanaka et al. [16] tested Masado fill that liquefied during the 1995 Kobe earthquake and found that despite its high dry density and gravel content, the gravel fill liquefied at CSRs from 0.15 to 0.23 which is similar to Toyoura sand tested at a relative density of 70%. Evans and Zhou [11] performed undrained triaxial tests of gravel-sand mixtures with gravel contents ranging from 0% to 60% and found that the inclusion of gravel particles increased the liquefaction resistance. Kokusho et al. [22] evaluated the undrained cyclic and post-cyclic shear strength of granular soils with different particle gradations utilizing a triaxial test apparatus. The authors found that the liquefaction strength of well-graded granular soils is similar to poorly-graded sands with identical relative densities. Chang et al. [6] performed cyclic simple shear tests of gravel-sand mixtures with a D_{50} value for the gravels of 5.3 mm. The authors found that the transition from sand-like to gravel-like response was in the 50–70% gravel content range. The authors measured shear wave velocity (V_S) of each specimen and found that adding gravel to sand-like specimens increased V_S. A comparison of cyclic data with Andrus and Stokoe [2] showed that the Andrus and Stokoe [2] curve, which is based on field-liquefaction case history data, should be shifted to lower values of V_{S1}. In summary, while laboratory testing of gravelly soils has been undertaken, most testing has been performed using triaxial devices (which are prone to membrane compliance issues) and the influence of several parameters (particle angularity, density, vertical stress) still remains to be investigated. Study of these parameters aids in the understanding of gravelly soil response that has been observed in the laboratory (including uniform gravel tests in [18]) and in the field (with limited data) during earthquake events.

A large-size Cyclic Simple Shear (CSS) device was utilized to perform monotonic and cyclic shear tests on mixtures of either subrounded 9 mm Pea Gravel or angular 8 mm Crushed Limestone (CLS8) with subrounded Ottawa C109 sand. Tests were performed at constant volume conditions and shear wave velocity of each specimen was measured for comparison between test data and existing relationships for liquefaction evaluation [2,21,5]. This paper presents some of the first cyclic simple shear data for gravel-sand mixtures, and provides an evaluation of parameters that affect the shear response of gravelly soils under monotonic and cyclic loading conditions.

2. Test materials and methods

2.1. Test equipment

A large-size Cyclic Simple Shear (CSS) device developed at the University of Michigan in collaboration with a laboratory equipment manufacturer was utilized to evaluate the monotonic and cyclic response of gravelly soils. The CSS specimen is 307.5 mm in diameter and the specimen height can range from approximately 100 mm to 120 mm. The development and validation of the CSS device is presented in detail in Zekkos et al. [31]. In addition to monotonic and cyclic, stress or strain controlled, constant load or constant volume simple shear testing, V_S measurements using bender elements and miniature accelerometers can be conducted. In this research, accelerometers were utilized for shear wave velocity measurements since they were found to yield identical V_S values with bender elements, but the latter were getting damaged often by the gravelly soils. Details of the accelerometer setup and measurement is given in Hubler [20] and Zekkos et al. [31].

2.2. Test materials

The materials tested in this study included a uniform sand (Ottawa C109 sand) and two uniform gravels (9 mm Pea Gravel and 8 mm Crushed Limestone (CLS8)). These uniformly-graded materials were extensively tested (and the results are presented in [18]) before studying gravel-sand mixtures of Pea Gravel with Ottawa C109 sand and CLS8 with Ottawa C109 sand. Gravel-sand mixtures were prepared at mixture percentages (by weight) of 80% Sand/20% Gravel, 60% Sand/40% Gravel, and 40% Sand/60% Gravel. These mixtures will be referenced by their sand percentage throughout this paper. Two different types of gravels of similar size were used for testing so that effects of particle angularity could be assessed. The grain size distributions of the Pea Gravel mixtures are given in Fig. 1a, while the grain size distributions of the CLS8 mixtures are given in Fig. 1b. The 80% Sand, 60% Sand, and 40% Sand specimens have gap-graded distributions for the Pea Gravel mixtures and CLS8 mixtures. The relevant properties of the Pea Gravel and Ottawa C109 sand mixtures are given in Table 1, and the properties of the CLS8 and Ottawa C109 sand mixtures are given in Table 2. The evaluation of the maximum density of gap-graded

Table 1
Properties of Pea Gravel/Ottawa C109 sand mixtures.

Properties	Pea Gravel	60% Gravel/ 40% Sand	40% Gravel/ 60% Sand	20% Gravel/ 80% Sand	Ottawa C109 Sand
G_S	2.74	2.70	2.69	2.67	2.65
$\gamma_{d,max}$ (kg/m^3)	1741	2114	1978	1848	1733
$\gamma_{d,min}$ (kg/m^3)	1546	1960	1818	1665	1512
e_{max}	0.772	0.379	0.477	0.602	0.752
e_{min}	0.574	0.279	0.358	0.443	0.529

Table 2
Properties of CLS8/Ottawa C109 sand mixtures.

Properties	8 mm Crushed Limestone	60% Gravel/ 40% Sand	40% Gravel/ 60% Sand	20% Gravel/ 80% Sand	Ottawa C109 Sand
G_S	2.65	2.65	2.65	2.65	2.65
$\gamma_{d,max}$ (kg/m^3)	1751	2223	2032	1870	1733
$\gamma_{d,min}$ (kg/m^3)	1357	2068	1842	1660	1512
e_{max}	0.953	0.419	0.455	0.586	0.752
e_{min}	0.513	0.313	0.335	0.413	0.529

gravelly soil mixtures can be challenging, and therefore the method described in Hubler et al. [19] was used to assess the maximum density. This method resulted in experimental values of maximum density that were similar to those predicted by the alpha method [12]. Minimum density was evaluated by placing gravel-sand mixtures as loose as possible with a funnel at zero drop height. Fig. 2 shows the minimum and maximum void ratios as a function of percent sand for Pea Gravel and CLS8 mixtures. As shown in the figure, the range of possible void ratios becomes smaller for the mixtures and is smallest in the 40–60% range compared to the uniform materials.

2.3. Test procedure

The large-size CSS device was utilized to perform a series of 92 monotonic and cyclic tests on Ottawa C109 sand, Pea Gravel, CLS8, and mixtures of Ottawa C109 sand with either Pea Gravel or CLS8. Specimens were prepared at two target relative densities (D_r) for each material: $D_r = 47\% +/-3\%$ and $D_r = 87\% +/-5\%$. These D_r values are based on the global void ratio of the specimen, including both gravel and sand skeletons. This relative density can be considered the global relative density, as it accounts for the entire mixture of sand and gravel in the specimen. Evans and Zhou [11] referred to this overall relative density as the composite relative density of the specimen. This relative density can also be converted to a global void ratio value when the specific gravity of the mixing materials is known. The void ratios of the sand skeleton and gravel skeleton can also be defined using the intergrain state framework [6]. A similar framework has been used for sand and silt mixtures [27], where the intergranular and interfine void ratio were defined for sand and silt mixtures. Although it is recognized that the stress-strain response of sand-silt mixtures and gravel mixtures is complex, the alternative skeleton void ratios offer an interesting approach in interpreting material response and therefore were evaluated in this study. In the interpretation of gravelly soils, the sand skeleton void ratio is found to be critical when the gravel is not contributing to the force chain and the gravel skeleton void ratio is critical when the sand is filling the void space between gravel particles and not contributing to the force chain. The sand skeleton void ratio can be defined as:

$$e_{sk} = \frac{e}{1-GC} \tag{1}$$

where e is the global void ratio and GC is the gravel content in percent. The gravel skeleton void ratio can be defined as:

$$e_{gk} = \frac{e+(1-GC)}{GC} \tag{2}$$

Specimens were prepared at the loose dry state by placing the gravel-sand mixtures with a funnel in lifts of approximately 25 mm. The sand and gravel was first mixed together before placement with the funnel to ensure uniform particle distribution and minimize particle segregation. Specimens were prepared at the dense dry state by dropping a 5-kg weight with a circular diameter of 150 mm from a height of 50–75 mm. An average of 25 drops per layer in 3 layers (approximately 35–40 mm layer height) was used. This small drop height was used to minimize particle damage during specimen preparation and a greater number of drops was used for successive layers to ensure specimen uniformity.

Specimens of 100% Sand, 80% Sand, 60% Sand, 40% Sand, and 0% Sand (or 100% Gravel) were tested at $D_r = 47\%$ and $D_r = 87\%$ at initial vertical stresses (σ'_{v0}) of 100, 200, and 400 kPa. Specimens were first subjected to the desired vertical stress σ'_{v0}. V_S was measured following vertical load application and before shearing (either monotonic or cyclic) using accelerometers. Monotonic tests were strain-controlled and sheared at a rate of 0.3% per minute, which enabled precise control of constant volume conditions and was similar to other strain rates used in simple shear testing of cohesionless soils [26]. Cyclic tests were stress-controlled with different cyclic stress ratios (CSRs) ranging from 0.04 to 0.14 and a loading frequency of 0.33 Hz. Monotonic and cyclic tests were performed at constant volume conditions, which have been shown to accurately represent truly undrained conditions in simple shear testing [8]. In constant volume simple shear testing, the measured change in the vertical stress is assumed to be equal to the pore pressures that would develop in a truly undrained test. Constant volume conditions are maintained in the CSS device by active control through a feedback loop, which suppresses movement of the vertical cap and allows for accurate measurement of the change in vertical stress.

3. Test results

3.1. Shear wave velocity

Shear wave velocity was measured following vertical load

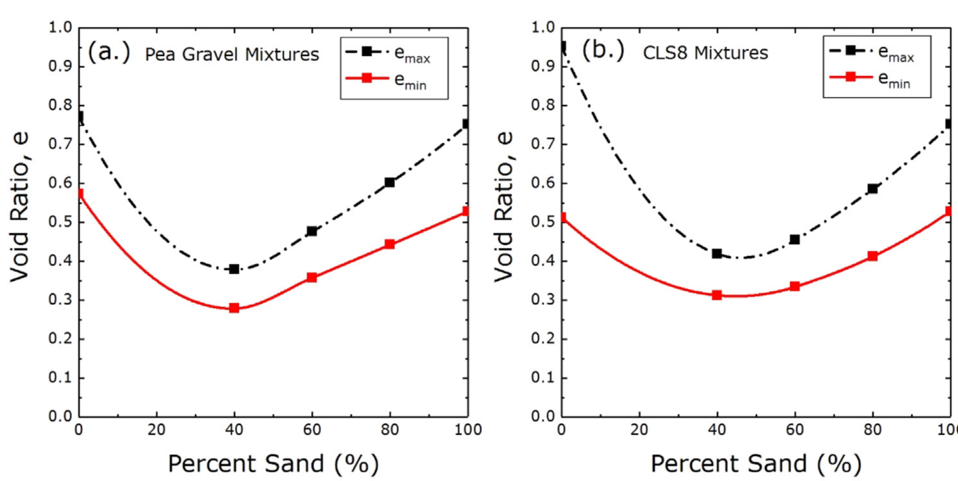

Fig. 2. Minimum and maximum void ratio for (a) Pea gravel mixture and (b) CLS8 mixtures.

