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# Effect of Plastic Fines on Liquefaction Characteristics of Gravelly Soil

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## ABSTRACT

To identify the effect of plastic fines on the liquefaction characteristics of gravelly soil, a series of cyclic undrained triaxial tests were conducted on the specimens. The liquefaction resistance, pore pressure and axial strain of the specimens were investigated, respectively. The contact state theory was proposed to categorize gravelly soil with different palstic fines content as three cases, which show different failure mechanisms. Pore pressure ratio of a 100% and accumulated axial strain of a 5% were adopted as the criterion for the cyclic liquefaction and the cyclic failure of the specimens, respectively.The testing results reveal that the liquefaction resistance decreased with the increase of plastic fines contents till reaching 10%, thereafter, it started to increase. Moreover, liquefaction resistance almost did not increase when the plastic fines content are more than 30%. The result of pore pressure ratios demonstrated the existence of phase transition point as plastic fines content increasing. This phase transition point transformed the liquefaction characteristics of gravelly soil from gravel soil-like to clay-like as plastic fines content increasing.

## Introduction

Gravelly soils may be encountered in natural soil strata, such as residual, fluvial, and glacial deposits, as well as man-made fills such embankments and reclaimed gravelly fills. Despite the high hydraulic conductivity and high small strain modulus of such soils, liquefaction of loose to medium dense gravels and gravelly sands have been observed in major earthquakes. the 1964 Alaska Mw9.2 earthquake (Ishihara 1985) in the United States of America, the 1975 Haicheng Ms7.3 (Wang 1984), the 1976 Tangshan Ms7.8 (Ishihara 1985) and the 2008 Wenchuan Ms8.0 earthquakes (Cao et al. 2011) in China, and the 1976 ML 6.2 and 6.1 subsequent earthquakes in North-eastern Italy (Sirovich 1996), the 1988 Armenia Ms6.8 earthquake (Yegian et al. 1994).

Previous studies performed on the gravelly soils reveal that the liquefaction resistance of gravelly soils is effected by the gravel content (Ishihara 1985; Evens and Zhou 1995; Lin et al. 2004; Chang et al. 2014). But few attention has been focused on the effect of fines contents on the gravelly soils. The liquefied case histories of gravelly soils used by Andrus and Stokoe (2000) to establish the correlation between the shear wave velocity and cyclic resistance ratio show that the fines content for liquefied natural gravelly soils is less than 5%, and that the highest fines content for liquefied man-made fills is 18%. But a unified conclusion has not yet formed.

In this paper, a series of cyclic undrained triaxial tests were conducted on the gravelly soil with various amounts of plastic fines. The failure mechanisms of the gravelly soil with different plastic fines content were studied. The contact state theory was proposed to explain the effect of plastic fines on the liquefaction characteristics of gravelly soil.

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## Cyclic Triaxial Tests on Reconstituted Gravelly Soil

### *Materials Used*

The gravelly soil used in this study with plastic fines content (*FC*) of 0%, 5%, 10%, 20%, 25%, 30%, 35% and 40% was man-made, plastic fine with a subangular shape and a mean particle size of 0.051 mm was used as the finer fraction. The gravel soil with a round shape and a particle size of 10 - 0.075 mm were used as the coarse fraction. The physical properties of the testing materials used in this study are given in Table 1. The specimens were prepared by adding plastic fines in various percentages (by weight of total specimens) to the gravel soil without plastic fines. The grain size distribution of gravel with different *FC* is presented in Figure 1.

Table 1. Index properties of component soils used in this study

| **Soil type** | **Gravel soil** | **Plastic fine** |
|---|---|---|
| Mean grain size $D_{50}$ (mm) | 1.85 | 0.051 |
| Uniformity coefficient ($C_u$) | 4.77 | 5.23 |
| Specific gravity ($G_s$) | 2.68 | 2.74 |
| Maximum index density (g/cm$^3$) | 2.086 | - |
| Minimum index density (g/cm$^3$) | 1.454 | - |
| Maximum index density ($e_{max}$) | 0.734 | - |
| Minimum index density ($e_{min}$) | 0.285 | - |
| Liquid limit (%) | - | 39.71 |
| Plastic limit (%) | - | 24.75 |
| Plasticity index (%) | - | 14.96 |

