EFFECT OF GRAVEL CONTENT ON THE UNDRAINED CYCLIC BEHAVIOR OF GRAVELLY SANDS IN TORSIONAL SHEAR TESTS

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ABSTRACT:
The liquefaction of sands has been widely studied in the past. However, several cases of gravelly soils liquefying have been observed in recent case studies. Due to their large particle sizes, element testing has been limited. Furthermore, membrane penetration (MP) must also be considered in element tests. MP may be defined as the intrusion/extrusion of the membrane as confining pressure is increased/decreased, respectively. In this study, MP was examined by comparing elimination and correction methods. The effect of gravel content on the cyclic behavior of gravelly sands in torsional shear tests was also investigated. It was found that that MP had a large effect on the undrained behavior of gravelly soils. The cyclic resistance of sand with 30% gravel content (GC) only increased slightly when GC was increased to 50%. Furthermore, strain localization occurred at lower strain levels when the GC was increased.
Key Words: *Liquefaction, Gravelly Soil, Torsional Shear, Large Strain*

INTRODUCTION

Historically, soil liquefaction has generally been considered to only occur in sandy soils. Since the 1964 earthquakes in Niigata and Alaska, much of liquefaction research was focused mainly on sandy soils. In contrast, gravelly soils were considered as non-liquefiable. This was because of their inherently large particle size and high permeability (Cao et al., 2011). As a result, important structures such as nuclear power plants are built on gravelly soil deposits (Konno et al., 1993). However, several gravelly soil liquefaction case studies have been reported during past earthquakes.

A literature review of past gravelly soil liquefaction events is presented in Table 1. The earliest mention of gravelly soils liquefying may be traced to as early as 1891. After this, several other cases were reported around the world. Cases were observed in both man-made and natural structures. Furthermore, most of the man-made structures are reclaimed port areas or embankments. It was also found that gravelly soils could liquefy in both flat and sloping ground. More recently, the world's largest naturally deposited gravely soil liquefaction case was observed (Yuan et al., 2019). In the last decade alone, three cases were observed in port areas (e.g. Cubrinovski et al., 2017). These cases may be the precursor to even more port liquefaction cases. Gravelly soils are extensively used in these kinds of developments and are expected to be used more in the future.

In element tests, the effect of membrane penetration (MP), which hinders pore water pressure generation must be considered. There are generally two approaches to deal with MP in undrained cyclic loading tests. The first is to apply corrections to the test result. In this approach, the effect of MP on the measured effective stress is corrected after the test (Tokimatsu & Nakamura, 1987). The second approach is to eliminate the MP during the test. Common methods include placing fine sand around the specimen

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(Miura & Kawamura, 1996; Toyota & Takada, 2019), sluicing, or by using an water injection system (Nicholson et al., 1993b; Sivathayalan & Vaid, 1998; Tokimatsu & Nakamura, 1986). In this study, MP was eliminated using fine sand placed around the specimen. The results are then compared with available correction methods. It is important to note that, most of the previous studies are based on triaxial tests on uniform soils. As far as the authors know, there have been no study on the use of MP-reducing layer on the hollow cylindrical torsional shear apparatus.

Table 1: Case histories of gravelly soil liquefaction

Earthquake	Year	M_W	Reference
Mino-Owari, Japan	1891	7.9	(Kishida, 1969)
San Francisco, USA	1906	8.3	(O'Rourke & Hamada, 1992)
Kanto, Japan	1923	7.9	(Hamada & O'Rourke, 1992)
Fukui, Japan	1948	7.1	(Ishihara, 1985)
Alaska, USA	1964	8.4	(Coulter & Migliaccio, 1966)
Haicheng, China	1975	7.3	(Wang, 1984)
Friuli, Italy	1976	6.5	(Sirovich, 1996)
Tangshan, China	1976	7.8	(Wang, 1984)
Miyagiken-Oki, Japan	1978	7.4	(Tokimatsu & Yoshimi, 1983)
Borah Peak, Idaho, USA	1983	7.3	(Andrus, 1994)
Nalband, Armenia	1988	6.8	(Yegian et al., 1994)
Roermond, Netherlands	1992	5.8	(Maurenbrecher et al., 1995)
Hokkaido, Japan	1993	7.8	(Kokusho et al., 1995)
Kobe, Japan	1995	6.9	(Kokusho & Yoshida, 1997)
Chi-Chi, Taiwan	1999	7.7	(Lin & Chang, 2002)
Wenchuan, China	2008	7.9	(Cao et al., 2011)
Ibaraki, Japan	2011	9.0	(Towhata et al., 2014)
Cephalonia, Greece	2014	6.1	(Nikolaou et al., 2014)
Muisne, Ecuador	2016	7.8	(Lopez et al., 2018)
Kaikoura, New Zealand	2016	7.8	(Cubrinovski et al., 2017)
Sulawesi, Indonesia	2018	7.5	(Okamura et al., 2020)