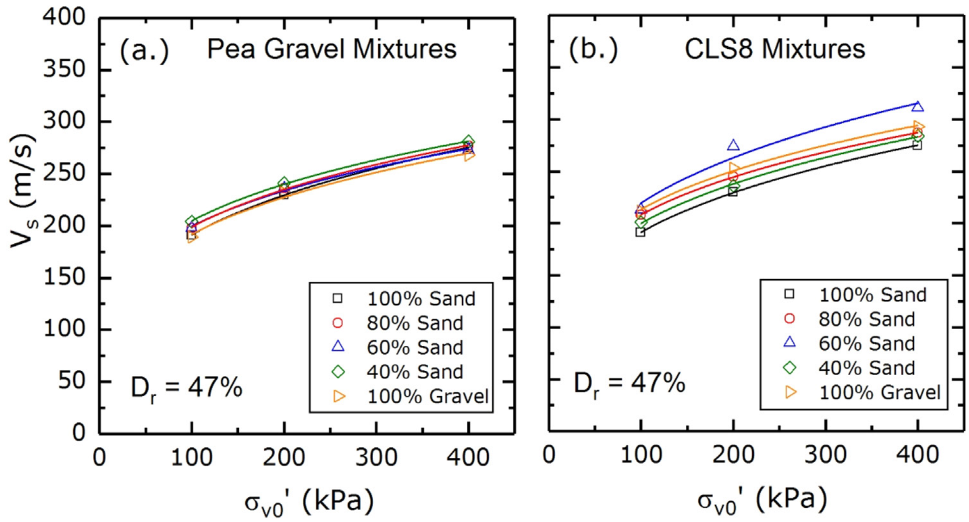

Fig. 3. Shear wave velocity measurements for (a) Pea Gravel mixtures and (b) CLS8 mixtures.

Table 3
Shear wave velocity parameters for Pea Gravel/Ottawa C109 sand mixtures at D_r = 47%.

Parameter	Ottawa C109 sand	80% sand	60% sand	40% sand	Pea gravel
α (m/s)	189	196	199	204	188
β	0.26	0.23	0.23	0.23	0.25

Table 4
Shear wave velocity parameters for CLS8/Ottawa C109 sand mixtures at D_r = 47%.

Parameter	Ottawa C109 sand	80% sand	60% sand	40% sand	8 mm crushed limestone
α (m/s)	189	206	215	201	210
β	0.26	0.23	0.27	0.25	0.23

application and compression of the gravel-sand mixture specimens and results are presented for Pea Gravel mixtures at D_r = 47% in Fig. 3a and CLS8 mixtures at D_r = 47% in Fig. 3b. For both Pea Gravel and CLS8 mixtures, V_S increased with increasing σ'_{v0} from 100 to 400 kPa. Fig. 3a shows that the mixing of Pea Gravel with Ottawa C109 sand does not significantly affect the V_S. This is due to the similarity in V_S of the uniform Pea Gravel and Ottawa C109 sand materials at σ'_{v0} = 100 kPa (Pea Gravel V_S = 189 m/s, Ottawa C109 sand V_S = 191 m/s). The mixture with the highest V_S values was the 40% Sand specimen. Fig. 3b shows that the mixing of CLS8 with Ottawa C109 sand results in changes in V_S compared to the uniform CLS8 specimens. This is due to the differences in V_S at σ'_{v0} = 100 kPa for the uniform crushed limestone CLS8 gravel (V_S = 212 m/s) and Ottawa C109 sand (Vs = 191 m/s). The mixture with the highest V_S was the 60% Sand specimen followed by the 100% Gravel specimen. Power functions of the form in Eq. (3) were fit to the data and values determined for the α and β parameters are presented in Table 3 for Pea Gravel mixtures and Table 4 for CLS8 mixtures. The V_S relationship with vertical stress can be described by:

$$V_S = \alpha\left(\frac{\sigma'_v}{1\ atm}\right)^{\beta} \tag{3}$$

where α (expressing V_S in m/s at 1 atm (101.3 kPa)) and β are fitting parameters determined from laboratory testing. The α value for the Pea Gravel mixtures was the highest for the 40% Sand specimen, while the highest α value for the CLS8 mixtures was for the 60% Sand specimen. The β values for the Pea Gravel mixtures ranged from 0.23 to 0.26, with lower values of 0.23 for the 80% Sand, 60% Sand, and 40% Sand specimens. The β values for the CLS8 mixtures ranged from 0.23 to 0.27, with the 60% Sand specimen having the highest β value of 0.27. The β values evaluated in this study are reasonable for the soils tested [23]. V_{S1}, which will be used for analysis and comparison of data in this study, was calculated using the following equation:

$$V_{S1} = V_S C_V = V_S\left(\frac{P_a}{\sigma_v'}\right)^{0.25} \tag{4}$$

where P_a is atmospheric pressure (101.3 kPa) and σ'_v is the vertical effective stress.

3.2. Monotonic constant volume simple shear

Monotonic shear tests were performed on mixtures of Pea Gravel with Ottawa C109 sand and CLS8 with Ottawa C109 sand. Specific parameters, including mixture percentage, initial vertical stress, and relative density, were targeted to provide insight into their effect on shear response. Data was interpreted by evaluating the peak, phase transformation (PT), and ultimate state (US) shear strengths. Fig. 4 shows data for 100% Sand, 40% Sand, and 100% Pea Gravel for shear stress-shear strain response and corresponding stress paths. The peak, PT, and US points are identified in Fig. 4b for the 100% Sand specimen. The peak point corresponds to the peak shear strength, which is easily identified for the 100% Sand specimen. The PT point is the point of maximum pore pressure generation (i.e. minimum vertical effective stress), while the US point corresponds to the line of maximum obliquity that is attained as the specimen is sheared to greater shear strain.

Pea Gravel and Ottawa C109 sand mixtures were tested at mixture percentages of 100% Sand, 80% Sand, 60% Sand, 40% Sand, and 100% Gravel. Fig. 5 presents monotonic shear test results for specimens at D_r = 47% and σ'_{v0} = 100 kPa. Fig. 6 presents the monotonic results for specimens at D_r = 87% and σ'_{v0} = 100 kPa. In these figures the following plots are presented: (a) stress-strain response, (b) stress path response, (c) τ/σ'_{v0} versus shear strain, and (d) τ/σ'_v versus shear strain. Of particular interest is the change in stress-strain response of the gravel-sand mixtures compared to the uniform gravel or sand. For the D_r = 47% specimens, the uniform sand and gravel exhibit a peak on the stress-strain curve (Fig. 5a), followed by strain softening to some minimum shear resistance value and then exhibit a strain hardening response. This trend becomes less pronounced for the gravel-sand mixtures and at 40% Sand (the optimum mixture) the specimen does not exhibit any softening, but only strain hardening response. For the D_r = 87% specimens, strain softening is not observed. The uniform materials have

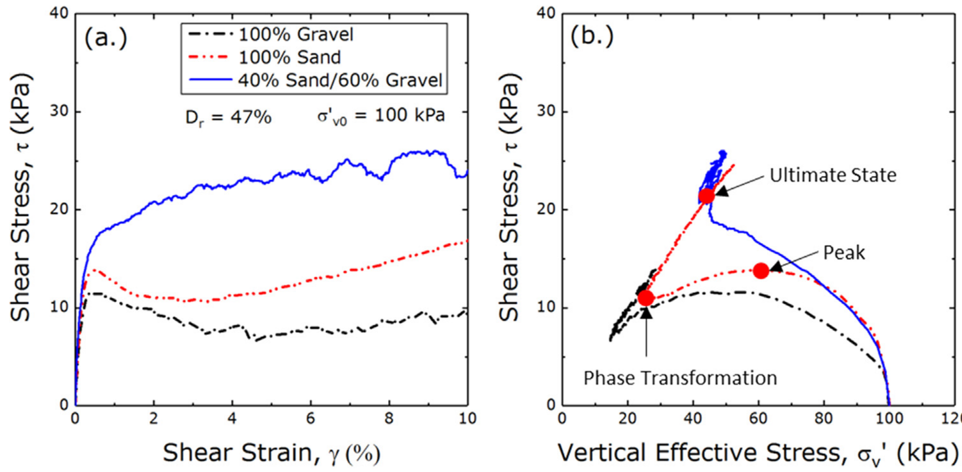

Fig. 4. Comparison of Pea Gravel, Sand, and Pea Gravel-Sand mixture monotonic (a) Shear stress-shear strain response and (b) Shear stress-vertical effective stress response.

lower peak shear strengths, but post-peak strain hardening is greater than in the gravel-sand mixtures. The effect of mixture percentage is more pronounced for the $D_r = 47\%$ specimens than the $D_r = 87\%$ specimens. For example, $D_r = 47\%$ specimens at initial vertical stress of 100 kPa show a significant increase in shear strength for a mixture percentage of 40% Sand, as shown in Fig. 5a. Conversely, for the $D_r = 87\%$ specimens, the shear strength in the peak range (approximately 1% shear strain) is more tightly grouped with the 60% Sand and 40% Sand displaying the greatest peak strength. As shear strain increases to 10% the shear resistance variation increases, with the 80% Sand specimen displaying the greatest shear strength. For the $D_r = 47\%$ specimens, the 40% Sand mixture is considered the optimum mixture since it has the greatest shear resistance. Fig. 5b shows the effect of mixture percentage on stress path response. As the mixture percentage increases to the optimum, the response changes to fully strain hardening, with no post-peak softening.