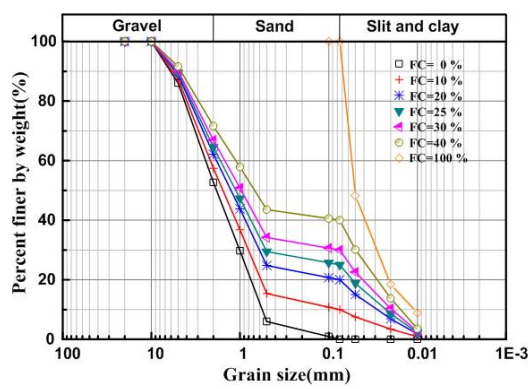


Figure 1. Grain size distribution of the specimens with different plastic fines content

### *Specimen Preparation, Saturation and Consolidation*

Specimens used in this study were 100 mm in diameter and 200 mm in height. The specimens were formed using tamping method in a split mold. The inner diameter of the molds is 100 mm and height 200 mm. The pure clean gravel soil is mixed with plastic fines and then compacted in several equal layers by a hammer that delivers some blows to each layer to achieve the target void ratio (*e*) 0.6. Number of layers and number of blows per layer were determined by trial to achieve the target void ratio. The hammer weighs 1 kg.

Once the preparation of the specimen was complete, initial saturation of the specimen was done by passing carbon dioxide through the specimen for 1 h. After that the distilled water was passed through the specimen by a gravity pressure of 5 kPa for 2 h. At the end of this process the cyclic machine was switched on. The machine is capable of applying sufficient back pressure till it was ensured that Skempton's *B* parameter is higher than 0.95. The specimen were then isotropically consolidated to a initial effective confining stress ($\sigma_{3c}'$). The duration for the process of consolidation was varied from about 3 h (for *FC* = 0 ) to 10 h (for *FC* = 40%)

### *Testing Apparatus and Experimental Program*

The testing apparatus used in the current study is English GDS hollow cylinder torsional shear apparatus (HCA). The HCA was partly modified to perform the tests of a solid cylinder specimen (Sun et al. 2014). All tests were conducted at a cyclic loading frequency ($f$) of 1 Hz. The excess pore pressure was measured at the top of each specimen. The specimens were then loaded with a sinusoidal axial stress at the appropriate cyclic stress ratio ($CSR = \sigma_d/2\sigma_{3c}'$) until they liquefied. Experimental program are shown in Table 2.

Table 2. The undrained cyclic triaxial experimental program

| ***FC*(%)** | **$\sigma_{3c}'$(kPa)** | ***CSR*** | ***e*** | ***FC*(%)** | **$\sigma_{3c}'$(kPa)** | ***CSR*** | ***e*** |
|---|---|---|---|---|---|---|---|
| 0 | 200 | 0.25-0.35 | 0.595~0.612 | 25 | 200 | 0.25-0.40 | 0.595~0.612 |
| 0 | 100 | 0.3 | 0.595~0.612 | 25 | 100 | 0.3 | 0.595~0.612 |
| 5 | 200 | 0.30-0.40 | 0.595~0.612 | 30 | 200 | 0.35-0.45 | 0.595~0.612 |
| 5 | 100 | 0.3 | 0.595~0.612 | 30 | 100 | 0.3 | 0.595~0.612 |
| 10 | 200 | 0.25-0.35 | 0.595~0.612 | 35 | 200 | 0.30-0.45 | 0.595~0.612 |
| 10 | 100 | 0.3 | 0.595~0.612 | 35 | 100 | 0.3 | 0.595~0.612 |
| 20 | 200 | 0.30-0.40 | 0.595~0.612 | 40 | 200 | 0.25-0.45 | 0.595~0.612 |
| 20 | 100 | 0.3 | 0.595~0.612 | 40 | 100 | 0.3 | 0.595~0.612 |

