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GRANULE SHAPE EFFECT ON THE SHEAR MODULUS AND DAMPING RATIO OF MIXED GRAVEL AND CLAY*

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Abstract– Gravel-Clay mixtures are abundant materials in nature and are frequently used in certain civil engineering projects such as earth dams, levees and landfills. The advantage of using these soils lies in their low permeability owing to clay fraction along with the high shear strength due to the presence of the non-cohesive granular part. Karkheh dam is an example where these soils were used as the impervious core of the embankment. To date, little research has been carried out to investigate the performance of these soils, and therefore, their behavior under cyclic loading is still not well known. In order to investigate the cyclic behavior of gravel-clay mixtures, 51 cyclic and monotonic triaxial tests were performed on specimens with 11 different mixtures and under various confining pressures. Two different types of gravel, i.e. angular and round grains, were utilized to prepare specimens with the same gravel content in order to investigate the effect of the granule shape on the cyclic behavior of the mixtures. All the specimens were prepared with a constant compaction effort. Normalized shear modulus and damping ratio data showed differences in the behavior of mixtures with different kinds of gravel. Based on samples of macro structure performance under cyclic loading, two different mechanisms for the behavior of angular and round granules at contact level were hypothesized. The importance of the sampling method and specimen size for intermediate soils was also noticed.

Keywords– Cyclic loading, damping ratio, granule shape, granule contact, gravel-clay mixture, hyperbolic model, micromechanics, shear modulus

1. INTRODUCTION

Gravel-Clay mixtures are abundant materials in nature and are frequently used in certain civil engineering projects such as earth dams, levees and landfills. The advantage of using these soils lies in their low permeability owing to clay fraction and high shear strength due their non-cohesive granular part. Karkheh dam, located in the southwest of Iran is one the largest earthdams in the world. Its impervious core was made from mixed clay and gravel soils. The response of these soils to cyclic and dynamic loads is not yet well known. In this paper, the results of 51 monotonic and cyclic triaxial tests on mixtures of clay and gravel with different gravel contents are presented. All test specimens were prepared with a constant compaction effort, i.e. standard Proctor test energy (595 kN·m/m^3). Two types of gravel were selected for the tests, a complete round shaped gravel and an angular shape. A threshold for the gravel content was found at which it starts to play a major role in the behavior of the mixture. For the mixtures with a gravel content higher than this threshold, research efforts was focused on the effect of the gravel particles shape

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on the shear modulus (G), and the damping ratio (D) variation with shear strain. Then, a mathematical model was presented for normalized shear modulus and damping ratio variation with shear strain, incorporating the gravel content and its shape as the input parameters. Based on the test results and the observations of macro structure behavior, two different mechanisms for the interaction of grains at contact level were hypothesized for angular and round gravel mixtures.

It is widely known that soils act nonlinearly during small shear strains as well as during dynamic and earthquake loads. Cyclic and dynamic soil response is not only needed for earthquake analysis but also for the design of machine foundations, wind power plant foundations, fast transportation systems, impacts and blast resistant structures, etc. While more sophisticated models are available today to predict soil behavior under cyclic loads [1,2], obviously more accurate input data on soil parameters are needed to feed these models. Two important dynamic characteristics of soils are the shear modulus (G) and damping ratio (D). At shear strains smaller than 10^{-6} (γ=0.0001%), G and D are essentially constant and at their maximum and minimum values, respectively. When shear strain increases above 10^{-5}, G and D begin to change nonlinearly. Seed and Idriss [3] obtained relationships for the variations of these two parameters with shear strain for sands for the first time. Subsequently, numerous works have been done to study the dynamic properties of sands [4], clays [5-6], gravels [4, 7], silty sand, silty clays, natural soil deposits and residual soils [8], and earthfill and rockfill materials [9-10]. Further extensive research has been performed on different soils from sand to clay and gravel [11]. Yet, little work has been performed to study thoroughly the variation of G and D with shear strain for mixed gravel and clay soils. The closest work is a research on the cyclic parameters of gravelly deposits conducted by Lin et al. [8], in which the deposits considered were not clayey. Therefore, this study not only helps geotechnical engineers to obtain a better knowledge and understanding of the behavior of mixed gravel-clay soils, but also contributes to the current database of geo-material cyclic behavior.

2. SHEAR MODULUS (G) AND DAMPING RATIO (D) OF SOILS

a) G-γ relationships

Shear modulus, G, is generally defined as the slope of the line connecting two extreme points on the hysteresis loop at a certain shear strain (Fig. 1). The stiffness of the soil degrades dramatically as the level of strain amplitude increases above 10^{-5}. Different kinds of soils behave differently but all follow an almost similar rule and their nonlinear behavior can be formulized. Hyperbolic models have been widely used to describe nonlinear soil behavior under cyclic loading [11, 14]. Generally, the cyclic nonlinear behavior of the soil is defined by a loading-unloading curve which is governed by Masing's rule [15].

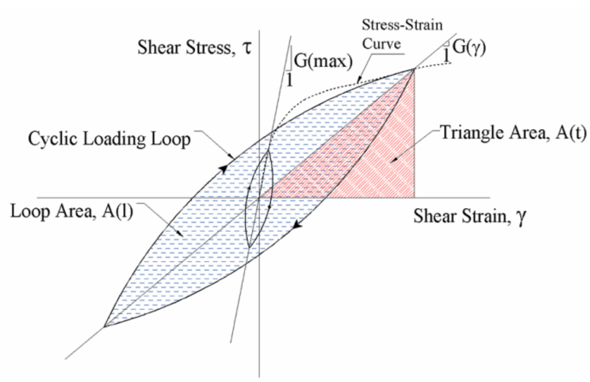

Fig. 1. Hysteretic shear stress-shear strain relationship

b) D-γ relationships

The damping ratio, D, is a measure of the dissipated energy (W_D) versus elastic strain energy (W_S)

through one cycle of loading-unloading and is defined [16] with the equation

$$D = \frac{W_D}{4\pi \cdot W_S} = \frac{1}{4\pi} \cdot \frac{A(l)}{A(t)} \tag{1}$$

where *A(l)* is the area enclosed by the hysteresis loop and *A(t)* is the area of the shaded triangular, as shown in Fig. 1. A common approach to model *D* is to relate the damping ratio with G/G_{max}. The advantage of this approach is that the better defined G/G_{max} is used to infer *D*. Several researchers have studied relationships between G/G_{max} and *D*. Hardin and Drnevich [14] assumed that *D* is proportional to ($1\text{-}G/G_{max}$). Others associated *D* with G/G_{max} using a polynomial function [17]. The general damping equation adopted by Darendeli [11] for his study had the following form: $D=f(G/G_{max})+D_{min}$.

Ishihara [19] carried out extensive investigations to determine the soil properties at different strain levels. He found that the hyperbolic model combined with the Masing's rule can only be used for medium levels of shear strains and up to 1%. For larger strain levels, the implementation of the Ramberg-Osgood model provided satisfactory results. However, there are few situations that soils can withstand cyclic shear strains above 1% and the clay-gravel mixtures do not belong to that group.

c) Parameters affecting G and D in soils

The most important parameters studied so far that affect shear modulus (*G*) variation include: shear strain amplitude, mean effective confining stress, soil type and its plasticity index (*PI*). Other effective parameters according to Darendeli [11] include: frequency of loading, number of loading cycles, overconsolidation ratio, void ratio, saturation degree, and grain characteristics. Investigators have shown that the important factors influencing damping ratio *(D)* include shear strain amplitude, mean effective confining stress, soil type and plasticity index, frequency of loading, and number of loading cycles. The effect of plasticity index change on *D* is complex; however, Stokoe et al. [20] found that values of D_{min} increase with increasing plasticity index, while values of damping ratio decrease at high shear strains with increasing plasticity index.