TEST APPARATUS, MATERIAL, AND PROCEDURE

A modified strain controlled hollow cylindrical apparatus in Fig. 1 was used for the experiments in this study Fig. 1. Modifications were made to the original apparatus to investigate large strain behavior of soils. The details of the original apparatus and modifications have been discussed in previous studies (Kiyota et al., 2008; Koseki et al., 2005). Further modifications on the specimen size were made to accommodate testing of larger particle sizes. An outer diameter of 200mm, inner diameter of 120mm, and height of 300mm were used in this study.

Membrane force correction must be considered for torsional shear tests (Koseki et al., 2005). To calculate the actual stress applied on soils, the shear stress measured by the load cell must be corrected for the apparent shear stress induced by the membrane. This correction is significantly more important at larger shear strains as point out by previous studies (Chiaro et al., 2015; Kiyota et al., 2008). Due to the modification in specimen dimensions, a confirmatory water specimen test was conducted to verify the applicability of the previous corrections. The test was performed by filling both the cell and specimen with water and shearing under the undrained condition. The initial and sheared state of the water specimen are shown in Figs. 2a. and 2b, respectively. Fig. 2c shows the comparison of the current and previous studies (Chiaro et al., 2015; Umar et al., 2021). The results show that the previously proposed hyperbolic correction is still valid for this study. This means that the membrane force correction is valid for different specimen size and height/diameter ratios.

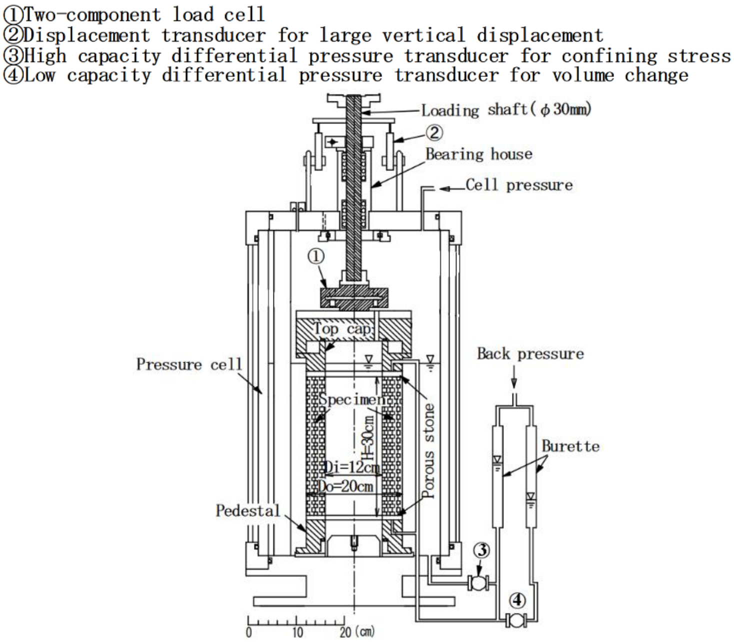

Fig. 1. Schematic diagram of apparatus (modified from De Silva, 2008)

The material tested in this study was a mixture of silica sand #7 and gravel. Gravel size was restricted to less than 6.4mm due to sizing requirements for torsional shear tests (JGS, 2009b). Gravel content (GC) is defined as mass ratio between gravel and total dry weight of specimen. For this study, GC=30% and 50% were used. The grain size distribution curves of all the materials are shown in Fig. **3**. Material properties of the soils are shown in Table 2. Maximum and minimum densities were determined based on (JGS, 2009a). Two test cases were conducted in this study to compare the effect of the sand layer at the specimen surface to the undrained cyclic behavior of gravelly sands.