## Testing Results

### *The Failure Mechanisms for Gravelly Soil with Different Plastic Fines Content*

The typical undrained cyclic triaxial testing results for the gravelly soil with *FC* = 10% and 35% are shown in Figures 2(a) and 2(b), respectively. The results indicate the failure mechanisms of the gravelly soil with *FC* = 10% and 35% are different. The failure mechanism of gravelly soil with *FC* = 10% is more gravel soil-like. Thus the initial liquefaction of the specimen occurs when a peak pore pressure in the cyclic fluctuation becomes momentarily equal to the initial confining pressure. This initial liquefaction phenomenon of the specimens can be considered to be cyclic liquefaction. In addition, the failure mechanism of gravelly soil with *FC*=35% is more clay-like. Thus, the permanent axial strain of the specimen can be continuously accumulated and the fluctuation accumulated axial strain was almost linearly. The accumulated axial strain of about 5% does not bring any state of instability involving intolerably large deformations. Therefore, any level of accumulated axial strain than 5% may be considered appropriate to define a state of failure for the gravelly soil with more *FC*. Thus, the failure phenomenon of the specimens can be considered to be an excessive accumulated axial strain, but the excess pore

pressure of the specimens was still much less than the initial effective confining pressure. For simplicity, the term "cyclic failure" is used to describe this type of failure state of the specimens. From this observed phenomena, there are two failure mechanisms for the gravelly soil with different *FC* are classified as cyclic liquefaction and cyclic failure.

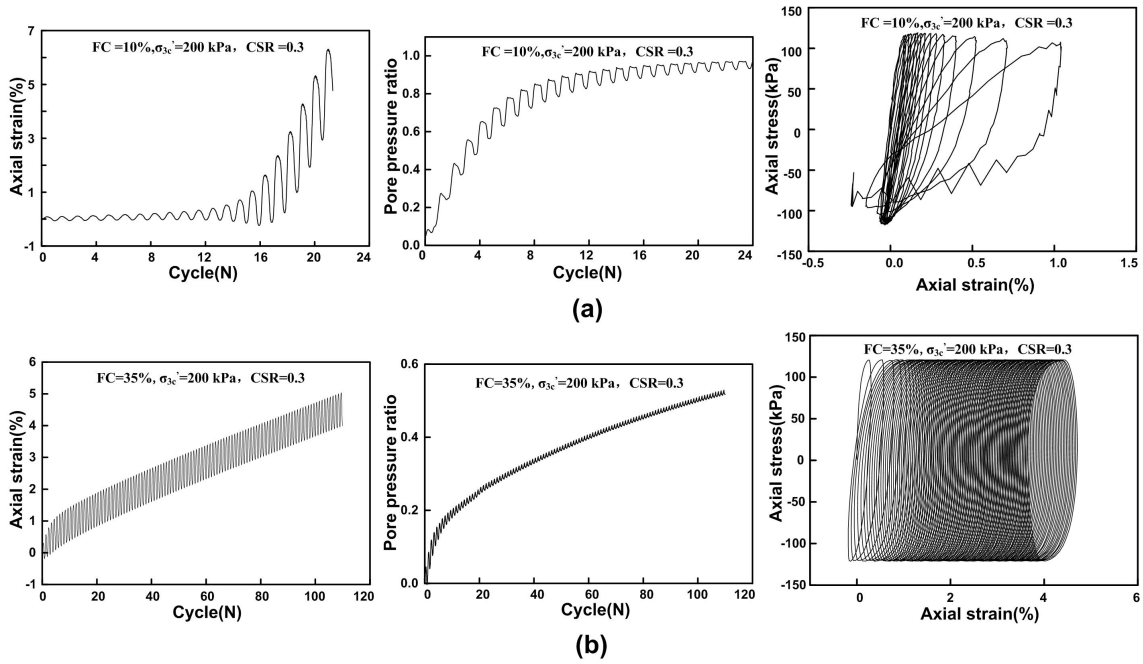


Figure 2. The typical undrained cyclic triaxial testing results for gravelly soil. (a) the specimen with *FC*=10%, $\sigma_{3c}'$=200 kPa, *CSR*=0.3; (b) the specimen with *FC*=35%, $\sigma_{3c}'$=200 kPa, *CSR*=0.3;