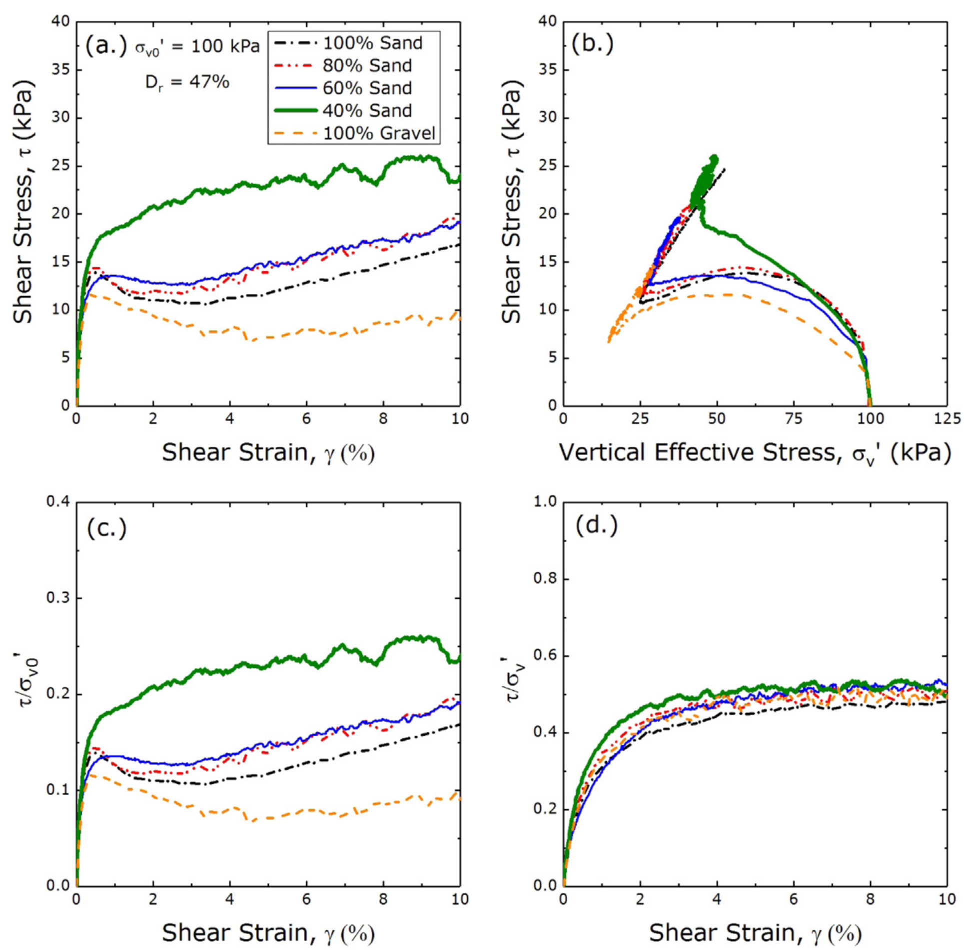

Fig. 5. Monotonic simple shear results for Pea Gravel mixtures at $D_r = 47\%$ and $\sigma'_{v0} = 100$ kPa for (a) Shear stress- shear strain response, (b) Stress path response, (c) τ/σ'_{v0} versus shear strain, and (d) τ/σ'_v versus shear strain.

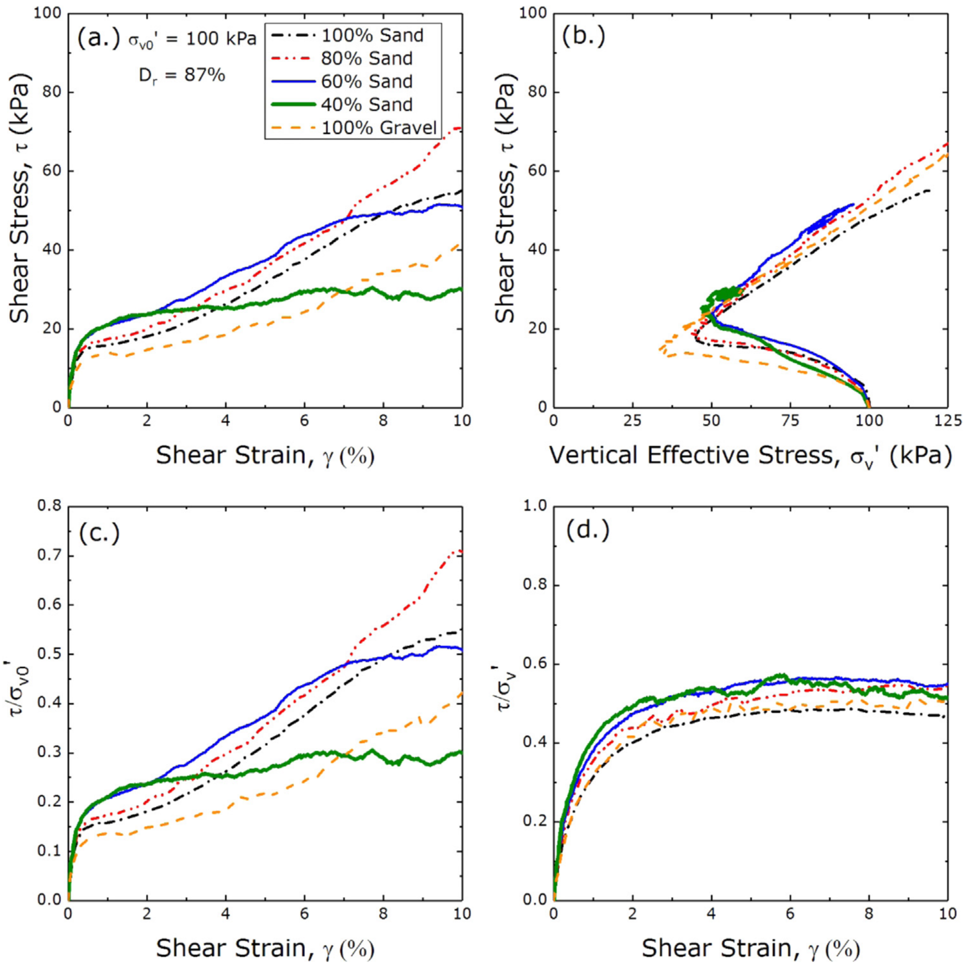

Fig. 6. Monotonic simple shear results for Pea Gravel mixtures at D_r = 87% and σ'_{v0} = 100 kPa for (a) Stress-strain response, (b) Stress path response, (c) τ/σ'_{v0} versus strain, and (d) τ/σ'_v versus strain.

To better illustrate the effects of relative density, 100% Sand, 40% Sand (the optimum mixture), and 100% Gravel τ/σ'_{v0} versus shear strain response at D_r = 47% and D_r = 87% were plotted in Fig. 7. This figure shows that the difference in τ/σ'_{v0} versus shear strain response for looser and denser specimens is greater for the uniform specimens of 100% Sand and 100% Gravel. As the mixture reaches the optimum percentage (i.e. 40% Sand for Pea Gravel mixtures), the effect of relative density is reduced. This response can be attributed to the small variation of void ratios of the specimens at the optimum mixture percentage in the loose and dense states. The D_r = 47% optimum mixture specimen has a global void ratio of 0.334, while the D_r = 87% optimum mixture specimen has a global void ratio of 0.294, which is generally a narrower range than that of the other specimens.

The effect of initial vertical stress on D_r = 47% specimens was investigated by plotting τ/σ'_{v0} versus shear strain as shown in Fig. 8 for 100% Sand, 40% Sand, and 100% Gravel. Fig. 8a shows that as initial vertical stress increases, the 100% Sand specimens display less strain hardening response in the post-peak range (i.e. the specimen dilates less

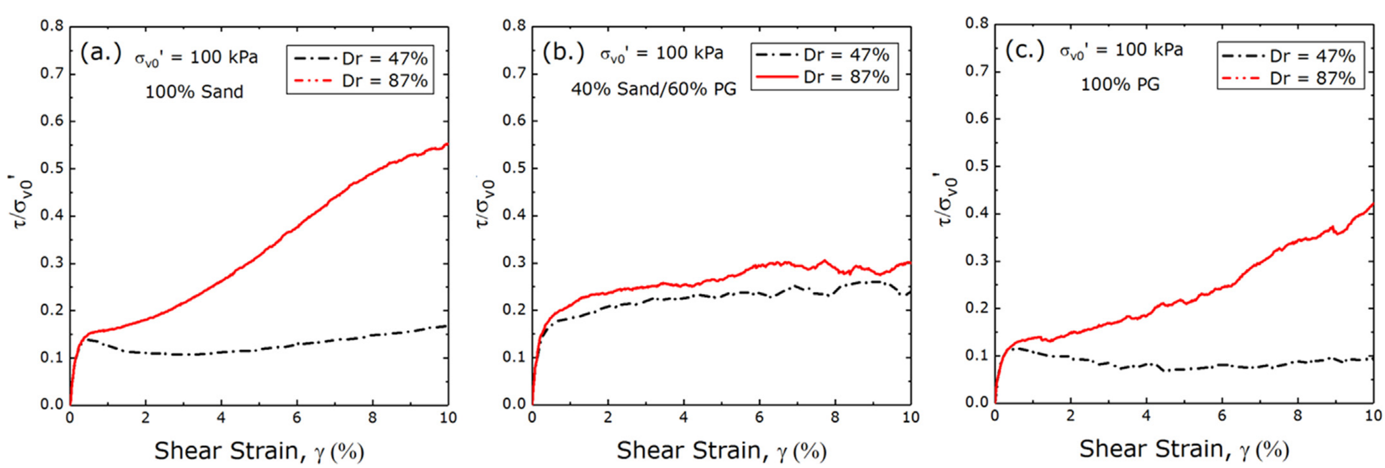

Fig. 7. Comparison of Effect of Relative Density for (a) 100% Sand, (b) 40% Sand/60% PG, and (c) 100% PG.

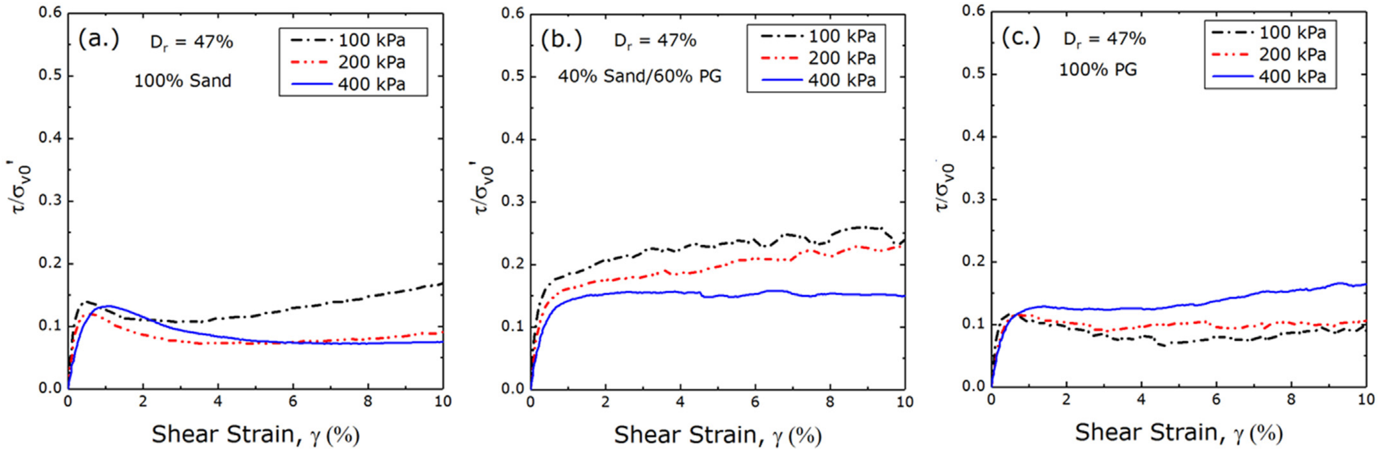

Fig. 8. Comparison of effect of initial vertical stress for (a) 100% sand, (b) 40% sand/60% PG, and (c) 100% PG.

post-peak). A different trend was observed for the 100% Gravel specimens in Fig. 8c, with the 400 kPa specimen attaining greater τ/σ'_{v0} values post-peak. Fig. 8b shows significant differences in both the peak and post-peak ranges for the 40% Sand mixture, with τ/σ'_{v0} values decreasing with increasing vertical stress. These figures also show that the uniform sand and gravel specimens soften post-peak and then strain harden, but the optimum mixture specimens do not soften at 100 or 200 kPa and do not harden at 400 kPa.

