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Experimental study on liquefaction resistance characteristics of fine-grained coralline soils

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Abstract: Coralline soils are the emerging preferred geomaterial for island-reef engineering and harbor engineering in tropical regions, and wide gradation is the main feature of their structural composition. The liquefaction resistance of coralline soils is gaining attention due to the high seismic risk of related engineering projects. In order to investigate the liquefaction resistance of coralline soils containing fine grains, the saturated undrained dynamic strengths of three groups of representative graded samples with designed relative compaction of 0.4–0.8 and two groups of samples without fine grains were measured by large-scale dynamic triaxial liquefaction test based on an actual site of coralline soils in a tropical harbor engineering in the eastern Pacific Ocean. The test results indicated that the power function could simulate the relationship between the cyclic stress ratio and the number of cycles to cause liquefaction for coralline soils containing fine grains; the presence of fine grains and the increase of relative compaction did not significantly improve the liquefaction resistance of coralline soils; the excess pore water pressure development pattern of liquefaction process of coralline soils was similar to that of sandy soils, and the inverse sine model with two or three parameters could simulate the excess pore water pressure development of liquefaction process of coralline soils containing fine grains. The study revealed that the fine-grained coralline soils still belonged to liquefiable soils. Based on an actual engineering project, the same type of engineering project needed to consider the prevention of seismic liquefaction hazards in the design, construction and use stages, and this study provided technical support for the prevention and control of liquefaction for coralline soils.

Keywords: coralline soils; fine-grained soil; liquefaction test technique for coralline soils; excess pore water pressure; hydraulic fill coralline soil engineering

1 Introduction

Coralline soils are broadly graded cohesionless soils from gravel to fine fractions produced by coral or coral reef erosion, decomposition and artificial processing[1–2]. They are widely distributed in island-reef and harbor engineering projects in tropical marine and offshore regions as engineering site soils. Such engineering projects often face high earthquake risk. Authors' research group examined the coralline soil from a harbor engineering project in a country in the eastern Pacific Ocean. The earthquake fortification acceleration corresponding to the earthquake fortification intensity of this coral soil engineering site is designed to be 0.50*g*–0.53*g*.

Research on the engineering properties of coralline soils originated from offshore engineering needs in the 1980s, and the first recorded and studied case of coralline soil site liquefaction disaster occurred in the Guam earthquake in 1993[3], followed by the Hawaii earthquake in 2006[4–5] and Haiti earthquake in 2010[6–7], which also induce large-scale liquefaction in coral soil sites. Earthquake damage investigations have shown that liquefaction is the primary earthquake disaster-causing factor. However, in the traditional understanding, earthquake liquefaction is limited to sand, and the test devices and techniques such as the large dynamic triaxial test apparatus are relatively imperfect. The research on the liquefaction of coralline soils mostly focuses on narrowly graded coral reef sand, calcareous sand and coral sand[8–13]. The results of the liquefaction investigation of the Haiti earthquake and the actual survey of coralline soil sites indicated that broadly graded coralline soils with grains from gravelly to fine particle widely existed. Wang et al.[14] proposed a liquefaction test method adapted to the characteristics of coralline soils through the investigation of coralline soils containing large grains in an area of the South China Sea based on large-scale dynamic triaxial apparatus, and stated that the liquefaction risk of such widely graded coralline soils and the effect of gravel content on their liquefaction resistance.

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In addition to large-grained coralline soils in actual engineering sites, the content of fine grains in some sites is also relatively high, and the content of fine grains in coralline soils of some harbor engineering sites in the eastern Pacific Ocean even reaches up to 30%. The results of previous studies on the liquefaction of sands have shown that fine grain content affects the liquefaction resistance of soils. The *Code for seismic design of buildings* (GB 50011-2010)[15] of China adopts the modified clay content to express this effect, while the method of the National Center for Earthquake Engineering Research (NCEER)[16] uses the modified fine grain content to express this effect. Dong et al.[17] analyzed the influence of clay and fine grain contents on liquefaction discrimination according to the liquefaction hazard survey data of the Chi Chi earthquake, and put forward suggestions for optimization and improvement of the above two main discrimination methods. Polito et al.[18] pointed out that the influence of fine grain content on the liquefaction resistance of sand increased first and then decreased by conducting liquefaction tests with different fine grain contents and different skeleton sands. Liu et al.[19] stated that the liquefaction resistance of silt and fine sands in the Nanjing area was the lowest when the clay content was about 10% according to the liquefaction test results. Wu et al.[20] studied the influence of fine grain content on the maximum dynamic shear modulus of reef-island hydraulic fill coral sand.