As clear from the above discussions, there is a wide area of research open to study the effect of granule shape on the shear modulus and damping ratio.

3. EXPERIMENTAL PROGRAM

a) Materials used in laboratory testing

The clay used in this study was obtained from the Khatoon Abad quarry, north of Tehran. This soil had been used in previous laboratory tests at IIEES (Int. Inst. of Earthquake Engrg. and Seismology) [13]. Two types of gravel were used in this study, a completely round shaped gravel taken from the Mahmoud-Abad coast of the Caspian Sea and a crushed angular gravel obtained from the Tellow asphalt factory located north east of Tehran (Fig. 2). The physical properties of these materials are given in Table 1 and their grading curves are presented in Fig. 3. The gravel was passed through a 3/8" sieve and the material retained on a #4 sieve was selected. Then flaky and elongated granules were picked out from the material by hand. The gravel thus obtained had a mean size of 7.1mm with an almost uniform granule shape. Standard compaction test following ASTM D698 was performed on the clay soil to obtain its maximum dry density and optimum moisture content. The results indicated that the maximum dry density was 16.9 kN/m^3 with an optimum water content of 16.8%. The internal friction angle of the same materials was reported by Shafiee et al. [21] and shown in Fig. 4.

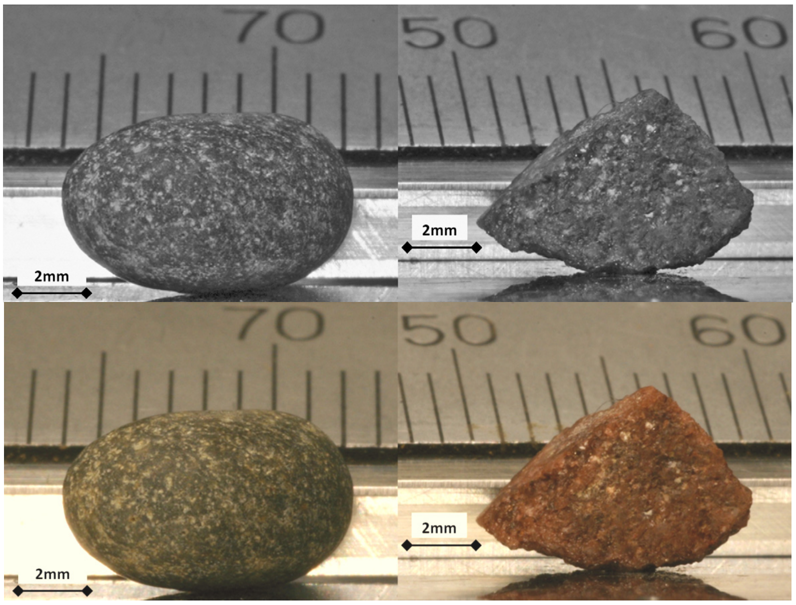

Fig. 2. Typical round (left) and angular (right) gravel grain used in the tests

Table 1. Physical properties of materials used in the current study

Material	G_s	PI %	LL %	Mean grain size (mm)	USCS symb.
Clay	2.72	8.4	27.4	-	CL
Round gravel	2.69	-	-	7.1	GP
Angular gravel	2.63	-	-	7.1	GP

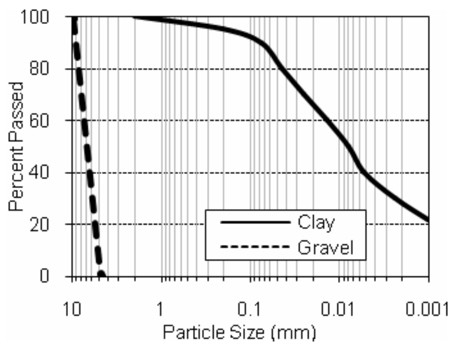

Fig. 3. Sieve analysis results for clay and gravel material used in the tests

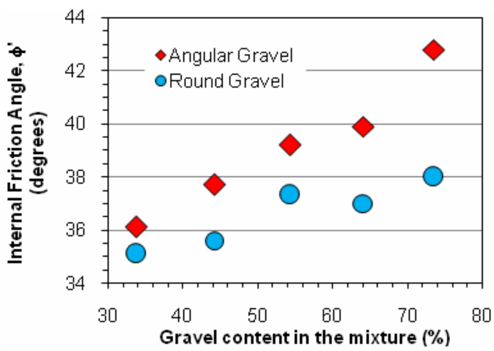

Fig. 4. Internal friction angle of mixtures at different gravel contents [21]

b) Tests setup and sample preparation

In order to study the effects of granule shape, mixture ratio, and confining pressure on *G* and *D* variation with shear strain, two series of triaxial tests were performed. First, thirty-three samples were prepared for medium ($5\times10^{-5}<\gamma<2\times10^{-3}$) to large ($10^{-3}<\gamma<\gamma_{failure}$) strain cyclic loading. In addition to three unmixed clayey samples, thirty mixed samples were prepared with gravel contents of 33.8%, 44.2%, 54.3%, 64.1%, and 73.5%. The gravel content was calculated by considering the dry weight of the samples. In these tests, as will be discussed later, samples with 33.8% and 44.2% of angular and round gravel content did not show any readable difference in the shear modulus values compared with each other and also with 100% clayey samples. Then, the behavior of the mixtures with a gravel content of 54.3% and more in a wider range of shear strain amplitudes was studied. It would also be possible to find out the effect of granule shape on the dynamic properties of these mixture ratios under a wide range of shear strains. To obtain the maximum shear modulus of the mixtures, it was necessary to measure axial deformation at very small strains ($10^{-6}<\gamma<10^{-5}$). Local Displacement Transducers (LDT) [22] were employed to catch the sample deformation at very small strains (Fig. 5). Eighteen small strain consolidated undrained (CU) triaxial shear tests were performed at three mixture ratios (i.e. 54.3%, 64.1% and 73.5%), three confining pressures of 100, 300, and 500kPa and with two kinds of gravel. Tables 2 and 3 list the specimen designation codes together with their clay and gravel contents as well as their initial dry density for cyclic triaxial and small strain CU tests samples, respectively.

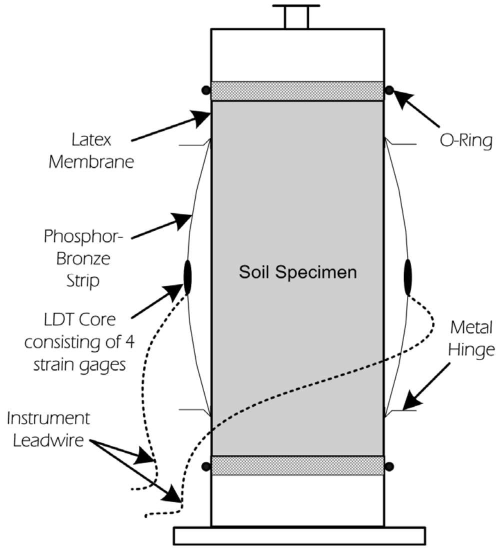

Fig. 5. Local displacement transducer (LDT) setup for triaxial testing

The method of sample preparation consisted of mixing the clay with a certain volume of water, adding a predetermined percentage of gravel and leaving them in a closed canister for at least 24 hours to stabilize. Then, the specimen was prepared by wet tamping the soil in a mold in 6 layers with undercompaction technique [23] so that a uniform density for all 6 layers was achieved. Table 4 lists density values of the 6 layers for a 100% clayey sample along with a number of hammer blows for each layer.