### *The Proposed Failure Criterion for Gravelly Soil with Different Plastic Fines Content*

There are now two different criteria for determining when sand liquefaction occurs. The first uses the standard of the pore pressure ratio of 100% as the threshold when the initial liquefaction occurs. The second uses the standard of 2-3% for single-amplitude or 5% for double-amplitude cyclic shear strain as the threshold when the liquefaction occurs. However, there is no consistent criterion for determining when gravelly soil with different plastic fines content liquefaction occurs. The difference in the criteria for the initial liquefaction of gravelly soil with different plastic fines content may be due to the specimen preparation and the testing method used. Therefore, using a suitable liquefaction criterion becomes critical for gravelly soil with different plastic fines content, and the cyclic resistance should be determined based on an appropriate criterion needs to be specified. In this study, the pore pressure ratio of 100% is adopted as the criterion for the initial liquefaction of the specimens with *FC* < 30%. However, an excessive accumulated axial strain of 5% of the specimens with *FC*⩾30% was observed when the excess pore pressure ratio was still less than 0.6. Thus, the accumulated axial strain of 5% was adopted as the criterion for the failure state of the specimens in this study.

### *Effect of Plastic Fines on Cyclic Resistance Ratio*

To determine the cyclic shear strength or cyclic resistance ratio (*CRR*) of specimens with desired *FC* corresponding to the concerned approach, a series of undrained cyclic triaxial tests were carried out at different *CSR* till the 100% pore pressure ratio or 5% accumulated axial strain was reached as discussed earlier. Thereafter these *CSR* were plotted against the corresponding cycles to 100% pore pressure ratio or 5% accumulated axial strain as shown in Figure 3, *CRR* was determined as the *CSR* corresponding to accumulated axial strain reach to 5% or the pore pressure ratio reach to 1 at 20 cycles.

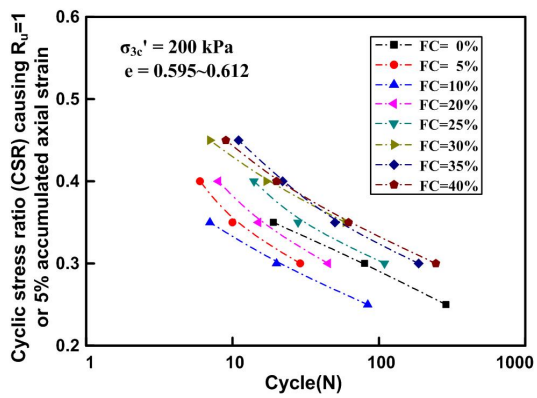


Figure 3. Influence of the plastic fines on the cyclic stress ratio required for a 100% excess pore pressure ratio or 5% accumulated axial strain of the specimens

Figure 4 shows the influence of *FC* on the *CRR* of specimens. The result show that the *CRR* of specimens can be divided into three groups based on *FC*, i.e., $FC \leqslant 10\%$, $10\% < FC < 30\%$, and $FC \geqslant 30\%$. In the first group, the *CRR* of specimens decreases with the increase of *FC*. In the second group, the *CRR* increases as *FC* increase, and increased significantly when $FC > 20\%$. The *CRR* of specimens with $FC = 25\%$ is larger than that of pure gravel soil ($FC = 0\%$). In the last group, the *CRR* value is around 0.4 and keeps almost unchanged with the increase of *FC*.