Currently, there are few reports on the research of liquefaction resistance of coralline soils containing fine grains, which have lagged behind the engineering practice and led to the potential risk of related engineering projects. In this paper, according to the actual requirements of liquefaction prevention and control of coralline soils containing fine grains in the engineering sites, the characteristics of liquefaction resistance of coralline soils containing fine grains and the development pattern of excess pore water pressure under cyclic loading were studied through saturated undrained large diameter cyclic triaxial tests of coralline soils with representative gradations under different relative compactions. The goal of this study was to expand the research scope of coralline soil liquefaction resistance and provide technical support for the engineering safety of coralline soil sites.

2 Test program

2.1 Test material

The test material was taken from a coralline soil site of a harbor engineering project in the eastern Pacific Ocean, and the grain size analysis curves of the site survey sampling are shown in the light gray curve family in Fig. 1. The grain composition below the 50 mm fraction is plotted in Fig.1, the content of fine grains (grain size less than 0.075 mm) was 7%–42%. The field soil was sieved into different fractions, and representative samples were prepared by comprehensively considering the mean of the grain size distribution curve of the in situ sampling and the amount of soil taken from each fraction via sieving. The grain size distribution curves of the samples and the samples of the comparison group with fine grains removed are shown in Fig. 1, where the group CFT indicates the group containing fine grains and the group NFT indicates the group without fine grains. The basic physical properties are shown in Table 1.

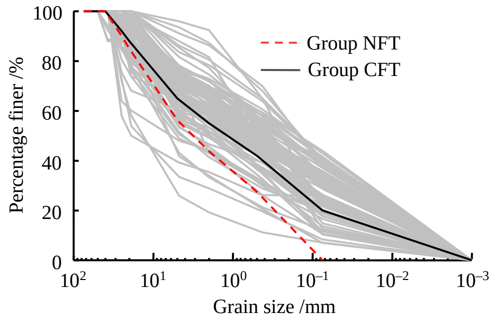


Fig. 1 Grain size distributions of the in situ and representative samples

Table 1 Physical properties of the hydraulic fill coralline soils in this study

Sample group	C_u	C_c	I_p	G_s	Maximum dry density /(g • cm^{-3})	Minimum dry density /(g • cm^{-3})
Group CFT	93.3	0.55	—	2.78	1.71	1.43
Group NFT	30.3	0.33	—	2.78	1.64	1.45
Fine	—	—	14.6	2.78	—	—

Note: C_u is the nonuniformity coefficient; C_c is the coefficient of curvature; I_p is the plastic index and G_s is the specific gravity.

It can be seen from Table 1 that both the representative samples and the comparison group samples without fine grains have good grading. The plastic index of the fine grains fraction in the test samples is 14.6, belonging to silty clay with low cohesion.

2.2 Test equipment and method

The tests in this paper were conducted in the GDS-DYNTTS-60 large-scale dynamic-static triaxial test system of the Institute of Engineering Mechanics, CEA. The adaptive dynamic loading function was used. Cylindrical samples of ϕ300 mm×600 mm were compacted in 5 layers based on controlling the relative compaction or converted the dry density. The demoulded samples controlled by low negative pressure are shown in Fig. 2. Fig. 2(a) is the typical appearance of the group CFT after preparation, and Fig. 2(b) is the typical appearance of the group NFT after preparation.

(a) Group CFT (b) Group NFT

Fig. 2 Typical sample appearance of different sample groups

The saturation, consolidation and loading methods of samples in terms of the *Standard for geotechnical testing method* (GB/T 50123 — 2019)[21] and the saturated undrained cyclic triaxial test method for large grains proposed by Wang et al.[14], which could overcome the phenomenon of "non-stationary saturation" during the sample preparation of coralline soils. As seen from Fig. 2, after removing the support mold, the surface of the rubber membrane was smooth and flat because the content of fine grains in the samples of the group CFT was high, and the dynamic strength data did not need to be modified by rubber membrane compliance. However, after the samples of the group NFT were demolded, there were significant granular undulations on the sample surface, and the dynamic strength test results needed to be corrected for compliance. In this paper, the method of rubber membrane compliance for the samples of the group NFT was adopted from the dynamic triaxial compliance correction method for large-grained saturated soils based on the two-scale method proposed by Wang et al.[22] and Liu et al.[23].