The samples were 140 mm high and 70mm in diameter for cyclic triaxial tests and 190 mm high and 50mm in diameter for small strain CU tests.

Since the clayey soils are often compacted 2% wet of optimum, especially in earthdams and landfills, the compaction moisture content for clay was chosen to fall between 18.8% and 19.2%. The increased moisture was needed to preserve the integrity of the specimens at 64.1% and 73.5% gravel contents when removing the mold and before putting the latex membrane on the specimens. Each sample was compacted with the same amount of energy following the standard Proctor method. As expected, the samples with different mixture ratios had different dry densities when compacted with the same effort (Tables 2, 3).

c) Cyclic triaxial tests

Strain controlled cyclic triaxial tests were carried out on 33 samples. The method of performing the cyclic triaxial test is described in ASTM D3999-91 in detail. The samples were brought to complete saturation (Skempton's B-value>0.97) before consolidation and were kept saturated during cyclic loading. In order to cover the earthquakes dominant frequency and most of the natural dynamic and cyclic loads, the highest possible cyclic loading frequency (f=0.1Hz) that ensured pore water pressure equalization through the samples during the cyclic loadings was selected. This rate would also reduce the creep effects and air diffusion through the membrane during cyclic loading phase. This rate of loading was considered as a compromise between the advantages and drawbacks of both rapid and slow cyclic rates.

Table 2. Specimens designations and samples characteristics for 33 cyclic triaxial tests

Test No.	Designation	Clay %	Gravel %	Gravel type	Conf pres. (kPa)	Initial γ_d (gr/cm^3)
c1	*c101*	100	-	-	100	1.64
c2	*c103*	100	-	-	300	1.71
c3	*c105*	100	-	-	500	1.71
c4	*r301*	66.2	33.8	round	100	1.88
c5	*r303*	66.2	33.8	round	300	1.87
c6	*r305*	66.2	33.8	round	500	1.85
c7	*a301*	66.2	33.8	angular	100	1.79
c8	*a303*	66.2	33.8	angular	300	1.79
c9	*a305*	66.2	33.8	angular	500	1.81
c10	*r401*	55.8	44.2	round	100	1.93
c11	*r403*	55.8	44.2	round	300	1.92
c12	*r405*	55.8	44.2	round	500	1.92
c13	*a401*	55.8	44.2	angular	100	1.79
c14	*a403*	55.8	44.2	angular	300	1.79
c15	*a405*	55.8	44.2	angular	500	1.86
c16	*r501*	45.7	54.3	round	100	2.03
c17	*r503*	45.7	54.3	round	300	1.96
c18	*r505*	45.7	54.3	round	500	1.97
c19	*a501*	45.7	54.3	angular	100	1.78
c20	*a503*	45.7	54.3	angular	300	1.78
c21	*a505*	45.7	54.3	angular	500	1.68
c22	*r601*	35.9	64.1	round	100	2.10
c23	*r603*	35.9	64.1	round	300	1.97
c24	*r605*	35.9	64.1	round	500	2.00
c25	*a601*	35.9	64.1	angular	100	1.77
c26	*a603*	35.9	64.1	angular	300	1.77
c27	*a605*	35.9	64.1	angular	500	1.76
c28	*r701*	26.5	73.5	round	100	2.13
c29	*r703*	26.5	73.5	round	300	2.03
c30	*r705*	26.5	73.5	round	500	2.11
c31	*a701*	26.5	73.5	angular	100	1.77
c32	*a703*	26.5	73.5	angular	300	1.77
c33	*a705*	26.5	73.5	angular	500	1.77

Table 3. Specimens designations and samples characteristics for 18 small strain triaxial tests

Test No.	Designation	Clay %	Gravel %	Gravel type	Conf. pres. (kPa)	Initial γ_d gr/cm^3
s1	*a511*	45.7	54.3	angular	100	1.79
s2	*a533*	45.7	54.3	angular	300	1.79
s3	*a555*	45.7	54.3	angular	500	1.79
s4	*r511*	45.7	54.3	round	100	1.98
s5	*r533*	45.7	54.3	round	300	1.98
s6	*r555*	45.7	54.3	round	500	1.98
s7	*a611*	35.9	64.1	angular	100	1.76
s8	*a633*	35.9	64.1	angular	300	1.76
s9	*a655*	35.9	64.1	angular	500	1.76
s10	*r611*	35.9	64.1	round	100	2.07
s11	*r633*	35.9	64.1	round	300	2.07
s12	*r655*	35.9	64.1	round	500	2.07
s13	*a711*	26.5	73.5	angular	100	1.78
s14	*a733*	26.5	73.5	angular	300	1.78
s15	*a755*	26.5	73.5	angular	500	1.78
s16	*r711*	26.5	73.5	round	100	2.11
s17	*r733*	26.5	73.5	round	300	2.11
s18	*r755*	26.5	73.5	round	500	2.11

d) Small strain CU tests

Local displacement transducers (LDT) were first introduced by Goto et al. [22] to be used in the triaxial testing of soils for the measurement of a wide range of strains from 10^{-5}% to 1%. The other advantage of using LDTs is that they measure the actual deformation of the sample, surpassing the bedding effect error, which is very important in measuring very small strains [22]. For the specimens with 54.3%, 64.1% and 73.5% gravel content, monotonic triaxial tests were conducted at three different confining pressures of 100, 300, and 500 kPa according to ASTM D4767-04. Sample preparation and saturation procedure were similar to the cyclic triaxial tests.

4. RESULTS AND DISCUSSION FOR THE SHEAR MODULUS (*G*)

a) Shear modulus versus shear strain

Shear modulus was determined from the slope of the line connecting the two extreme points on the hysteresis loop for a certain shear strain amplitude as shown in Fig. 1, at the 10th cycle of the loading as the representative value for all specimens. This is in accordance with the works of previous researchers who calculated G in the 5th to 10th cycle of loading [14, 24]. Figure 6 shows variation of G versus γ for the 33 specimens tested. Each graph shows the results for angular and round gravel for a certain gravel content together with the results of unmixed clay samples at the three different confining pressures adopted in this research program.

b) Small strain triaxial tests and maximum shear modulus

The maximum shear modulus of soils (G_{max}) occurs at very small shear strains as shown in Fig. 1 and can be obtained either from monotonic or cyclic tests [25]. The stress-strain relationship of the soil can be accurately modeled by a second order polynomial function in the elastic region and at very small strain range (γ<1~3×10^{-4}). It was observed from the cyclic triaxial test results that the difference in shear modulus between angular and round gravels is significant for the mixture ratios of 54.3% and more. In order to study the effect of granule shape on the cyclic behavior of these mixtures more thoroughly, G/G_{max}-γ curves were needed for mixture ratios of 54.3% and more. The stress-strain curve for samples *a511* is shown in Fig. 7. The range of axial strains covered in these tests varied from 10^{-6} to 10^{-4} and G_{max} was calculated at $\gamma=10^{-6}$ for all specimens, where $G=E/3$ and $\gamma=1.5\varepsilon_1$ in saturated conditions. E and ε are modulus of elasticity and axial strain, respectively. The source of scatter in the data (Fig. 7) was symmetrical electrical noise, which was cancelled out by curve fitting. The high coefficient of correlation proves this point.

c) Shear modulus versus shear strain results

-33.8% Gravel content: When the gravel content is 33.8%, the gravel particles float in the clay matrix and they are not in contact with each other, so the applied load is mainly carried by the clay part. Hence, shear modulus values are practically the same for angular and round gravels and they are also close to the results from unmixed clayey samples (Fig. 6).