The monotonic response of CLS8 and Ottawa C109 sand mixtures was also investigated for mixture percentages of 100% Sand, 80% Sand, 60% Sand, 40% Sand, and 100% Gravel. Limited testing of 40% Sand specimens was completed since particle segregation was observed in these specimens because there was not enough sand to fill in the gravel void space. Fig. 9 shows the monotonic shear results for specimens at D_r = 47% and σ'_{v0} = 100 kPa. Fig. 9a shows that the 60% Sand specimen exhibits the greatest post-peak shear strength and is considered the optimum mixture percentage. The 60% Sand, 40% Sand, and 100% Gravel specimens had very similar peak shear strengths and all exhibit

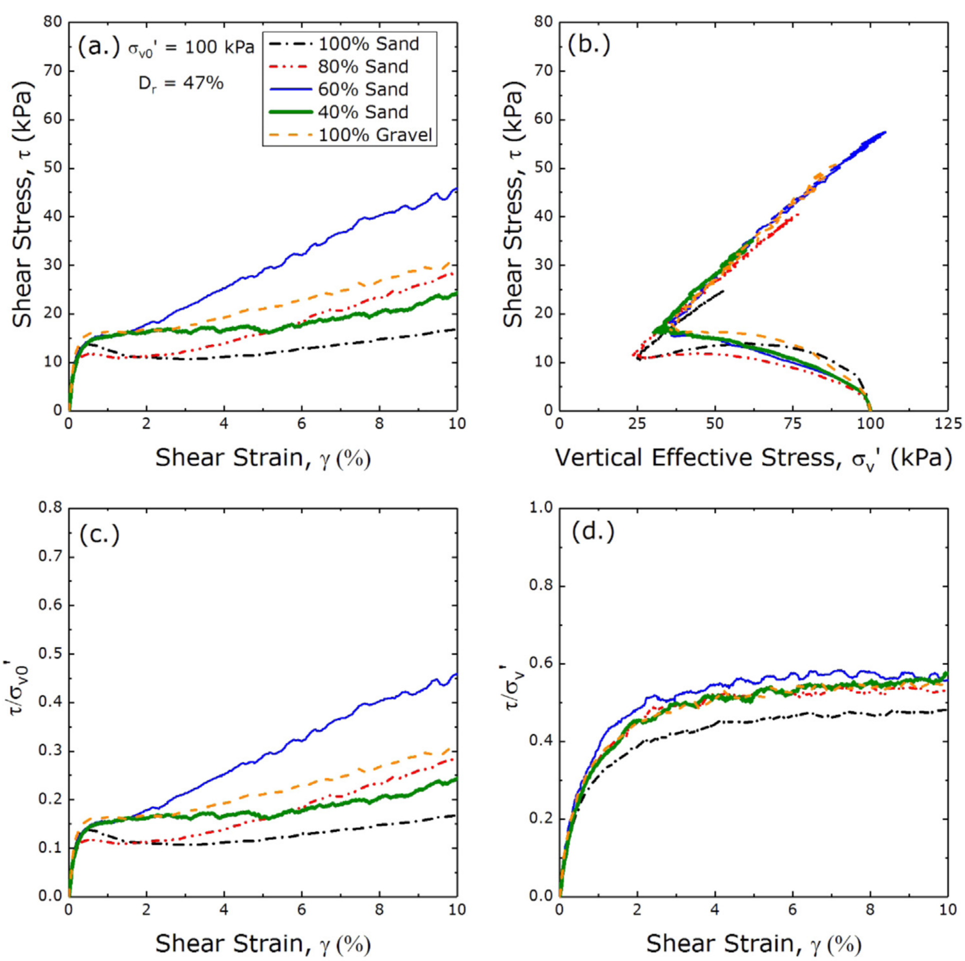

Fig. 9. Monotonic simple shear results for CLS8 mixtures at D_r = 47% and σ'_{v0} = 100 kPa for (a) Stress-strain response, (b) Stress path response, (c) τ/σ'_{v0} versus strain, and (d) τ/σ'_v versus strain.

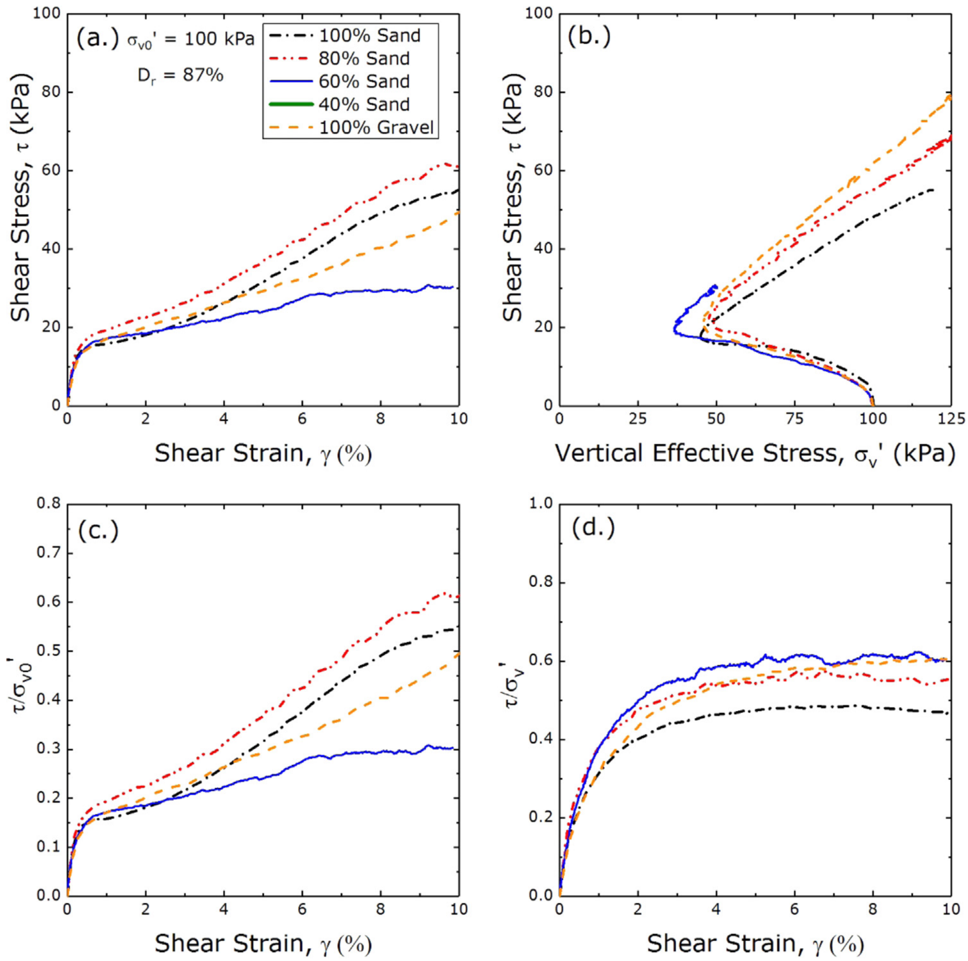

Fig. 10. Monotonic Simple Shear results for CLS8 mixtures at D_r = 87% and σ'_{v0} = 100 kPa for (a) Stress-strain response, (b) Stress path response, (c) τ/σ'_{v0} versus strain, and (d) τ/σ'_v versus strain.

strain hardening response, which suggests that the gravel portion of the specimen is controlling response once the gravel content has reached 40% (in the 60% Sand specimen). Fig. 9b shows the effect of mixture percentage on stress path response. Similar to the Pea Gravel mixtures, as the mixture percentage increases to the optimum, the response changes to fully strain hardening, with no post-peak softening. The uniform sand displays post-peak softening, while the gravel-sand mixtures and uniform gravel are strain hardening. Fig. 9d shows the effect of mixture percentage on τ/σ'_v ratio as strain increases. All tests reach an US, with the 60% Sand specimen having the highest τ/σ'_v ratio (approximately 0.58). This plot also shows that even 20% gravel has a significant influence on the US response since every specimen that contains gravel (even 80% Sand specimen) has a higher τ/σ'_v ratio than 100% Sand. Fig. 10 presents the stress-strain response for D_r = 87% specimens of CLS8 and Ottawa C109 sand mixtures. The optimum mixture percentage in these tests is the 80% Sand specimen, while the 60% Sand specimen, which had the greatest post-peak shear strength (both PT, US) for D_r = 47% specimens, has the weakest response. This response can be explained if the void ratio of the sand skeleton only is examined. The void ratio of sand skeleton only was 0.574, 0.559, and 0.590 for 100% Sand, 80% Sand, and 60% Sand, respectively. The 80% Sand, which had a sand skeleton void ratio of 0.559, had the greatest shear strength, while the 60% Sand specimen had the highest sand skeleton void ratio and exhibited the lowest shear strength. The void ratio of the 100% Gravel (global void ratio) was 0.575 and this specimen displayed response similar to the 100% Sand which had a 0.574 void ratio. Therefore, the response can be explained by considering the sand skeleton void ratio.

To further investigate the effect of relative density, τ/σ'_{v0} versus shear strain was plotted in Fig. 11 for 100% Sand, 60% Sand, and 100% Gravel. The 100% Sand and 100% Gravel display greater τ/σ'_{v0} values post-peak (i.e. more strain hardening occurring in the dense specimens). The 60% Sand data shows that the D_r = 87% specimen displays less strain hardening post-peak. The global void ratios for these specimens are very similar with the D_r = 47% specimen having a void ratio of 0.394 and the D_r = 87% specimen having a void ratio of 0.354. Therefore, when specimens are near the optimum mixture, the range of possible void ratios is lower and the specimens display similar response at low and high relative densities. The effect of initial vertical stress for CLS8 mixtures was also investigated by plotting τ/σ'_{v0} versus shear strain as shown in Fig. 12 for 100% Sand, 60% Sand, and 100% Gravel. For each mixture percentage, the post-peak response is less strain hardening as initial vertical stress increases. The 100% Sand specimens exhibit post-peak strain softening and then hardening; however, the 100% Gravel and 60% Sand specimens are fully strain hardening. This observation is important to consider when evaluating response at larger shear strains (greater than 2%).