Table 2 Test case information

Case No.	Sample group	Relative density after consolidation	CSR
C40-1	CFT	0.480	0.17
C40-2		0.495	0.23
C40-3		0.465	0.25
C40-4		0.496	0.27
C60-1		0.590	0.17
C60-2		0.667	0.22
C60-3		0.596	0.25
C60-4		0.639	0.31
C80-1		0.846	0.19
C80-2		0.865	0.25
C80-3		0.846	0.33
C80-4		0.829	0.40
N40-1	NFT	0.452	0.22
N40-2		0.456	0.29
N40-3		0.432	0.34
N40-4		0.443	0.44
N80-1		0.816	0.23
N80-2		0.826	0.36
N80-3		0.821	0.39
N80-4		0.812	0.40

2.3 Case design

The saturated undrained cyclic shear tests were conducted on the samples in the group CFT and the group NFT prepared with different relative densities under different cyclic stress ratios (CRS) with an effective consolidation confining pressure of 100 kPa and a cyclic loading frequency of 0.5 Hz, and the design information for each case in the test is shown in Table 2. The group C40-1 is given as an example, C means that the sample belongs to the group CFT, 40 denotes the controlled relative density $D_r = 0.4$ when the sample is prepared, and 1–4 represents the cyclic stress ratio serial number.

3 Test results

The typical test results of representative cases for different test groups are shown in Fig. 3. Figs. 3(a) and 3(b) represent the typical test results for the samples in the group CFT, and Figs. 3(c) and 3(d) represent the typical test results for the samples in the group NFT. As could be seen from the figure, due to the adaptive dynamic loading module adopted in this sample system, the axial dynamic stress applied to the sample did not decay significantly with the increase of pore water pressure, which guaranteed the stability and reliability of the tested results. In terms of strain, all the different test cases showed a stable development of strain in the early stage, and when the excess pore pressure reached a certain critical condition, the dynamic strain entered a rapid development stage, the critical excess pore pressure ratio was about 0.8 for the samples of the group CFT while the critical point was about 0.7 for the samples of the group NFT. The excess pore water pressure of the samples could reach the effective consolidation pressure, i.e., the test reached the initial liquefaction level. Liquefaction could occur in both low relative density and high relative density samples under this test conditions.

The comparison of the development mode of excess pore water pressure for the representative cases in the group CFT and the group NFT indicated that the cyclic amplitude of the pore water pressure of the latter was significantly larger than that of the former when the pore water pressure reached a certain level, which was consistent with the pore water pressure development characteristics under the influence of rubber membrane compliance. This comparison also verified the reasonableness and necessity of the compliance correction for the dynamic strength of the group NFT in terms of test results, which coincided with the intuitive judgment described in the section 2.2.

4 Liquefaction characteristic analysis

The liquefaction resistance is the most important mechanical characteristic of liquefiable soils, and the excess pore pressure development model of the liquefaction process is one of the basic models for seismic response analysis of liquefaction sites. Based on the results of this test, the liquefaction characteristics of the fine-grained coralline soils were analyzed from the following aspects.

4.1 Liquefaction resistance comparison

The method in Chinese standards generally expresses the anti-liquefaction capacity of soil samples through the relationship between cyclic stress ratio CSR and number of cycles to cause liquefaction N_f in the dynamic triaxial test, i.e., the CSR– N_f relationship curve[15], which is also one of the commonly-used methods in the world.

The effect of the fine grains on the liquefaction resistance of coralline soils was analyzed by comparing the difference in the CSR– N_f curves of the group CFT and the group NFT, and the curves of the two test groups are plotted in Fig.4.