-44.2% Gravel content: For the next group of samples with the gravel content of 44.2%, the results are again similar, though a little deviation appears between the results from specimens made with round gravel and angular gravel at a confining pressure of 300 kPa and higher (Fig. 6). This may be due to the better compaction of clay paste in the vicinity of round gravels when the granules slip on each other during the compaction process. Slightly higher local densities in the clayey phase connecting the round granules may be a reason for the higher shear moduli.

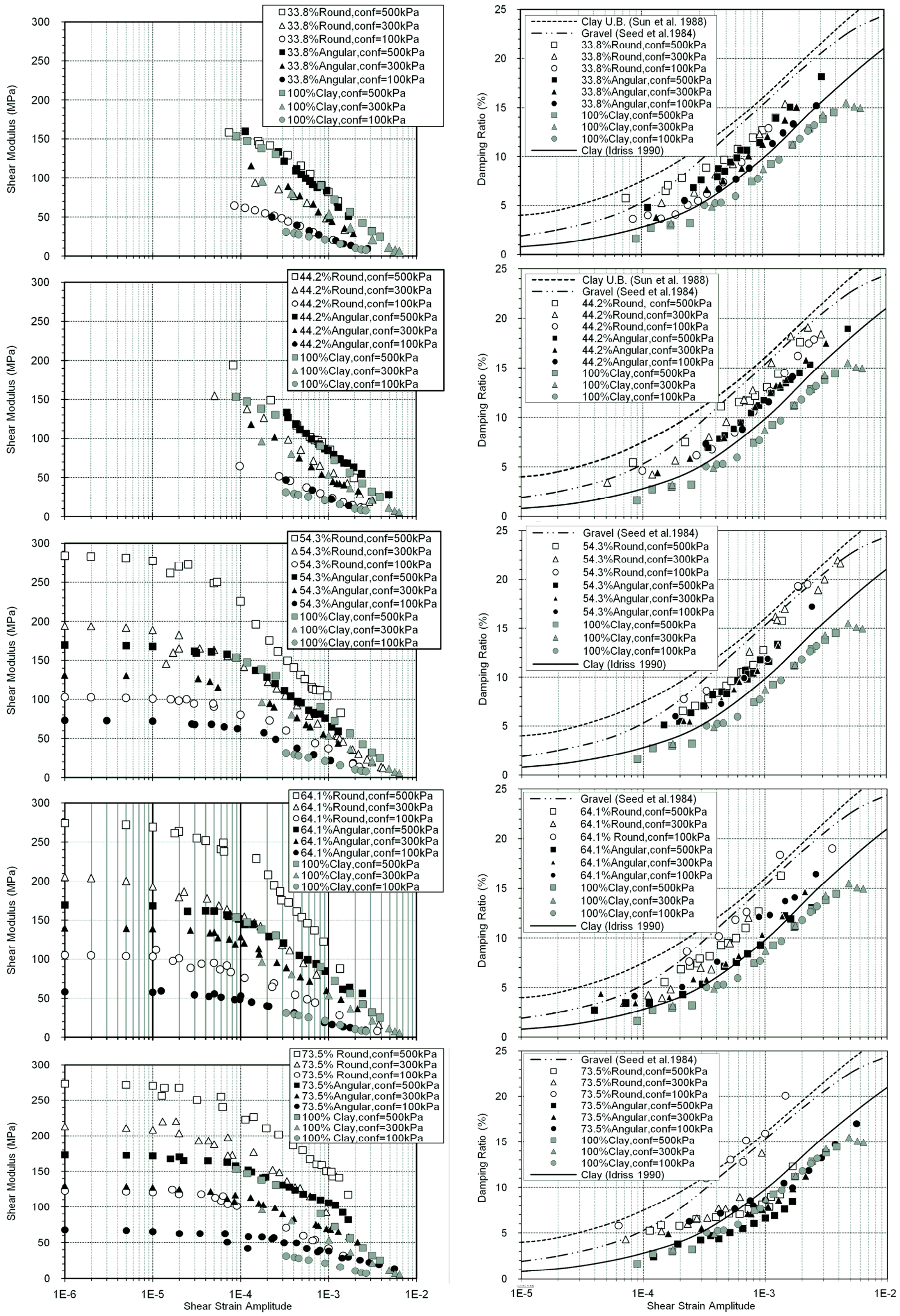

Fig. 6. Shear modulus (left) and damping ratio (right) versus shear strain for specimens with different gravel contents, compared with clay specimens tested at different confining pressures

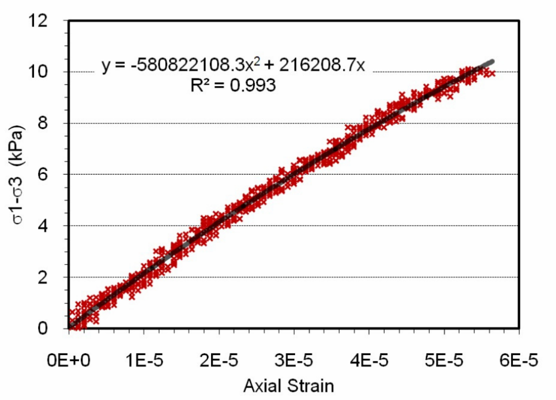

Fig. 7. Stress-strain curves for monotonic consolidated undrained test on specimen a511

-54.3% Gravel content: In the group of samples with 54.3% gravel content, the angular and round gravel samples had different shear moduli for all confining pressures but specimens with angular gravel content still acted closer to the pure clay samples in the range of data that was available for 100% clayey samples (Fig. 6). The specimens with angular gravel had a relatively lower shear modulus than the specimens with round gravel at the confining pressure of 100kPa. The difference between the obtained values became more readable as the confining pressure was increased. These observations may be described by considering the compression of clay between the round gravels during consolidation at high confining pressures, while the resistance of angular gravels from sliding on each other results in lower deformation under identical applied confining pressures. Even though the change of void ratio during consolidation was higher for samples with an angular mixture, it may be suggested that the clay between granules did not receive as much of the confining pressure to consolidate (Table 5). The void ratio of the clay part in Table 5 is calculated with reference to Shafiee [24] as shown below:

$$e_{clay} = \frac{e_{sample}}{\%(clay)} \tag{2}$$

where e_{clay} is the void ratio of the clay matrix, e_{sample} is the void ratio of the sample and *%(Clay)* is the clay content in the mixture in the decimal figure. This formula is only valid for the gravel contents of 54.3% and less since in the mixtures with higher gravel contents, the clay is not evenly distributed in the sample and some large voids are present in the specimens (Fig. 8).