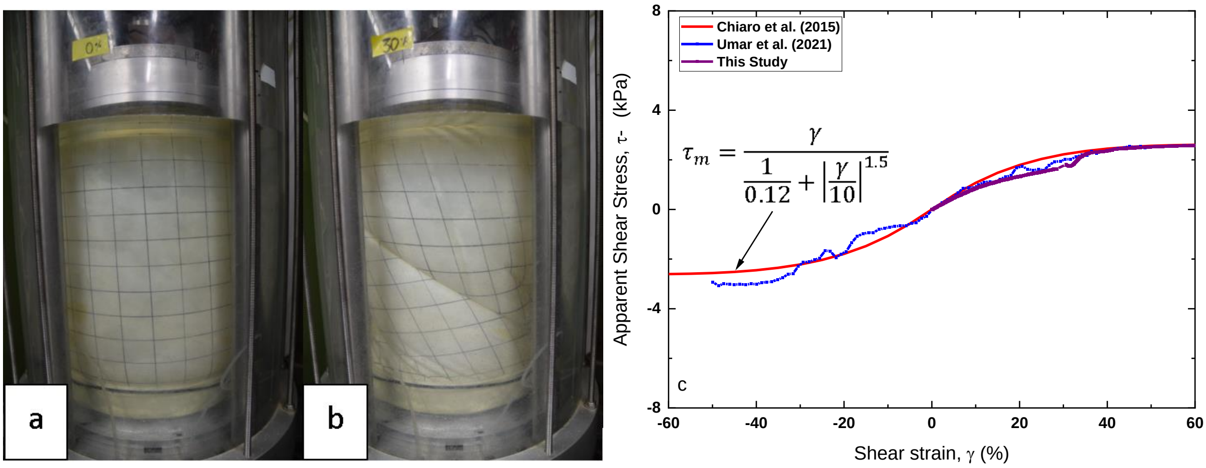

Fig. 2. a. Water specimen before shearing b. Water specimen at γ_{SA}=30% c. Comparison between the measured and calculated membrane force.

Dry tamping in 10 layers was used to form the specimens. A relative density (D_r) of 50% was maintained for the tests. Inner and outer split type molds were used to set up the specimen. For the specimen with sand layer, a triple pane mold was used to separate the core material and the outer and inner sand layers (Fig. 4). The sand layer's thickness used in this study was set to 4mm. The outer and inner sand layers was filled with silica sand #7 with equal relative density to that of the core material.

Table 2. Material properties of tested soils

	Sand	Gravel	GC=30%	GC=50%
Specific Gravity, G_s	2.64	2.75	2.68	2.7
Max density, ρ_{max} (g/cm^3)	1.589	1.853	1.772	1.991
Min density, ρ_{min} (g/cm^3)	1.264	1.685	1.513	1.718
Max void ratio, e_{max}	1.092	0.632	0.768	0.570
Min void ratio, e_{min}	0.664	0.484	0.510	0.355

After completing each layer of soil, the mold was carefully removed and a vacuum of -30 kPa was applied to the specimen. The cell was then placed and filled with water to an equal height of the specimen. The specimen was then double vacuumed for 12 hours and saturated while maintaining a -30 kPa pressure difference between the specimen and the cell. A back pressure of 200 kPa was applied and a B value not smaller than 0.98 was achieved in all the tests. The specimens were isotropically consolidated to an effective stress of 100 kPa.

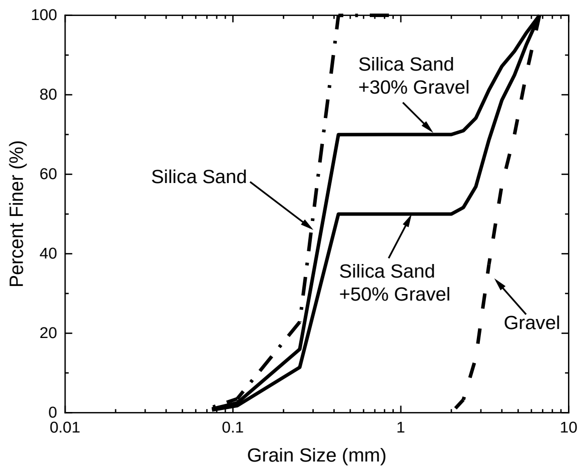

Fig. 3. Grain size distributions

Fig. 4. Specimen preparation to prevent MP

Undrained cyclic torsional tests with constant shear stress amplitude of 20kPa which is equivalent to a cyclic stress ratio (CSR) of 0.2 were then performed at shear strain rate of 3%/min. The number of cycles to achieve γ_{DA}=7.5% and excess pore water pressure ratio=0.95 were used to define liquefaction resistance for this study. The results of tests with sand layer and no sand layer at the specimen surface were compared and MP correction was applied to the test with no sand layer to verify if the use of the sand layer gives reasonable results. After which, the effect of gravel content was examined.