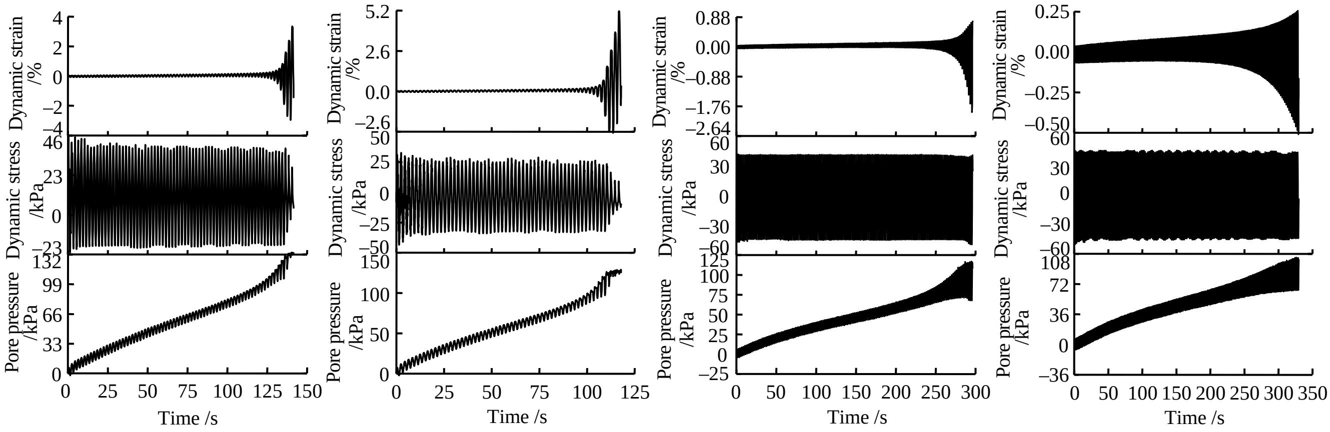


Fig. 3 Typical test results for different sample groups

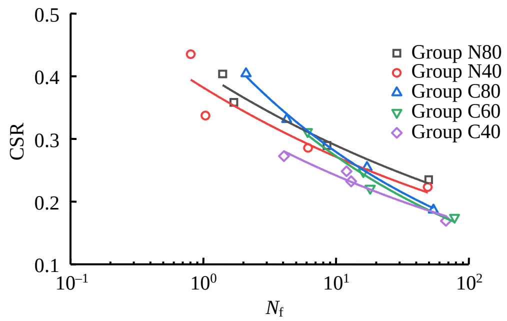


Fig. 4 CSR-N_f relationship curves of CFT and NFT sample groups

As seen in Fig.4, the CSR–N_f curves of coralline soils of different sample groups could be expressed by the power function relationship of Eq. (1), and the fitting analysis of the test results is shown in Table 3.

$$\mathrm{CSR} = mN_f^n \tag{1}$$

In the analysis of liquefaction resistance, the CSR at different numbers of cycles to cause liquefaction is commonly used to represent the liquefaction resistance of soils under different earthquake magnitudes. The test results for each sample group with different relative compactions at typical numbers of cycles are compared and analyzed, as shown in Table 4. For the convenience of analysis, the relative values of liquefaction resistance in Table 4 were expressed based on the case C40 within the sample group, i.e., the relative value of liquefaction resistance was considered to be 1, and the liquefaction resistances of other case groups were treated in equal proportion.

By comparing the two pairs of test results in Fig. 4 and Table 4 for the groups C40 and N40 and the groups C80 and N80, it could be found that after the number of cycles to cause liquefaction surpassed 8, the liquefaction resistance of the group CFT was smaller than that of the group NFT under the same relative compaction, i.e., when the earthquake magnitude outweighed 6.5, the liquefaction resistance of the sample groups with fine-grained coralline soils was less than that of the coarse-grained soil skeleton sample groups after removing the fine grains. The liquefaction resistance of fine-grained coralline soils was reduced by 10%–20%.

Table 3 Summary of liquefaction characteristic analysis parameters

Case No.	CSR-N_f curve parameter m	CSR-N_f curve parameter n	Goodness of fit of pore pressure model R^2: One-parameter model	Two-parameter model	Three-parameter model
C40-1	0.352	−0.163	0.919	0.999	0.993
C40-2			0.953	0.996	0.943
C40-3			0.922	0.994	0.947
C40-4			0.845	0.985	0.982
C60-1	0.469	−0.236	0.907	0.999	0.989
C60-2			0.906	0.998	0.957
C60-3			0.890	0.997	0.968
C60-4			0.699	0.981	0.993
C80-1	0.474	−0.229	0.898	0.998	0.998
C80-2			0.863	0.992	0.971
C80-3			0.607	0.997	0.986
C80-4			—	0.862	—
N40-1	0.382	−0.148	0.697	0.981	0.999
N40-2			0.647	0.989	0.999
N40-3			—	—	—
N40-4			—	—	—
N80-1	0.405	−0.146	0.704	0.989	0.999
N80-2			0.713	0.980	0.999
N80-3			—	—	—
N80-4			—	—	—