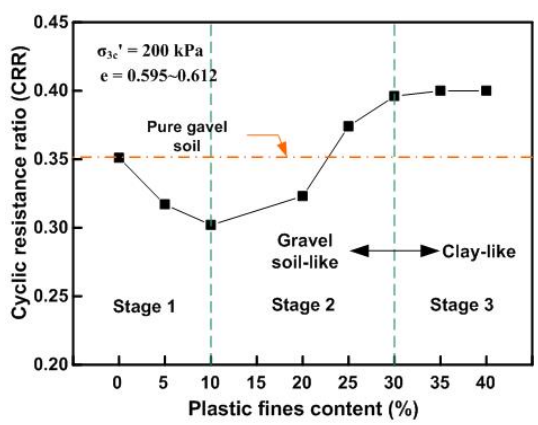


Figure 4. Relationship between cyclic resistance ratio and plastic fines content

### *Effect of Plastic Fines on Pore Pressure Generation*

In order to study the effect of plastic fines on pore pressure generation of gravelly soil at different *CSR* and $\sigma_{3c}'$, a series of undrained cyclic triaxial tests were carried out at *CSR* = 0.25 and $\sigma_{3c}'$ = 100 kPa. Figure 5 illustrates the relationship between pore pressure generation of gravelly soil with different *FC* and number of loading cycles for a constant void ratio at different *CSR* and $\sigma_{3c}'$. It is clearly seen from this figure that pore pressure ratio ($R_u$) increases with the increase in percentage of plastic fines (*FC* < 10%). With further increase in plastic fines (*FC* > 10%) $R_u$ decreased. In addition, number of cycles for initial liquefaction decreases with increase in the percentage of plastic fines content (*FC* < 10%). However, with further increase in the plastic fines content (10% < *FC* < 30%) the number of cycles for initial liquefaction has been found to increase. When *FC* equal to 30%, initial liquefaction don't appear and $R_u$ didn't reach 1 under 200 cycles, and thus initial liquefaction didn't occur. This clearly indicates that *FC* = 30% is a critical point for the change in the matrix of the gravelly soil with different *FC* from gravelly controlled matrix to plastic fines controlled matrix. As plastic fines dominate it prevent the building up of pore pressure, thus influencing the potential for liquefaction.

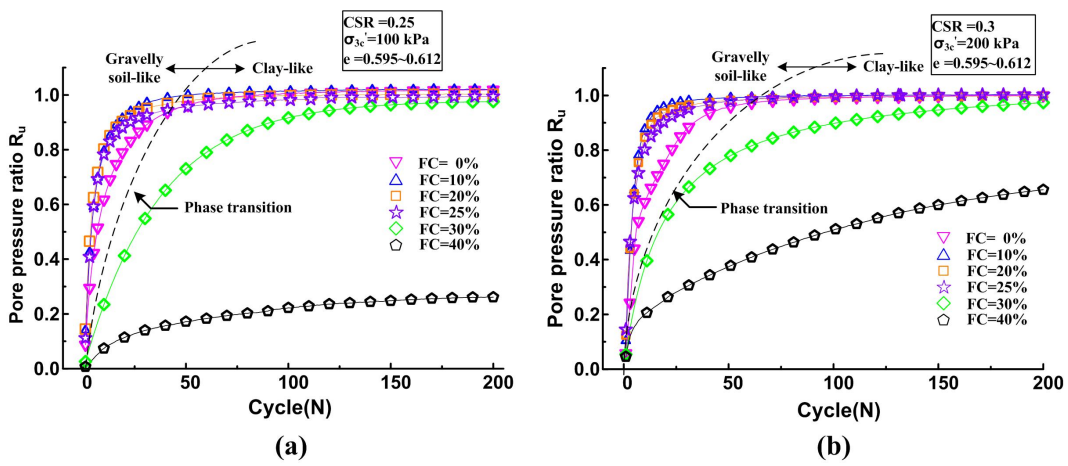


Figure 5. Relationship between pore pressure ratio $R_u$ and cycles of loading

### *Contact State Theory*

In order to explain the effect of fines contents on the mixture dynamic characteristics, Thevanayagam and martin (2001) and Thevanayagam et al. (2002) proposed a conceptual framework in which the soil mixture is assumed to be composed of spherical particles having two different diameter values, coarse grains and fine grains, and the contact state of particles determines the dynamic properties of the mixtures. According to this theory, the different of *FC* will change the contact state between plastic fines grain and gravel soil grain in gravelly soil. As Figure 6 shown, the contact states between plastic fines and gravel soil ingravelly soil may exist three cases.