Data from D_r = 47% specimens at 100, 200, and 400 kPa were used to evaluate the peak, PT, and US lines in Figs. 13 and 14. In Fig. 13 the peak, PT, and US shear strengths are plotted for Pea Gravel mixtures. Lines were drawn on the figure showing the peak strengths to more easily highlight the trends. 40% Sand had the highest peak line while the 100% Sand had the lowest peak line with the other mixtures falling in between. Fig. 13b shows that the PT points fall along a similar line (τ/σ'_v = 0.47), which is expected since the PT of the uniform sand and gravel is very similar. Similar trends are also observed for the US

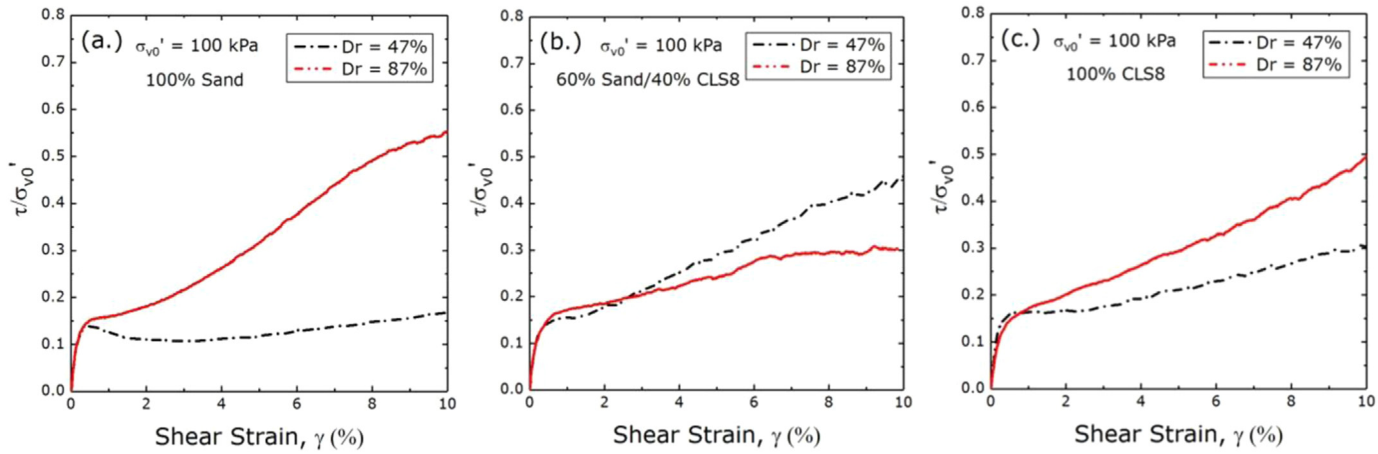

Fig. 11. Comparison of effect of relative density for (a) 100% Sand, (b) 60% Sand/40% CLS8, and (c) 100% CLS8.

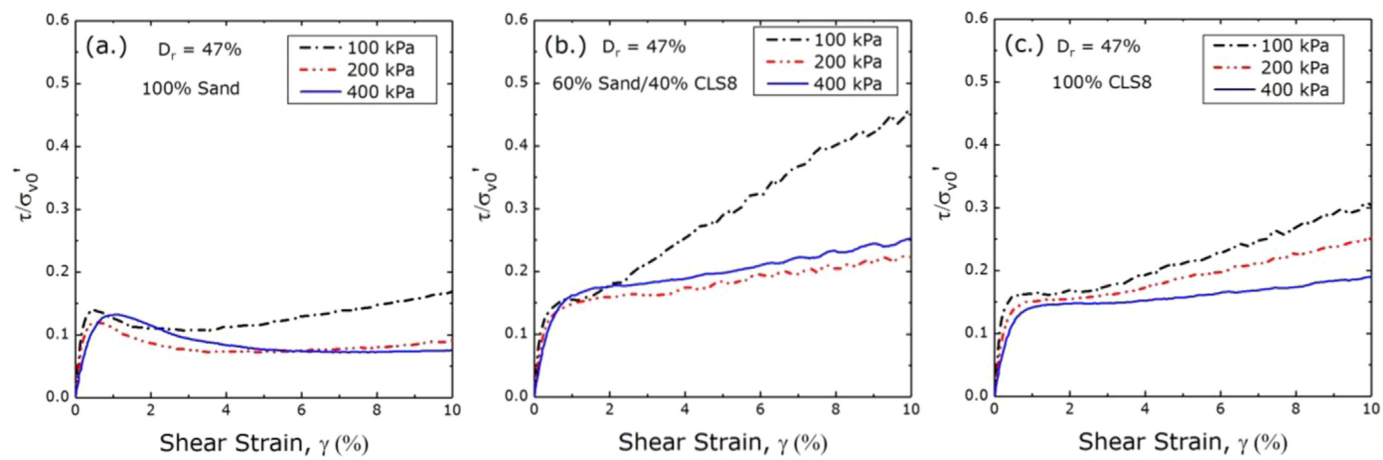

Fig. 12. Comparison of effect of initial vertical stress for (a) 100% Sand, (b) 60% Sand/40% CLS8, and (c) 100% CLS8.

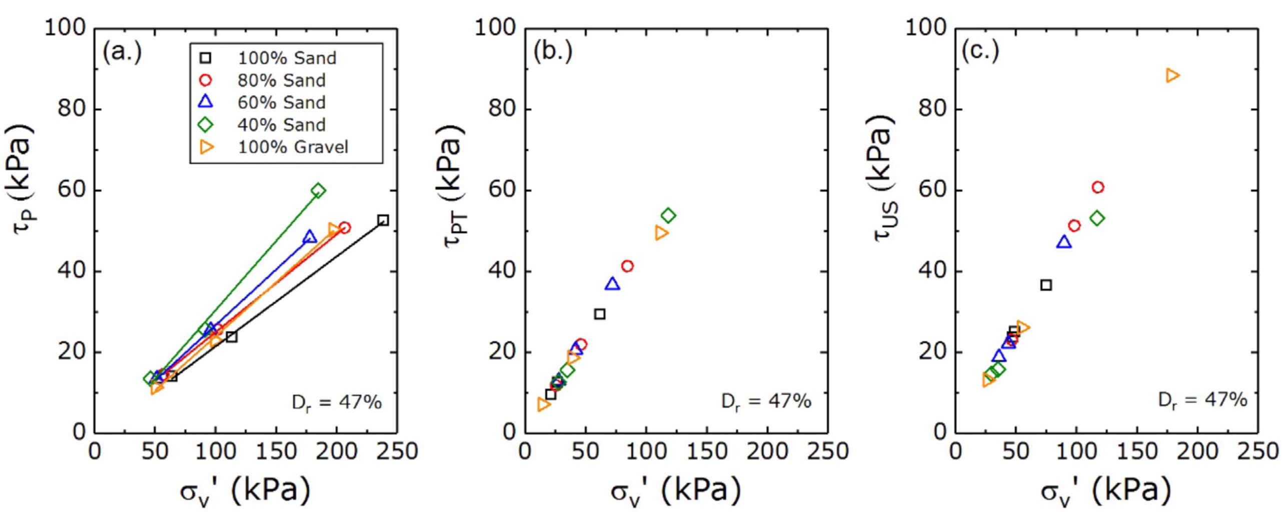

Fig. 13. Shear strength and vertical effective stress at (a) Peak, (b) Phase transformation, and (c) Ultimate state for Pea Gravel mixtures at D_r = 47%.

response in Fig. 13c. Fig. 14 illustrates the peak, PT, and US points for CLS8 mixtures. In this case, 60% Sand had the highest peak line, while the 100% Sand had the lowest peak line. A different response is observed for the CLS8 mixtures compared to the Pea Gravel mixtures, since the uniform CLS8 is stronger than the Pea Gravel due to increased particle angularity [18]. Therefore, for the CLS8 mixtures, the 80% Sand specimen peak line falls below the 100% Gravel which was not the case for Pea Gravel mixtures. A new optimum mixture percentage is also observed (40% Sand for CLS8 mixtures compared to 60% Sand for Pea Gravel mixtures). The PT points fall along a similar line again for the CLS8 mixtures ($\tau/\sigma_v' = 0.50$ for optimum mixture), but the value is greater than the Pea Gravel mixtures ($\tau/\sigma_v' = 0.46$ for optimum mixture), which is attributed to the increased shear resistance of the CLS8 gravel due to increased angularity. There is more scatter in the US

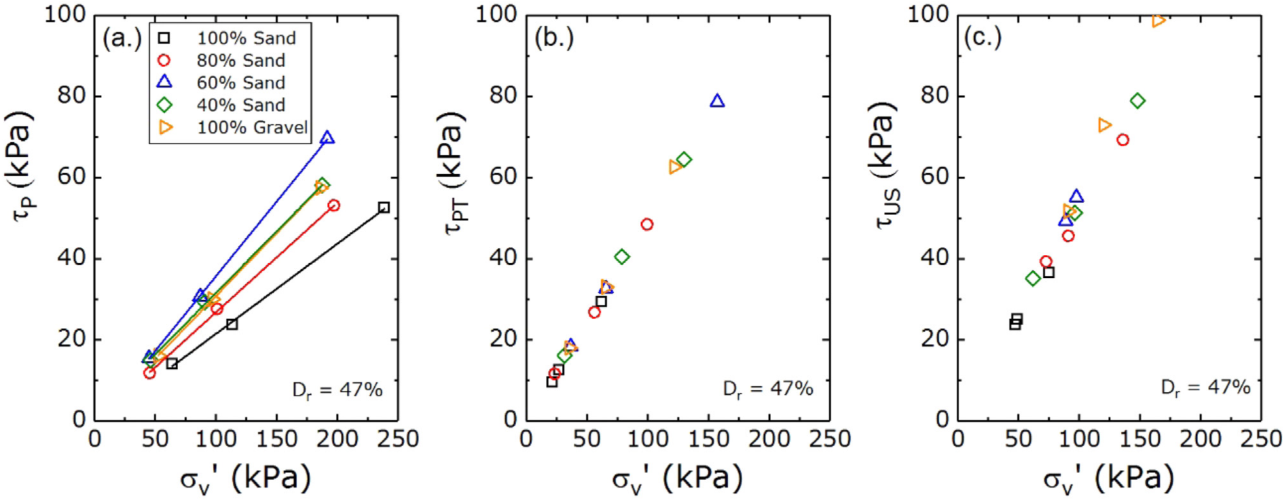

Fig. 14. Shear strength and vertical effective stress at (a) Peak, (b) Phase transformation, and (c) Ultimate state for CLS8 mixtures at D_r = 47%.