Table 3. List of tests performed

Test	Material used	Sand Layer	Cyclic stress ratio, $CSR = \tau_{cyclic}/p_0$'
TEST No. 1	Silica sand +30% GC	No	± 0.2
TEST No. 2	Silica sand +30% GC	Yes	± 0.15
TEST No. 3	Silica sand +30% GC	Yes	± 0.2
TEST No. 4	Silica sand +30% GC	Yes	± 0.3
TEST No. 5	Silica sand +50% GC	Yes	± 0.15
TEST No. 6	Silica sand +50% GC	Yes	± 0.2
TEST No. 7	Silica sand +50% GC	Yes	± 0.3

TEST RESULTS AND DISCUSSIONS

Comparison of membrane penetration elimination and membrane penetration correction

The first part of the experiment was done to confirm the effectivity of the MP elimination layer in undrained cyclic torsional shear tests. Existing MP corrections methods were used to correct the data and compare to the results of that MP elimination method. Following the procedure presented in Tokimatsu & Nakamura (1987), the compliance ratio (Crm) was first determined by Eq. (1). The values of rebound factor (C) and normalized membrane penetration (S) are obtained graphically (Tokimatsu & Nakamura, 1987). Tokimatsu (1987) proposed to use D_{50}, while Nicholson (1993a) proposed D_{20} to represent the normalized membrane penetration. Nicholson argued that D_{20} was a better representation for non-uniform soils. Tokimatsu based his approach on uniform soils. A_m/V is the surface area to volume ratio of the specimen. The cyclic ratio (Cn) is also obtained graphically from the Crm. The number of cycles without membrane penetration (No) is obtained by diving Nc by Cn.

$$Crm = \frac{S}{C}\left(\frac{A_m}{V}\right) \tag{1}$$

Table 4 and Table 5 show the coefficients obtained using the existing corrections. Regardless of liquefaction criteria, there is significant difference in the number of cycles between the case with the sand layer and that of without the sand layer. The correction using D_{20} is quite near to that of the results from the test with sand layer. This confirms that especially for non-uniform soils, D_{20} is a good indicator of MP characteristics. Based on these results, subsequent tests were conducted with the sand layer.

Table 4. Correction Coefficients (Tokimatsu & Nakamura, 1987)

	D_{50}	C	S	Crm	Cn	Nc	No	No’
γ_{DA}=7.5%	0.36	0.003	0.002	0.309	1.673	29	17	4.5
EPWPR=0.95						22.5	13.5	3.5

Table 5. Correction Coefficients (Nicholson et al., 1993a)

	D_{20}	C	S	Crm	Cn	Nc	No	No’
γ_{DA}=7.5%	0.263	0.003	0.004	0.673	3.181	29	9	4.5
EPWPR=0.95						22.5	7	3.5

Effect of gravel content on the undrained cyclic behaviour of gravelly soils

Two series of tests were conducted for this series. 30% and 50% GC specimens were tested with the sand layer to eliminate the effect of MP. Fig. 6 shows the effective stress paths of the different GCs. Fig. 5b shows a slower reduction in effective stress. This may be due to the higher permeability of the 50% GC specimen. Furthermore, Fig. 7c and 7e show the stress strain of the different GCs. The strain development was slower in the case of 50% GC. This could be due to the higher engagement of the gravel particles in the soil skeleton.

Fig. 5. Effective stress paths for a) 30% GC and b) 50% GC

To observe the overall trend of the two GCs, additional tests were done applying 15 kPa and 30 kPa to complete the liquefaction curve. Fig. 6 shows the summary of cyclic resistance curves of the different GCs. There was a slight increase in the number of cycles as GC was increased from 30% to 50%. It is interesting to note that the typical liquefaction curves only capture the behaviour up to a γ_{DA}=7.5%. Beyond this region, there is a significant difference in the shear strain accumulation.

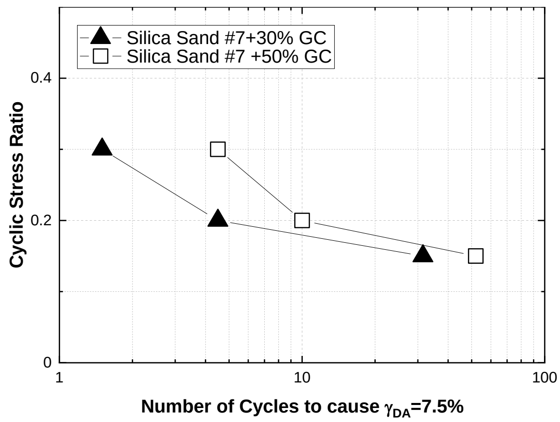

Fig. 6. Cyclic stress ratio vs Number of cycles

Effect of Gravel content on the limiting shear strain.