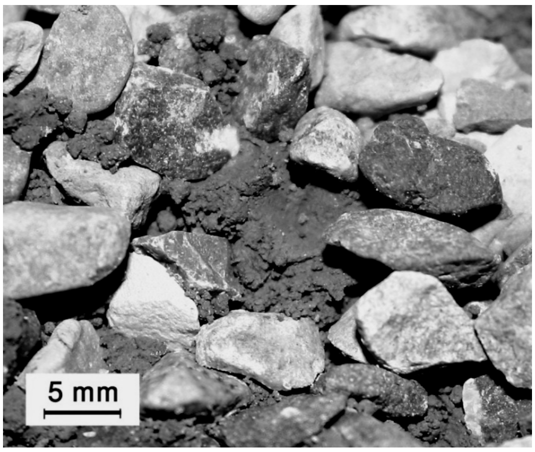

Fig. 8. Close up view of sample texture with 73.5% angular gravel before consolidation

From a qualitative point of view, the high deformation and volume change might be due to a combination of three reasons: 1-change in the matrix of angular granules; 2- change in the size of voids between granules and clay, which are filled with water in saturated condition; 3- change in the density of clay sitting distant from the granules. However, the extent of each mechanism was not measured in this study.

Table 4. Typical results and details of undercompaction method used for specimen preparation

Layer No.	Avg. thickness(mm)	Wet density gr/cm^3	Dry density gr/cm^3	Water Cont.(%)	Hammer drop* (mm)	Number of blows
1 (top)	23.45	1.99	1.68	18.80	120	78
2	23.55	1.99	1.67	18.89	143	66
3	23.50	1.99	1.67	18.91	166	57
4	23.40	2.00	1.68	18.84	190	50
5	23.20	2.02	1.69	18.93	93	105
6 (bottom)	23.10	2.02	1.70	18.80	117	86
Average	*23.37*	*2.00*	*1.68*	*18.80*		

*Hammer weight=0.543kgf

Table 5. Void ratio of the samples and their clayey part before and after consolidation

% Gravel in sample	Effc. conf. pres. (kPa)	Average initial void ratio				Avg. post consolidation void ratio				% Change (average)	
		round		angular		round		angular		round	angular
		whole	clay part	whole	clay part	whole	clay part	whole	clay part		
0	100	0.65				0.60					
	300	0.62				0.52				-9.1	
	500	0.58				0.56					
33.8	100	0.44	0.66	0.50	0.76	0.41	0.62	0.47	0.71		
	300	0.45	0.67	0.50	0.76	0.37	0.56	0.40	0.61	-15.3	-15.3
	500	0.46	0.69	0.48	0.73	0.35	0.53	0.38	0.53		
44.2	100	0.40	0.72	0.50	0.90	0.38	0.68	0.43	0.77		
	300	0.41	0.73	0.50	0.89	0.33	0.59	0.39	0.70	-15.5	-18.8
	500	0.41	0.73	0.44	0.79	0.32	0.57	0.35	0.63		
54.3	100	0.33	0.72	0.50	1.10	0.32	0.69	0.45	0.98		
	300	0.38	0.83	0.50	1.09	0.31	0.68	0.40	0.88	-14.0	-22.0
	500	0.37	0.81	0.59	1.29	0.30	0.65	0.37	0.81		
64.1*	100	0.28	-	0.51	-	0.25	-	0.43	-		
	300	0.37	-	0.51	-	0.30	-	0.39	-	-15.8	-19.6
	500	0.35	-	0.52	-	0.28	-	0.41	-		
73.5*	100	0.27	-	0.50	-	0.23	-	0.44	-		
	300	0.33	-	0.50	-	0.29	-	0.39	-	-15.6	-17.7
	500	0.28	-	0.50	-	0.22	-	0.40	-		

* Clay matrix void ratio is not calculated for these gravel contents for the reason given in the text

-64.1% Gravel content: For the samples with 64.1% gravel content, angular and unmixed clayey samples still had very close shear modulus values at medium to large shear strains. The round gravel samples showed noticeably higher shear moduli (Fig. 6).

-73.5% Gravel content: Finally, for samples with 73.5% gravel content, the angular gravel mixtures still have close shear modulus values to the unmixed clayey samples in the range of data available for pure clay samples. This is seen for all confining pressures considered in this study (Fig. 6). For this mixture ratio, the round gravel mixtures have distinctly higher shear modulus values compared with angular gravel and pure clay samples at shear strains smaller than 2×10^{-3}. At this mixture content, almost all gravel particles are in direct contact with each other, however, the clay is still filling some part of the voids between granules (Fig. 8). So the macro behavior of the sample under cyclic loading is more influenced by what happens at the granule contacts and less by the intergranular clay in the mixtures.

Based on the results of shear modulus variation shown in Fig. 6, a limit for gravel content can be defined below in which the granule content in the sample and its shape has no effect on the resulting shear modulus. After that limit, the dominant effect of the clay in the mixture decreases and the sample behavior is controlled by both of the clay and gravel parts. According to the results, this limit can be defined between 44.2% and 54.3%. It is obvious from Fig. 6 that in the samples that were composed of 54.3% gravel and more, round gravel mixtures resulted in higher shear modulus values compared with the angular gravel mixtures with the same percentage of gravel content. This may be partly due to the slippage of the round granules during compaction and consolidation, resulting in a more compact structure and partly due to the brittle lattice structure formed by angular gravels in the mixtures with dominant gravel content. This latter resists the compaction and consolidation and therefore, the samples with the same gravel content but of angular type achieve lower dry densities, and subsequently, lower shear modulus values. However, it was suggested that the initial dry density of the specimen might not be the only reason for this difference. Normalized shear modulus curves showed different behavior between round and angular gravel mixtures. This will be discussed in the next sections.

d) Mathematical model for maximum shear modulus (G_{max})

The maximum shear modulus of soils as a function of the corresponding effective confining pressure generally obeys a power rule with a function of void ratio as a multiplier. In order to describe a relationship among G_{max}, initial dry density and the effective confining pressure, the following equation is proposed:

$$\frac{G_{\max}}{p_0} = b \cdot \left(\frac{\gamma_d}{\gamma_w}\right) \cdot \left(\frac{\sigma_3'}{p_0}\right)^c \tag{3}$$

where p_0 is the reference atmospheric pressure, γ_d is the initial dry density of the sample, γ_w is the unit weight of water, σ_3' is the effective confining pressure, and b and c are dimensionless model parameters. To find the best fit for the observed G_{max} values, b was found to be equal to 379.4 for angular gravel mixtures and 530.1 for round gravel mixtures, while c was found to be equal to 0.591 for both types of gravels. Coefficient of correlation of the model was equal to 0.989. The c exponent varied between 0.35 and 0.70 in other research works for different kinds of soils [3]. For normally consolidated Kaolinite clays, G_{max} is proportional to $(\sigma_3')^{0.5}$, while for the materials tested in this study G_{max} was proportional to $(\sigma_3')^{0.591}$. This means that the maximum undrained shear modulus of mixed gravel-clay soils is more influenced by the change in confining pressure than NC clays.

e) Mathematical model for the normalized shear modulus versus shear strain (G/G_{max}-γ)