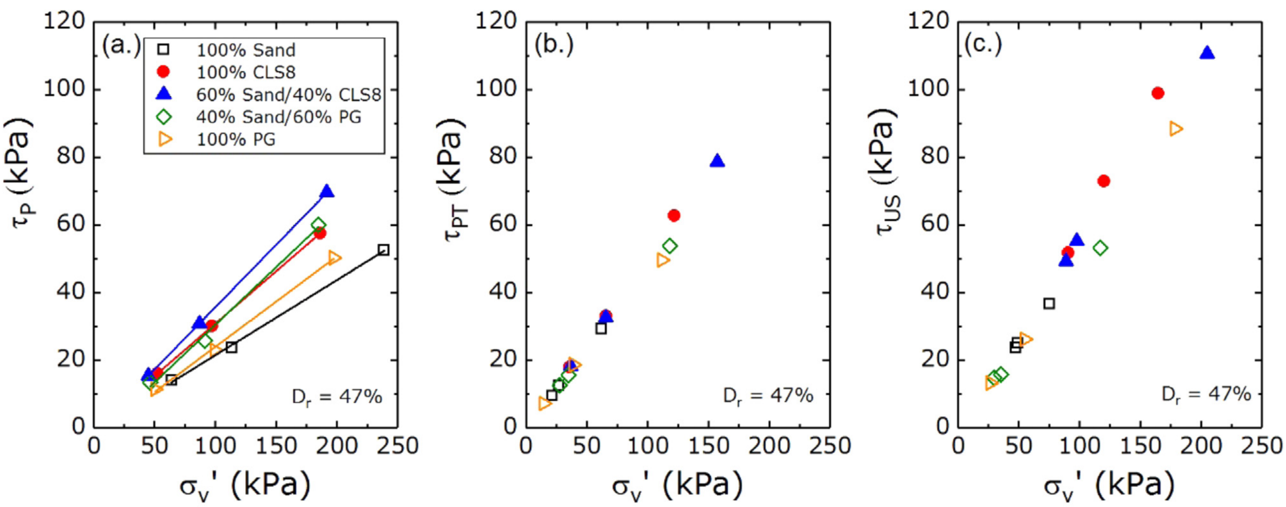

Fig. 15. Shear strength and vertical effective stress at (a) Peak, (b) Phase transformation, and (c) Ultimate state for evaluation of particle angularity effects.

response with 100% Sand having a lower US value than the 100% Gravel. The 60% and 40% Sand specimens have a similar US to the 100% Gravel, which shows that the gravel portion is controlling US response in those mixtures.

To further examine the effect of particle angularity, the peak, PT, and US for the optimum mixture percentages for Pea Gravel (40% Sand/60% PG) and CLS8 (60% Sand/40% CLS8) were plotted in Fig. 15 with 100% Sand, 100% CLS8, and 100% Pea Gravel for D_r = 47% specimens. The comparison shows that for the peak, PT, and US, the optimum mixture for the angular CLS8 gravel-sand mixture has the greatest shear resistance. Fig. 15a shows that the peak line for the optimum mixture for the subrounded Pea Gravel mixture is approximately the same as the 100% CLS8 gravel specimen. The 100% Sand and 100% Pea Gravel specimens display the least shear resistance as they are subrounded and uniformly graded. Fig. 15b shows that the PT lines are dependent on particle morphology, with the 100% CLS8 and 60% Sand/40% CLS8 mixture falling along a similar PT line which is above (i.e. greater shear resistance) the 100% Sand, 100% Pea Gravel, and 40% Sand/100% Pea Gravel mixture. Similarly, Fig. 15c shows that the US line for the 100% CLS8 and the optimum CLS8 mixture plot above the 100% Sand, 100% Pea Gravel, and 40% Sand/60% Pea Gravel specimens. As previously discussed, the gravel fraction controls the specimen's post-peak response, and therefore the angularity of the gravel is an important parameter when evaluating PT and US shear response.

3.3. Cyclic shear response

Constant volume cyclic simple shear tests were also performed on mixtures of Pea Gravel and Ottawa C109 sand as well as CLS8 and Ottawa C109 sand. Specific parameters, including mixture percentage, CSR, and initial vertical stress were targeted for evaluation. The baseline tests were performed at CSR = 0.09 and at an initial vertical stress of 100 kPa. Liquefaction was defined as the attainment of 3.75% single amplitude shear strain. Fig. 16 shows example results for a mixture of 40% Ottawa C109 sand and 60% Pea Gravel for stress-strain (Fig. 16a), stress path (Fig. 16b), pore pressure generation (Fig. 16c), and strain development (Fig. 16d).

3.3.1. Effect of mixture percentage

Fig. 17a presents data for Pea Gravel mixtures at D_r = 47% and σ'_{v0} = 100 kPa that were tested until liquefaction at a CSR = 0.09. This figure shows that the optimum mixture percentage according to the monotonic test results (40% Sand specimen (V_S = 201 m/s)) also exhibits the greatest resistance to liquefaction. The 100% Sand (V_S = 190 m/s) and 100% Gravel (V_S = 180 m/s) specimens liquefy in the same number of cycles, which is expected given the similarity of these materials from a particle morphology perspective (i.e., roundness) [18].

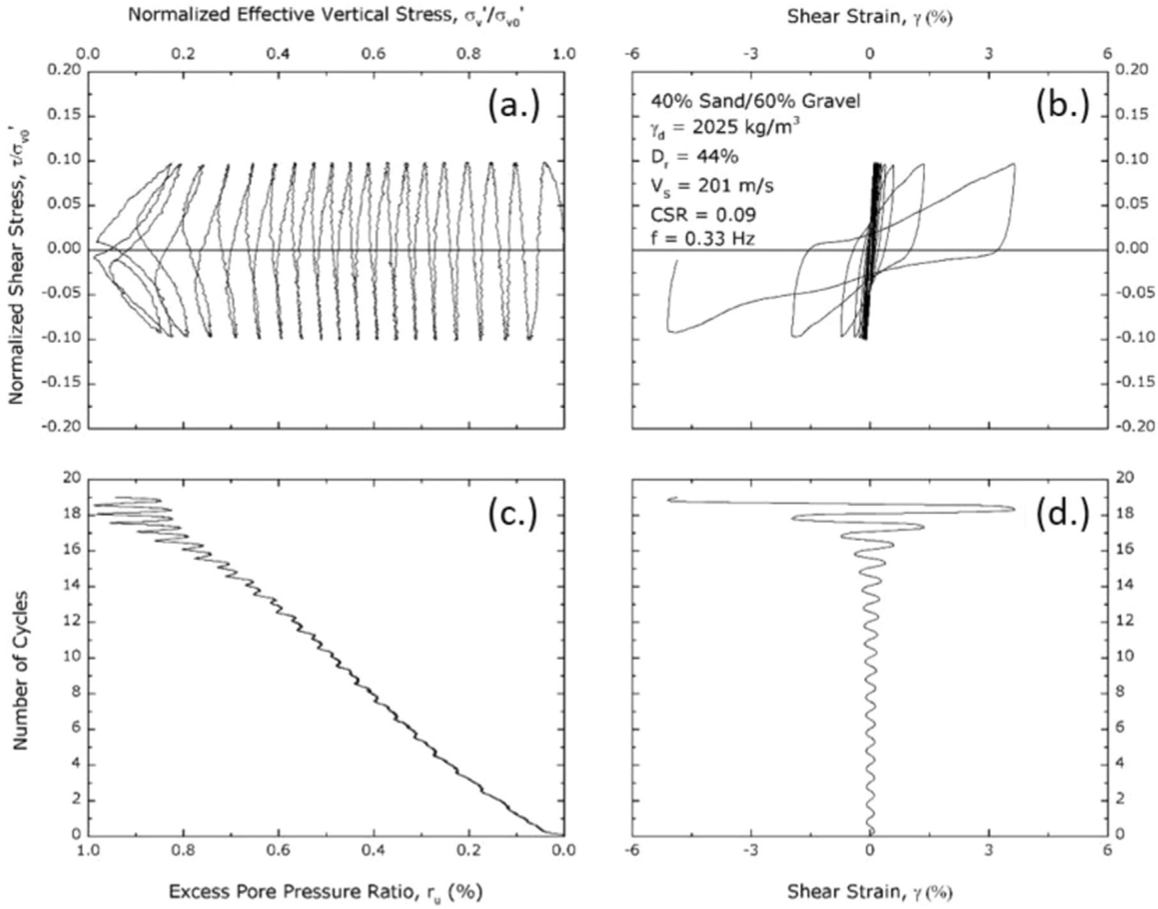

Fig. 16. Cyclic simple shear results for 40% Sand/60% CLS8 Gravel at D_r = 44% and CSR = 0.09.

Sand controls the cyclic response for the 80% Sand (V_S = 205 m/s) and 60% Sand (V_S = 217 m/s) specimens as evidenced by their number of cycles to liquefaction increasing by only 1 and 2 cycles, respectively, compared to 100% Sand, but the measured V_s is higher. Although the 60% Sand specimen had the highest V_S at 217 m/s it liquefied in fewer cycles than the 40% Sand which had a V_S of 201 m/s. The lower V_S of the 40% Sand specimen can be attributed to the higher gravel content (which has a V_S of 180 m/s for 100% Gravel which is less than 100% Sand). Similar results were also observed for the CLS8 mixtures as shown in Fig. 17b. The optimum mixture percentage from the monotonic test results again displayed the greatest resistance to liquefaction (60% Sand specimen (V_S = 230 m/s)). For the CLS8 mixtures, the 100% Gravel (V_S = 204 m/s) displays significant resistance to liquefaction and responds quite differently than 100% Sand, a difference that can be attributed to their differences in particle morphology (i.e. angular vs. rounded) as described in Hubler et al. [18]. A specimen with 80% Sand (V_S = 188 m/s) responded similarly to a specimen with 100% Sand indicating that the gravel skeleton was not involved in load transfer during cyclic shearing (i.e. the gravel was floating within the sand matrix). For the CLS8 mixtures an increase in V_S corresponded to an increase in liquefaction resistance.

3.3.2. Effect of CSR

Fig. 18a presents the results for Pea Gravel mixtures at D_r = 47% and σ'_{v0} = 100 kPa for CSRs of 0.04, 0.09, and 0.14. The b values from fitted power functions ranged from approximately 0.20 to 0.40. CSR was found to have an effect on the liquefaction response of the Pea Gravel mixtures. For CSR = 0.14 tests, the 60% Sand and 40% Sand specimens displayed higher resistance to liquefaction than the other mixes, while at CSR = 0.09, only 40% Sand shows an increase in resistance compared to the other mixtures. For CSR = 0.04, 80% Sand, 100% Sand, and 100% Gravel exhibit significantly more resistance to liquefaction compared to the 60% Sand and 40% Sand mixtures. This shows that at the lower CSR values the mixtures near the optimum mixture percentage (40% Sand) are not as resistant to liquefaction as the uniform materials. This could be explained by examining the particle interaction between the sand and gravel.