Case 1: *FC*⩽10%, gravel soil grains are in contact directly, plastic fines grains fill the inner void spaces between the gravel soils grain, the liquefaction characteristics of gravelly soil depend on the fabric of gravel soil. The effect of plastic fines can be ignored. The increase of *FC* will only reduce the contact area of gravel soil grains which make the collapse-type failure occur easier resulting in *CRR* decreasing and pore pressure ratio increasing easily.

Case 2: 10% < *FC* < 30%, the part of plastic fine grains participate in contact of gravel soil grains and increase the contact area between the particles, but the gravel soils grains dominant the contact state, the liquefaction characteristics of mixtures depend on the fabric of gravel soils mostly. As *FC* increasing, more energy is needed to break the contact between the particles, resulting in the increase of *CRR* and pore pressure ratio increasing slowly.

Case 3: *FC*⩾30% plastic fine grains are primarily in contact, the plastic fine grains begin to play a rather important role while the role of gravel soil grains begin to diminish, neglecting the effects of dispersed gravel soil grains, the liquefaction characteristics of mixture change to clay-like, the mixture will failure as the contact of plastic fine grains break. Thus, *CRR* will not increase with *FC* increase, and pore pressure ratio can't reach 1.

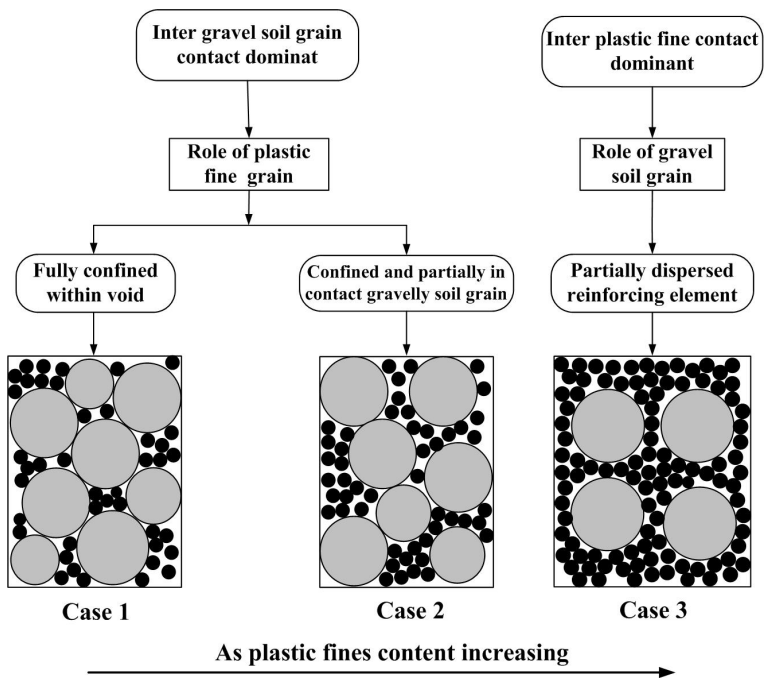


Figure 6. The contact station of particles in gravelly soil

Therefore, *FC* = 30% should be a point of the phase transition from gravel soils-like to clay-like for the gravelly soil tested in this study.

## Conclusions

A series of undrained cyclic triaxial tests were performed on the gravelly soil with 8 different plastic fines contents in the range of 0% to 40%. The testing results and findings are summarized as following:

(1) The failure mechanisms for the gravelly soil with different *FC* can be classified as cyclic liquefaction and cyclic failure. The pore pressure ratio of 100% and axial strain of 5% are adopted as the criterion for the initial liquefaction and cyclic failure state of the specimens in this study.

(2) Liquefaction resistance of the gravelly soil decrease till *FC* reach 10%, then increase with the increasing *FC*. Moreover, liquefaction resistance maintain constant when *FC* is more than 30%. In addition, the results of pore pressure generation show that *FC* = 30% is a phase transition point transform the liquefaction characteristics of the gravelly soil from gravel soils-like to clay-like.

(3) The contact state theory was proposed to explain the effect of plastic fines on the gravelly soil liquefaction characteristics. The contact states between plastic fines and gravel soil in the mixtures can be divided into three cases, which result in the mixtures with different FC be provided for different liquefaction characteristics.

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