To find the best relationship between G/G_{max} and γ, several mathematical models were tested. The best result was obtained using the following relationship that has two parameters:

$$\frac{G}{G_{\max}} = \frac{1}{1+\left(\frac{\gamma}{\alpha}\right)^{\beta}} \tag{4}$$

From the preliminary optimization results, parameters α and β showed correlation with gravel content and the following relationships were suggested:

$$\alpha(g_c)=m\cdot\exp(n\cdot g_c) \tag{5.1}$$

$$\beta(g_c)=p\cdot\ln(g_c)+q \tag{5.2}$$

where m, n, p, and q are the constants of the equations and were determined through an optimization process for the mixtures with 54.3% gravel content and more, yielding the following equations:

For mixtures with angular gravel: $\alpha(g_c)=9.38\times10^{-5}\cdot\exp(3.46\cdot g_c)$; $\beta(g_c)=-0.56\cdot\ln(g_c)+0.64$ (6)

For mixtures with round gravel: $\alpha(g_c)=8.73\times10^{-5}\cdot\exp(2.95\cdot g_c)$; $\beta(g_c)=-0.48\cdot\ln(g_c)+0.66$ (7)

where g_c is the gravel content in decimal figure. These equations were obtained from fitting to the results of optimized α and β. Coefficient of correlation of the model was equal to 0.967. Since G/G_{max} values of the samples with the same mixture ratio but with different confining pressures were very close (Fig. 9), the effect of confining pressure on the shape of the G/G_{max} curve was not considered for the range of σ_3' values applied in this research. Figure 10 shows the test results together with model prediction using Eqs. (3-7) and (6). This figure shows the effect of granule shape on the G/G_{max}-γ variation and it is seen that the shear modulus ratio of the specimens with angular gravel content starts decaying at a higher shear strain compared with similar specimens, but with round gravel grains. It may be interpreted from Fig. 10 that in specimens with round gravel, the friction between the granules can keep them at their initial position at small shear strains ($\gamma<1\sim2\times10^{-5}$), but slipping between round granules can occur prior to crushing in angular gravel mixtures. Hence, the shear strain at which round gravel mixtures show a drop in stiffness is smaller than that of the mixtures with angular gravel. The higher values of internal friction angle for angular gravel mixtures support this hypothesis (Fig. 4). The hypothesized mechanisms of slippage between round granules and contact crushing succeeded by slippage between angular granules are illustrated in Figs. 11 and 12, respectively.

The influence of gravel content on the G/G_{max}-γ curves is shown in Fig. 13. It is seen from this figure that as the percentage of gravel is increased in the specimen, the trend shifts higher. However, the curves have equal values up to $4\sim5\times10^{-5}$ shear strain and the differences are more pronounced at higher shear strain levels. This can also be described by increasing the internal friction angle of the specimens as the percentage of gravel is increased in the mixture (Fig. 4).

When cyclic loading is applied, the interlocking of angular granules acting together with the clayey matrix between them keeps the granular skeleton undeformed. After a certain shear strain amplitude (i.e. $\gamma>2\times10^{-5}$), the contact stress between the granules exceeds the yield stress for them and crushing happens at the granules contacts. It is obvious that the greater the contacting point per unit area, the higher is the shear strain amplitude that can be resisted by the skeleton without losing its stiffness. This was noticed by the increase in the shear strain threshold at which the specimens strength starts to deteriorate when the gravel percentage was increased from 54.3% to 73.5% (Fig. 13), and accompanied by an increase in the internal friction angle of the mixtures with a higher percentage of gravel (Fig. 4).

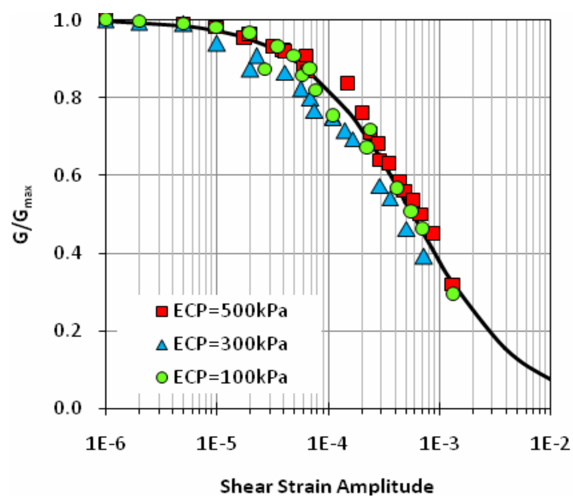

Fig. 9. Normalized shear modulus for specimen with 64.1% round gravel at different effective confining pressures (ECP)