Shear strain localization is an important sign of failure in the specimen but is often difficult to evaluate purely on visual observation. The limiting shear strain can be defined as the state which the increment of single amplitude shear strain increases (Kiyota et al., 2008). The previous study showed a good correlation between the limiting shear strain to the maximum shear deformation observed in previous case studies and shaking table tests (Kiyota et al., 2013). Fig. 7 shows the stress-strain curves and zoomed versions to show localization. It was found that the MP elimination layer did not affect the limiting shear strain (i.e. 16%). While the increasing the gravel content reduced the limiting shear strain to about γ_{SA} =12%.

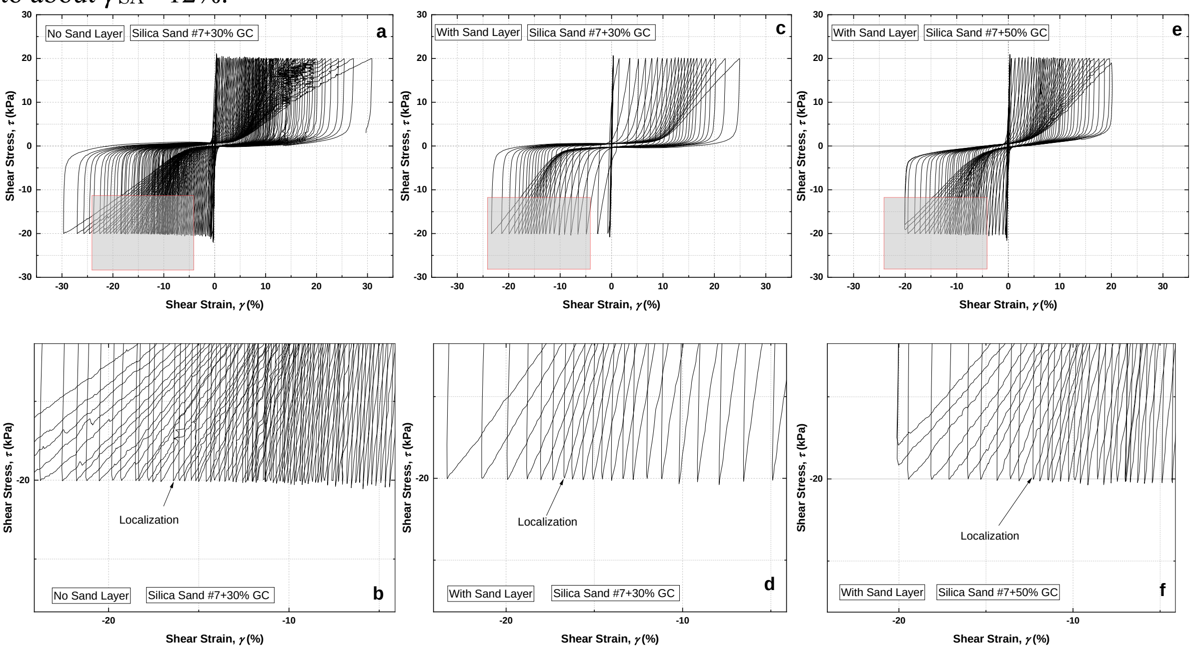

Fig. 7. Stress strain and localization trends (a&b), no sand layer-30% GC (c&d), with sand layer-30% GC, (e&f) with sand layer-50% GC

CONCLUSIONS

This study investigated several factors affecting the undrained cyclic characteristics of gravelly sands. The effect of MP to the undrained cyclic behavior of gravelly sands in torsional shear tests was found to be large when not eliminated or corrected. A sand layer was used in the peripherals of the specimen to eliminate the MP in torsional shear tests. In order to correct the data for MP, calibrations were done using D_{50} and D_{20}. Although D_{50} has a good correlation with MP for uniform soils, the use of D_{20} correction in non-uniform soils resulted in reasonable estimations compared to that of the sand layer method. Increasing GC had a positive effect on the liquefaction resistance of the specimen. it was found that increasing the GC from 30% to 50% resulted in a slight increase is the number of cycles to cause liquefaction. In the larger strain regions, localization may be observed. This can be regarded as the point of failure of the specimen. The results showed that the limiting shear strain was unaffected by the sand layer but decreased with the addition of GC. Additional tests will be carried out in order to better understand the undrained cyclic behavior of gravelly sands.

ACKNOWLEDGEMENT

This work was supported by JSPS KAKENHI Grant Number JP 20H02240.

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