Similar results were also observed for the CLS8 mixtures as shown in Fig. 18b. The b values from fitted power functions ranged from approximately 0.20 to 0.35. For CSR = 0.14 tests, 100% Gravel, 80% Sand, and 60% Sand liquefy in a similar number of cycles and exhibit higher resistance than the 100% Sand specimen. The 80% Sand/20% CLS8 specimen responds similarly to the 100% Gravel, while the Pea Gravel mixtures with 80% Sand responded similarly to the 100% Sand for CSR = 0.14. This suggests that at CSR = 0.14 the angular CLS8 is contributing to the load transfer for the 80% Sand/20% CLS8 specimen. For CSR = 0.14 tests, larger strains are applied which means that more movement of particles occurs in the specimen and for the CLS8 gravel engagement between the particles during these larger strains could lead to greater resistance to the cyclic loading. For CSR = 0.09 tests, the 80% Sand specimen responds similarly to the 100% Sand. This could be due to the smaller strains involved in the loading mechanism which do not allow the gravel particles to fully engage and contribute to the load transfer. 60% Sand and 100% Gravel displays the most resistance to liquefaction at CSR = 0.09. For CSR = 0.04 tests, again the uniform specimens (100% Sand and 100% Gravel) exhibited the most resistance to liquefaction compared to the 80% Sand and 60% Sand specimens. The data presented here shows that the level of CSR (and therefore

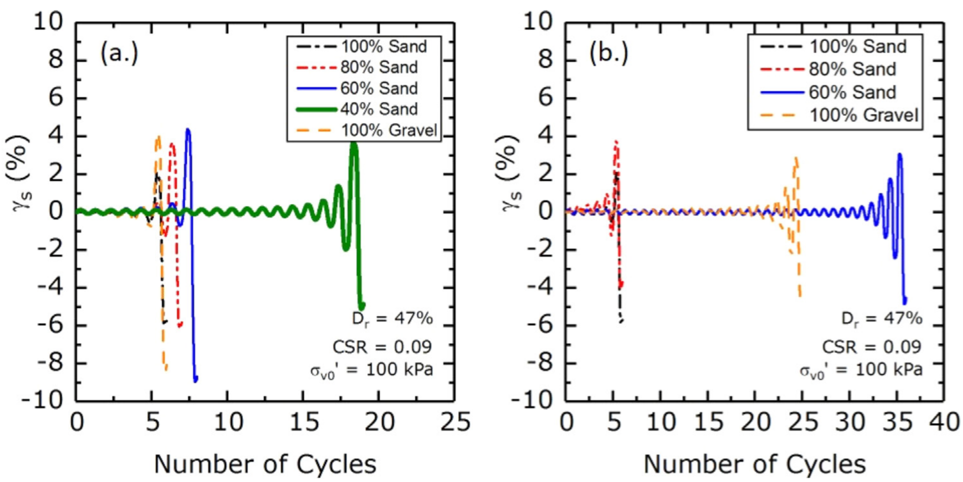

Fig. 17. Shear strain versus number of cycles to liquefaction for (a) Pea Gravel mixtures and (b) CLS8 mixtures.

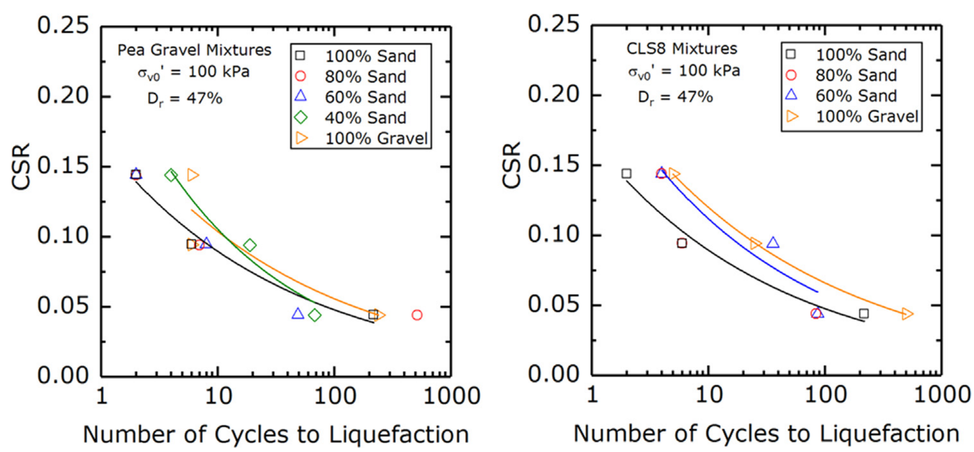

Fig. 18. CSR versus number of cycles to liquefaction for (a) Pea Gravel mixtures and (b) CLS8 mixtures.

strain applied) has an effect on the liquefaction resistance of gravel-sand mixtures.

3.3.3. Effect of initial vertical stress

Fig. 19a presents results for tests on Pea Gravel mixtures at $D_r = 47\%$ and a CSR = 0.09 at vertical stresses of 100, 200, and 400 kPa. The results show that there is not a significant effect of initial vertical stress on the number of cycles to liquefaction. The 400 kPa specimen (V_{S1} = N/A) liquefied in 17 cycles, the 200 kPa specimen (V_{S1} = 197 m/s) liquefied in 20 cycles, and the 100 kPa specimen (V_{S1} = 201 m/s) liquefied in 18 cycles. These differences are not significant and the response can be considered independent of initial vertical stress.

Similarly, Fig. 19b presents the results for CLS8 mixtures tested at $D_r = 47\%$ and a CSR = 0.09 at 100 kPa and 400 kPa. For the CLS8 mixtures, an effect of initial vertical stress is observed. The 100 kPa

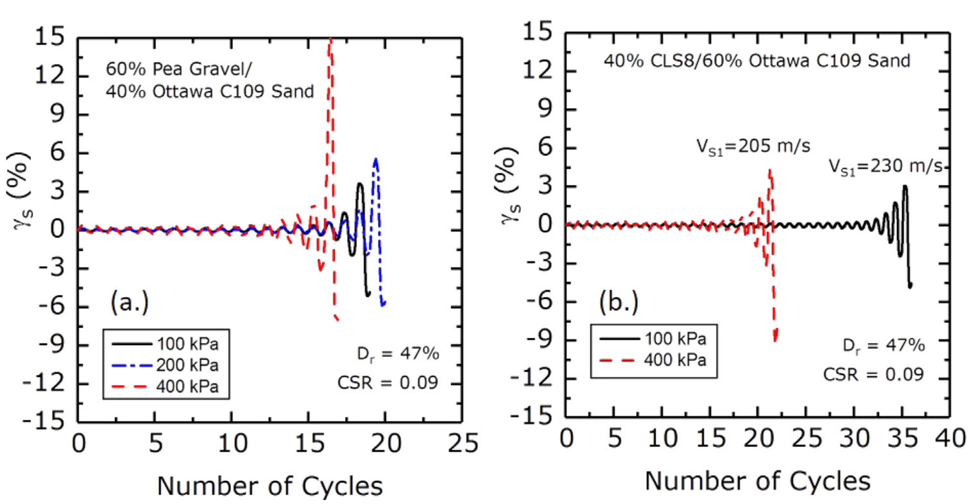

Fig. 19. Comparison of the effect of initial vertical stress for (a) Optimum Pea Gravel mixture and (b) Optimum CLS8 mixture.

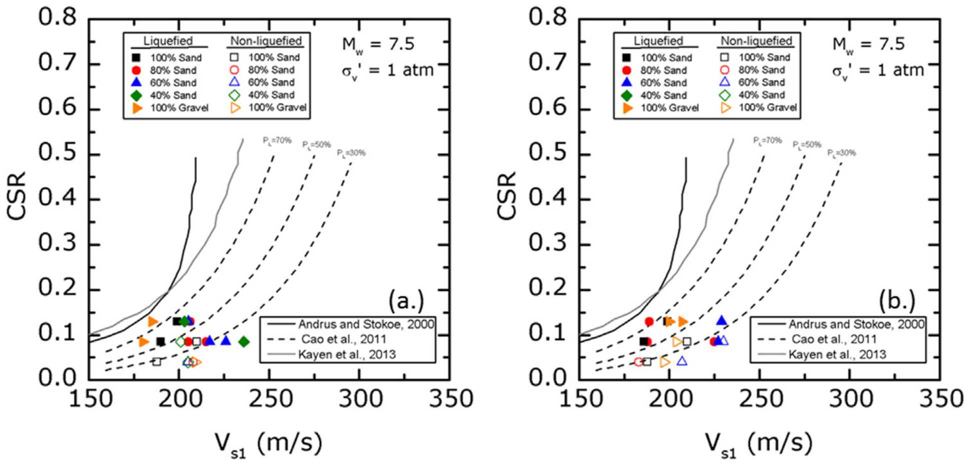

Fig. 20. Comparison with existing liquefaction triggering charts for gravelly soils and sands for (a) Pea Gravel mixtures and (b) CLS8 mixtures ($N_L = 15$, Liquefaction = 3.75% Single amplitude shear strain).

specimen (V_{S1} = 230 m/s) liquefied in 36 cycles while the 400 kPa specimen (V_{S1} = 205 m/s) liquefied in 22 cycles. This difference in response may be attributed to the tendency for less strain hardening (i.e. dilation) of the 400 kPa specimen (as shown in Fig. 12b). It is also possible that there are differences in specimen fabric as evidenced by the difference in V_{S1} values. The 100 kPa tests had a V_{S1} value of 230 m/s while the 400 kPa specimen had a V_{S1} value of 205 m/s.

3.3.4. Comparison with existing field-based liquefaction triggering charts

Since V_S was measured following consolidation and before cyclic shearing, comparisons of data from this study (CSR, V_{S1}) were made with existing liquefaction triggering charts based on field case histories for sandy and gravelly soils. The method for correcting data and plotting on the chart have been described in Hubler et al. [18].

Fig. 20a shows a comparison of the Pea Gravel mixture data with existing relationships from Andrus and Stokoe [2] for gravels, Kayen et al. [21] for sands (P_L = 15%), and Cao et al. [5] for gravels. Results show that the Pea Gravel mixtures liquefied in the laboratory at V_{S1} values higher than 200 m/s and as high as approximately 240 m/s, which is consistent with the findings of Hubler et al. [18]. All of the specimens that liquefied, including 100% Sand, would have been predicted as non-liquefiable in the field according to both Andrus and Stokoe [2] and Kayen et al. [21] for sands. The 100% Sand specimens fell slightly below the Kayen et al. [21] line. Previous researchers have shown that the CRR-V_{S1} relationship lines are material dependent (i.e. response depends on particle morphology) and may not always fit into the field-based V_S charts [4,28]. Laboratory-based data from Tokimatsu et al. [28] for sands and Baxter et al. [4] for silt are consistent with the data in this study for sand-gravel mixtures. Similarly, Fig. 20b plots the CLS8 mixtures with existing relationships from Andrus and Stokoe [2] for gravels, Kayen et al. [21] for sands, and Cao et al. [5] for gravels. Results show that the CLS8 mixtures liquefied at V_{S1} values higher than 200 m/s and as high as approximately 230 m/s. These findings suggest that gravelly soils liquefy in the laboratory at higher Vs than predicted based on field-based liquefaction triggering tests. This highlights the need for further investigation of the liquefaction behavior of gravelly soils, and especially comparing the “element tests” in the laboratory with the field response based on surficial observations.