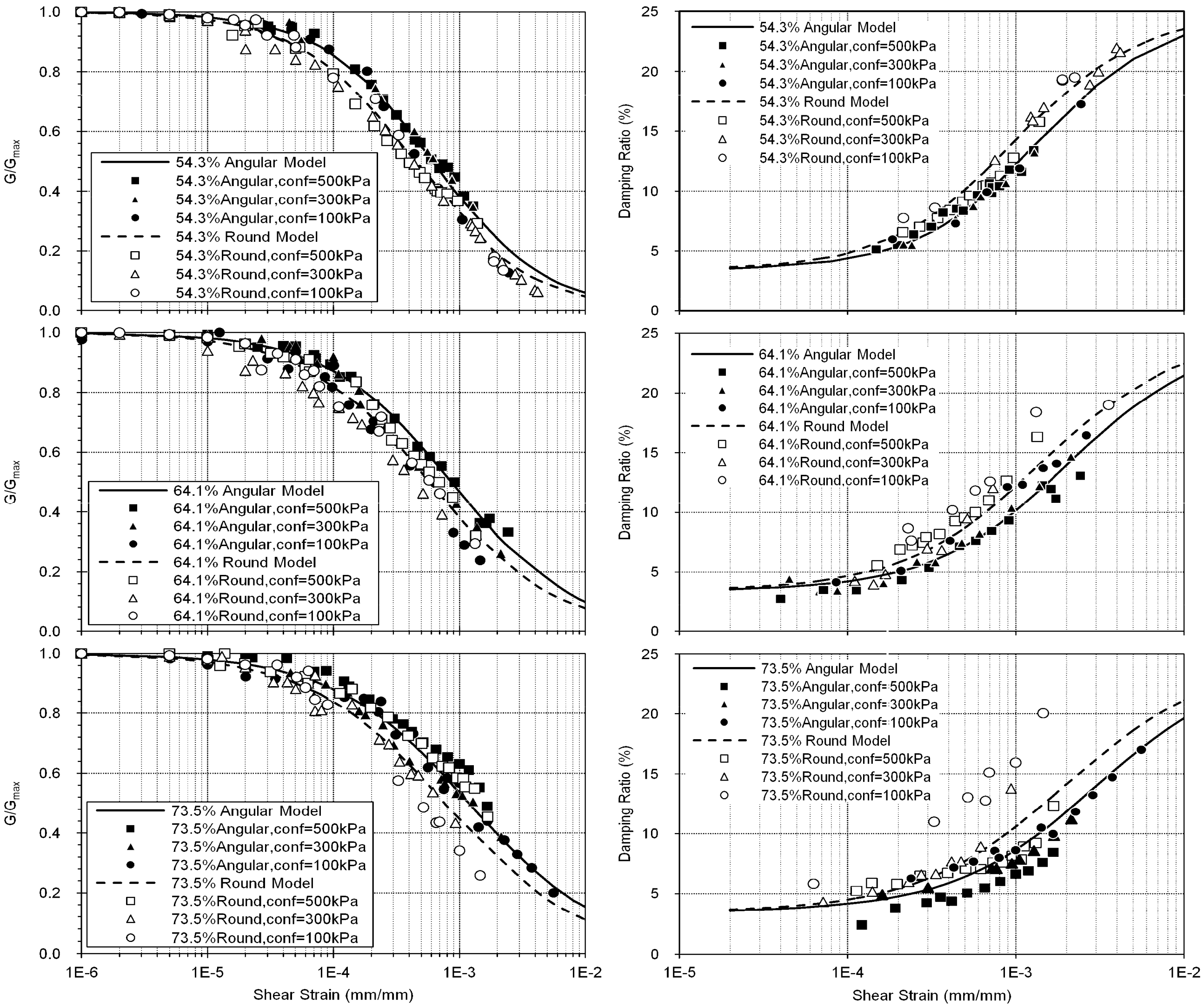

Fig. 10. Variation of normalized shear modulus (left) and damping ratio (right) with shear strain for specimens with different gravel types and contents

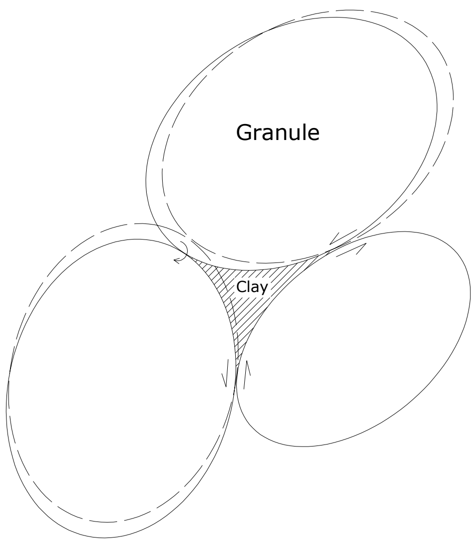

Fig. 11. Hypothesized mechanism of slippage and repositioning of round granules due to increase in shear strain level

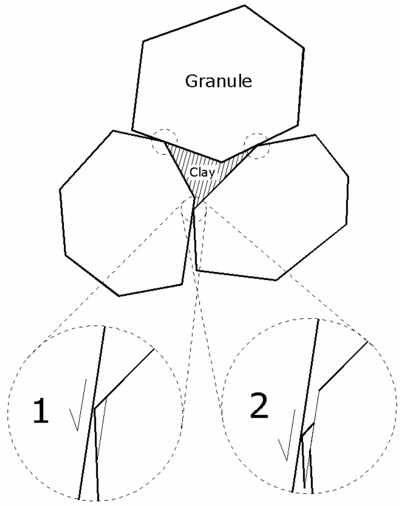

Fig. 12. Hypothesized mechanism of slippage after contact crushing due to increase in shear strain level

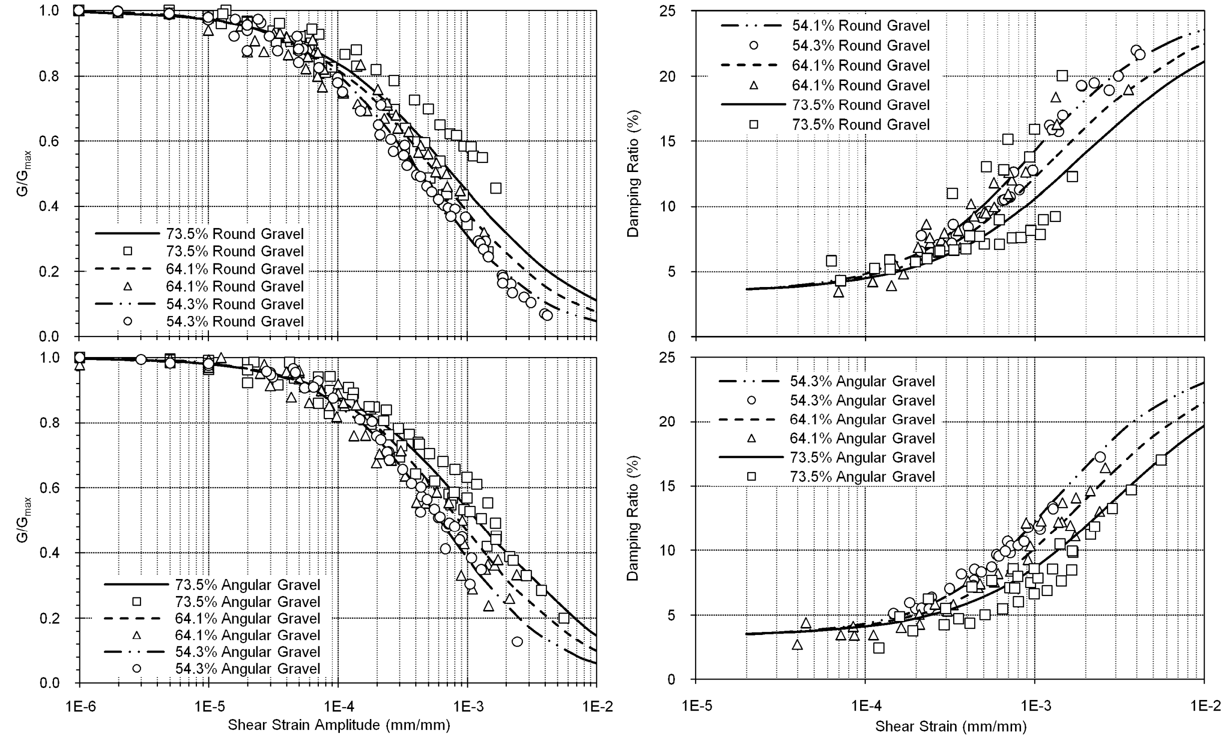

Fig. 13. The influence of gravel content on G/G_{max}-γ (left) and damping ratio (right) curves for different gravel types and contents

5. RESULTS AND DISCUSSION FOR DAMPING RATIO (*D*)

a) Damping ratio versus shear strain (D-γ)

-33.8% gravel content: In these samples, the specimens with angular gravel content displayed lower damping ratios compared with the values of specimens with round gravel. It is seen from the test results shown in Fig. 6 that as the confining pressure was increased, the damping ratio curves shifted slightly higher. However, the damping ratio values were always larger than those of the unmixed clay samples with the same confining pressure. For unmixed clay samples, the damping ratios were independent of the confining pressure and they lie close to the results of the tests on clay presented by Idriss [27].

-44.2% gravel content: At 44.2% gravel content, only round gravel mixtures at 300 and 500kPa confining pressures showed different damping ratios from other mixed samples (Fig. 6). The 100% clayey specimens had the lowest damping ratio, as before.

-54.3% gravel content: At this mixture ratio, all damping ratios were almost the same for both angular and rounded gravel mixtures at different confining pressures (Fig. 6), and were higher than values obtained for 100% clayey samples.