4. Conclusions

The monotonic and cyclic shear response of gravel-sand mixtures were evaluated in this study. Data was compared throughout the study for specimens at the same relative density. V_S was measured in each specimen and was also used for comparison of data. The data presented in this study represents some of the first cyclic simple shear data for gravel-sand mixtures tested in the laboratory. The following are conclusions of this study:

- For subrounded Pea Gravel/Ottawa C109 sand mixtures, there was not a significant change in V_S as the mixture percentages changed. This is likely due to the similarity in V_S of the uniform Pea Gravel and uniform Ottawa C109 sand. The mixture with the highest V_S value was the 40% Sand/60% Gravel mixture. Alternatively, mixture percentage had a significant effect on the V_S values for the CLS8/Ottawa C109 sand mixtures. In this case, the uniform angular CLS8 and uniform Ottawa C109 sand had different V_S values. The mixture with the highest V_S value was the 60% Sand/40% Gravel mixture, followed by the 100% Gravel specimen.
- The stress-strain response of gravel-sand mixtures is different compared to the uniform gravel and sands, particularly at the loose state and for subrounded materials. The subrounded uniform loose sand and Pea Gravel exhibit a characteristic peak stress followed by strain softening to a minimum shear resistance that is followed by strain hardening response at large strains. The angular uniform CLS8 does not display post-peak strain softening, which highlights the importance of particle morphology of the gravel-sand mixture materials. For gravel-sand mixtures, the strain softening and strain hardening becomes less pronounced, and for the optimum gravel-sand mixture, no strain softening is observed and strain hardening is less pronounced or completely absent.
- The percentage of gravel and sand in a mixture affects the monotonic shear response. There exists an optimum mixture percentage of gravel and sand that maximizes shear strength. The optimum mixture percentage for D_r = 47% specimens was found to be 60% Pea Gravel/40% Ottawa C109 sand for Pea Gravel mixtures and 40% CLS8/60% Ottawa C109 sand for CLS8 mixtures. This optimum mixture had the highest peak line (or friction angle) during monotonic testing as well as the highest V_S value. Initial vertical stress was shown to not influence the optimum mixture percentage, but did affect the stress-strain response. Relative density affected the optimum mixture percentage for dense monotonic specimens; however, further testing is required for dense specimens.
- The Phase Transformation points were not affected by mixture percentage for the Pea Gravel mixtures, but increased to the level of 100% CLS8 gravel with the addition of only 20% CLS8 to the CLS8 mixtures.
- τ/σ_v' ratio was nearly constant (approximately 0.50) at larger strains (i.e. the US) for Pea Gravel mixtures, regardless of mixture percentage; however, for CLS8 mixtures the τ/σ_v' ratio at the US was dependent on mixture percentage and varied from 0.48 to 0.58. The τ/σ_v' ratio did not change significantly for the Pea Gravel mixtures

since the uniform Pea Gravel and Ottawa C109 sand had similar shear response when tested independently. Conversely, CLS8 and Ottawa C109 had different shear response when tested independently, which led to different results when mixed, highlighting the possible effect of particle angularity.

- Mixture percentage was shown to also affect the cyclic response of gravel-sand mixtures. For tests at CSR = 0.09 and D_r = 47%, the optimum mixture that was found from the monotonic shear tests exhibited the greatest resistance to liquefaction.
- The effect of CSR on liquefaction resistance was not the same for all gravel-sand mixtures tested in this study. For tests at CSR = 0.04, mixtures of 60% Sand and 40% Sand were the least resistant to liquefaction (compared to the other mixtures and the 100% Gravel and 100% Sand), while at CSR = 0.14 these mixtures were the most resistant for both mixtures of Pea Gravel and CLS8. This could possibly be explained by examining the particle interaction between the sand and gravel. It is possible that for certain mixtures at the lower CSR value (and therefore lower strain level) the gravel is not engaged in the small cyclic movements (i.e. the sand is controlling the response). In the 60% Sand and 40% Sand specimens mixed with Pea Gravel, the sand in between the gravel is also in a looser state than at D_r = 47% (e = 0.64), which could explain the fewer number of cycles to liquefaction if the sand is controlling the response. The sand skeleton void ratio for the 60% Sand specimen at D_r = 47% is 0.70, while the sand skeleton void ratio for the 40% Sand specimen at D_r = 47% is 0.83. This shows that the sand skeleton is likely controlling response and therefore liquefying in fewer cycles than the e = 0.64 (D_r = 47%) 100% Sand specimen.
- The particle angularity of the gravel in a gravel-sand mixture affects the response during monotonic and cyclic tests. The subrounded Pea Gravel mixtures had a different optimum mixture percentage than the angular CLS8 gravel mixtures.
- Initial vertical stress was shown to not have a significant effect on the liquefaction resistance of Pea Gravel mixtures. CLS8 mixtures exhibited an effect of initial vertical stress, and this could be due to lower dilation occurring as initial vertical stress increases as well as differences in specimen V_S.
- The tested gravel-sand mixtures liquefied at V_{S1} values higher than 200 m/s and as high as approximately 240 m/s. All of the specimens that liquefied, including the clean sand, would have been predicted as non-liquefiable according to existing field-based triggering correlations [2,21].

Acknowledgments

This material is based upon work supported by the National Science Foundation Graduate Student Research Fellowship under Grant No. DGE 1256260 and by the National Science Foundation CAREER Grant No. 1351403 and CMMI grant No. 1663288. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the National Science Foundation. ConeTec Investigations Ltd. and the ConeTec Education Foundation are acknowledged for their support to the Geotechnical Engineering Laboratories at the University of Michigan. The authors would also like to thank Zonta Owens for his aid in specimen preparation and laboratory testing.

References

[1] Andrus RD. In situ characterization of gravelly soils that liquefied in the 1983 Borah Peak earthquake[Ph.D. thesis]. Austin, TX: University of Texas at Austin; 1994.
[2] Andrus RD, Stokoe II KH. Liquefaction resistance of soils from shear-wave velocity. J Geotech Geoenviron Eng 2000;126(11):1015–25.
[3] Banerjee NG, Chan CK, Seed HB. Cyclic behavior of dense coarse grained materials in relation to the seismic stability of dams. Berkeley, CA: University of California; 1979.
[4] Baxter CD, Bradshaw AS, Green RA, Wang JH. Correlation between cyclic resistance and shear-wave velocity for providence silts. J Geotech Geoenviron Eng 2008;134(1):37–46.
[5] Cao Z, Youd TL, Yuan X. Gravelly soils that liquefied during 2008 Wenchuan, China earthquake, M s = 8.0. Soil Dyn Earthq Eng 2011;31(8):1132–43.
[6] Chang WJ, Chang CW, Zeng JK. Liquefaction characteristics of gap-graded gravelly soils in K_0 condition. Soil Dyn Earthq Eng 2014;56:74–85.
[7] Chang WJ, Phantachang T. Effects of gravel content on shear resistance of gravelly soils. Eng Geol 2016;207:78–90.
[8] Dyvik R, Berre T, Lacasse S, Raadim B. Comparison of truly undrained and constant volume direct simple shear tests. Geotechnique 1987;37(1):3–10.
[9] Evans MD, Seed HB. Undrained cyclic triaxial testing of gravels – the effect of membrane compliance. 1987–07. Berkeley: Earthquake Engineering Research Center, University of California; 1987. p. 434. [UCB/EERC-87/08].
[10] Evans MD, Seed HB, Seed RB. Membrane compliance and liquefaction of sluiced gravel specimens. J Geotech Eng 1992;118(6):856–72.
[11] Evans MD, Zhou S. Liquefaction behavior of sand-gravel composites. J Geotech Engrg 1995;121(3):287–98.
[12] Fragaszy RJ, Su W, Siddiqi FH. Effects of oversize particles on the density of clean granular soils. Geotech Test J 1990;13(2):106–14.
[13] GEER (2014). “Earthquake Reconnaissance January 26th/February 2nd 2014 Cephalonia, Greece events, Version 1.” Nikolaou, S., Zekkos, D., Assimaki, D., Gilsanz, R. (Eds.), GEER/EERI/ATC, p. 492.
[14] GEER (2017). “Geotechnical Reconnaissance of the 2016 M_w 7.8 Kaikoura, New Zealand Earthquake, Version 1.0.” Cubrinovski, M. and Bray, J. (Eds.), GEER, p. 247.
[15] Harder Jr LF, Seed HB. Determination of penetration resistance for coarse-grained soils using the becker penetrationtest. Berkeley: University of California; 1986. [EERC Report No. UCB/EERC-86-06].
[16] Hatanaka M, Uchida A, Suzuki Y. Correlation between undrained cyclic shear strength and shear wave velocity for gravelly soils. Soils Found 1997;37(4):85–92.
[17] Holtz WG, Gibbs HJ. Triaxial shear tests on pervious gravelly soils. J Soil Mech Found Div, ASCE 1956;82(SM1):1–22.
[18] Hubler JF, Athanasopoulos-Zekkos A, Zekkos D. Monotonic, cyclic and post-cyclic simple shear response of three uniform gravels in constant volume conditions. J Geotech Geoenviron Eng 2017;143(9):04017043. [2017].
[19] Hubler JF, Athanasopoulos-Zekkos A, Zekkos D. Pore Pressure generation of pea gravel, sand, and gravel-sand mixtures in constant volume simple shear. In: Proceedings of the 3rd international conference on performance-based design in earthquake geotechnical engineering (PBD-III).
[20] Hubler JF. Laboratory and in-situ assessment of liquefaction of gravelly soils[Ph.D. dissertation]. AnnArbor: University of Michigan; 2017.
[21] Kayen R, Moss RES, Thompson EM, Seed RB, Cetin KO, Kiureghian AD, Tokimatsu K. Shear-wave velocity–based probabilistic and deterministic assessment of seismic soil liquefaction potential. J Geotech Geoenviron Eng 2013;139(3):407–19.
[22] Kokusho T, Hara T, Hiraoka R. Undrained shear strength of granular soils with different particle gradations. J Geotech Geoenviron Eng 2004;130(6):621–9.
[23] Menq F-Y. Dynamic properties of sandy and gravelly soils[Ph.D. thesis]. Austin: University of Texas; 2003.
[24] Nikolaou S, Zekkos D, Asimaki D, Gilsanz R. Reconnaissance Highlights of the 2014 sequence of earthquakes in Cephalonia, Greece. In: Proceedings of the 6th international conference on earthquake geotechnical engineering, 1–4 November 2015, Christchurch, New Zealand; 2015.
[25] Rashidian M. Undrained shearing behavior of gravelly sands and its relation with shear wave velocity[Ph.D. thesis]. Tokyo, Japan: University of Tokyo; 1995.
[26] Sivathayalan S. Fabric, initial state and stress path effects on liquefaction susceptibility of sands[Ph.D. thesis]. Vancouver, BC: University of British Columbia; 2000.
[27] Thevanayagam S. Effect of fines and confining stress on undrained shear strength of silty sands. J Geotech Geoenviron Eng 1998;124(6):479–91.
[28] Tokimatsu K, Yamazaki T, Yoshimi Y. Soil liquefaction evaluations by elastic shear moduli. Soils Found 1986;26(1):25–35.
[29] Wong RT, Seed HB, Chan CK. Liquefaction of gravelly soils under cyclic loading conditions. UCB/EERC-74/11, 1974-06, 46 p; 1974.
[30] Youd T, Harp E, Keefer D, Wilson R. The Borah Peak, Idaho Earthquake of October 28, 1983 Liquefaction. Earthq Spectra 1985;2(1):71–89.
[31] Zekkos D, Athanasopoulos-Zekkos A, Hubler JF, Fei X, Zehtab K, Marr A. Development of a large-size Cyclic simple shear device for characterization of ground materials with Oversized particles. ASTM Geotech Test J 2018;41(2GTJ20160271). https://doi.org/10.1520/GTJ2016027.