-64.1% gravel content: For the mixtures with 64.1% gravel, the round gravel mixtures had higher damping ratios than the angular samples (Fig. 6). The role of granules became more pronounced at this mixture ratio. It was noted that as the shear strain amplitude was increased, the difference between damping ratios at different confining pressures also increased. This may be described by considering the micro-slippage between the granules at high shear strain amplitudes. As the confining pressure increases, slippage occurs at higher shear strain amplitudes. For the angular gravel mixtures, the contacts of granules do not allow slippage to occur as easily as in the case of round granules. It is suggested that some micro-crushing happens, succeeded by slippage during cyclic loading between angular granules, especially at high shear strain amplitudes. As can be seen from Fig. 6, in the mixed specimens the damping ratio values are slightly lower for higher confining pressures and it may be concluded that granule slippage occurs more easily at lower confining pressures and more energy dissipates in the form of damping.

-73.5% gravel content: For the mixtures with the highest gravel content, i.e. 73.5%, the previous phenomenon can be observed more clearly. Samples with round gravel tested at 100kPa confining pressure had distinctly higher damping ratio values (Fig. 6). This difference is more pronounced at higher shear strain amplitudes (γ>0.05%). This may be explained by considering slippage between the round granules at low confining pressures, since at this mixture ratio less cohesive material exists between them to resist slipping. At higher confining pressures, the surface friction of the round granules does not allow slippage to occur easily, thus reducing the damping ratio.

b) Mathematical model for damping ratio variation with shear strain

It is easier to correlate damping ratios with the normalized shear modulus curves than correlating them with shear strain directly. In this research, the following relationship was found suitable for this purpose. The parameters of the proposed model were determined through an optimization process to minimize the fitness error.

$$D(\gamma) = r + s \cdot \tanh\left(w \cdot \frac{G(\gamma)}{G_{\max}} \right) \tag{8}$$

where $D(\gamma)$ is the damping ratio at the shear strain level γ; r, s, and w are the model parameters and $G(\gamma)/G_{max}$ is the normalized shear modulus at shear strain level γ obtained from Eq. (4). The model parameters were determined using the data obtained from 54.3%, 64.1% and 73.5% gravel content samples since the G/G_{max}-γ relationship was achieved for this range of gravel content. Parameters r, s and w thus determined were 25.390, −23.793, and 1.630, respectively. The fitted curves are shown for the above mentioned specimens in Fig. 10. Except for the results of specimen *r701*, other data showed a very good agreement with the proposed formula. The fitted equation had a coefficient of correlation of R^2=0.858. Application of the model for shear strains larger than 1% could be erroneous since there was no data point beyond this level. However, most researchers assume a flat D-γ relationship for shear strains larger than 1%.

The effect of gravel content in the mixtures on damping ratio is shown on Fig. 13 for the specimens prepared with angular and round grained gravels. It is obvious from this figure that for the shear strains smaller than 10^{-4}, the damping ratio value is practically the same for all specimens with the same gravel type. As the shear strain level was increased, the specimens with a lower percentage of gravel showed higher damping ratios. There may be a combination of two reasons for this observation: 1- the clay acts like a damper between the granules as the granular matrix begins to change its micro-structure due to the forces induced by larger shear strains, and 2- the number of contact points between granules decrease as the percentage of gravel decreases, so the internal friction angle of the mixture also decreases and sliding of granules on each other becomes easier.

The above argument can be followed by studying the effect of the granule shape on the damping ratio of the specimens with different mixture contents. It is seen from Fig. 10 that the specimens with round gravel showed higher damping ratios than the specimens with angular gravel with the same mixture ratio. This may describe the interlocking effect of angular granules that do not slide on each other as easy as round gravels. Hence, less energy is dissipated by friction between granules and interparticle clay due to deformation in angular mixtures.

The maximum damping ratio of the specimens is decreased as the percentage of the gravel is increased in the specimen (Fig. 13).

6. SUMMARY AND CONCLUSION

The results of this investigation indicated the complex cyclic behavior of mixed gravel and clay at different mixture ratios. A threshold value for the gravel content in the mixtures was defined below which the gravel type has no major effect on the G and D of the mixtures regardless of the shape of gravel granules. This threshold is more pronounced in the G-γ curves and is approximately between 44.2% and 54.3% gravel content. Below this limit, the gravel particles float in the clay and do not interact directly with each other, so the macro behavior of the sample is mainly dominated by the clay properties. Above this threshold, the behavior of the sample is mostly controlled by the granule structure holding the clay inside the granular matrix. As the gravel content increases, the role of gravel fraction increases, and consequently, the effect of the granule shape becomes more evident in G and D of the mixtures. The round gravel mixtures showed a higher shear modulus since they were better compacted under the same compaction effort, while the angular granules resisted compaction and maintained a lower initial dry density. This difference in G values is more pronounced under higher confining pressures.

It was observed that the maximum undrained shear modulus of the mixed gravel-clay soils is more influenced by the change in confining pressure than NC clays.

Comparing G/G_{max}-γ values for the specimens having round and angular gravel with different mixture ratios, it was concluded that in the mixtures with angular gravel, the threshold shear strain at which the material starts losing its shear modulus is more than that of mixtures with round gravel. In specimens with round gravel content, the friction between the granules can keep them at their initial position at small shear strains, but slippage occurs prior to crushing between angular grains, hence the shear strain at which these specimens show a drop in stiffness is smaller than that of the mixtures with angular gravel.

A phenomenon named *contact crushing succeeded by granule slippage* was hypothesized to explain the behavior of angular gravel mixtures in cyclic loadings. Shear modulus and damping ratio variations suggested different mechanisms for round and angular gravel mixtures under cyclic loadings. While it is believed that slippage occurs earlier during cyclic loading for round gravel mixtures, the angular granules initially resist displacement while being sheared cyclically. At low shear strain amplitudes, the specimens with angular gravel mixtures lose their shear resistance slower than the round gravel mixtures. At a certain shear strain amplitude at which the micro crushing happens at particle contacts, slippage occurs subsequently and these mixtures lose their shear strength at a slightly higher rate than the round gravel mixtures. It was suggested that the clay paste plays like a dashpot between the gravel particles which are practically rigid compared with the clayey material.

For both G/G_{max}-γ and D-γ data, the proposed models provided reasonable fit to the data for shear strains less than 1%. G/G_{max}-γ curves laid close to the curves proposed by Seed and Idriss [3] for gravel and clay. D-γ curves were close to the curves for clay proposed by Sun et al [6] and Idriss [27].

From the results of specimens with different gravel contents of angular and round gravel mixtures, it was concluded that as the percentage of gravel increases, the damping ratio reduces to lower values.

Comparing the results of specimens with angular and round gravel at different gravel contents revealed that the round gravel mixtures had higher damping ratios in the whole shear strain range.

The importance of gravel content and its granule shape on the dynamic properties of the mixtures was pointed out. Therefore, discarding the oversize aggregates in testing cyclic properties of the soils may end up in unrealistic results. Moreover, attention must be paid when deciding to use a riverbed deposit or quarry run as the borrow material for earth structures.

Considering the results of the present investigation, it may be concluded that the soil mixtures with round gravel content are more suitable for the impervious core of rockfill dams such as Karkheh since they possess higher shear moduli and larger damping ratios compared with angular gravel mixtures. They are less compressible, therefore, the post construction settlements will be lower. The higher gravel content results in higher shear modulus, but permeability data is needed to decide upon the suitable mixture ratio.

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