Table 4 Relative liquefaction resistance for different cases under typical number of cycles

Typical number of cycles	Sample group CFT: Case group C40	Case group C60	Case group C80	Sample group NFT: Case group N40	Case group N80
5	1	1.18	1.21	1.11	1.18
8	1	1.14	1.17	1.12	1.19
12	1	1.11	1.14	1.13	1.20
20	1	1.07	1.11	1.14	1.21
30	1	1.04	1.08	1.14	1.22
50	1	1.00	1.04	1.15	1.23

The results of previous studies on terrigenous soils with fine grains indicated that the influence of fine grains on the liquefaction resistance of soils was a comprehensive reflection of the interaction between the lubricating effect and the binding effect. Generally, when the content of fine grains is low, the fine grains can not wrap the coarse grains or the wrapping is thin, and its lubricating effect is dominant, the soils show the decrease of liquefaction resistance; as the content of fine grains reaches a certain level, the binding effect prevails, the soils exhibit in a growth in liquefaction resistance. For example, the study of Liu et al.[19] on Nanjing fine sand revealed that the lowest liquefaction resistance was obtained for the samples with 10% content of fine grains. In contrast, the experimental study by Polito et al.[18] showed that the liquefaction resistance was reduced by 30%–50% or even more when the content of fine grains was 20%.

The content of fine grains for the samples in this study was about 20%. From the morphology analysis of the samples, the samples were shown to be smooth, the fine grains wrapped the coarse grains sufficiently, and the presence of the fine grains reduced the liquefaction resistance of the samples by around 15%. On the one hand, from the plastic index of the fine fraction in Table 1, it could be seen that the fine fraction of the coralline soil samples was mainly composed of calcium carbonate minerals, which were less hydrophilic and showed lower viscosity, hence the lubricating effect in the overall samples was better than the binding effect and the fine grains reduced the liquefaction resistance in the test; on the other hand, the coarse fraction of coralline soils had a complex grain shape with many edges and branches, which made the lubricating effect of the fine grains not obvious, which contribute to a lower liquefaction resistance of the sample than that of the terrigenous soil.

4.2 Effect of relative density

There are few reports on the influence of relative density on the liquefaction resistance of boardly graded coralline soils. Salem et al.[8] investigated the effect of density on the liquefaction resistance of coral sand (soil) with a comparative test of Dabaa coral sand and Hussien quartz sand in North African sea areas. Their test results are plotted in Fig. 5 together with the test results of this study and the boardly graded terrigenous gravel soil sampled by freezing method.

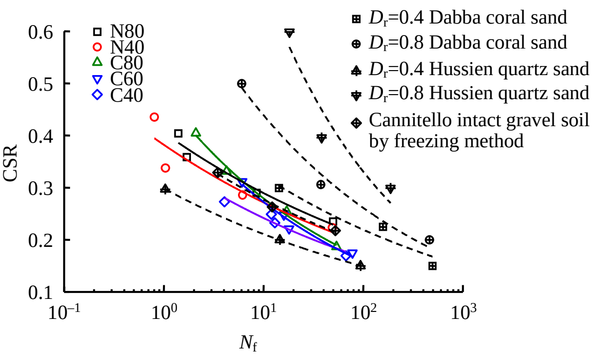


Fig. 5 CSR–N_f relationship curves of silica sands and coralline soils with different relative densities

Salem et al.[8] stated that the liquefaction resistance of coral sand was higher than that of normal quartz sand when the relative density was small, and the opposite was true when the relative density was high. The test results of the group CFT in Fig. 5 and Table 4 show that the liquefaction resistance of the group CFT increases with the increase of the relative density. When the number of cycles is greater than 8, i.e., the corresponding earthquake magnitude is greater than 6.5, the relative density increases from 0.4 to 0.8, and the liquefaction resistance only increases within 20%. The tests of the group NFT shows the same pattern as those of Salem. In contrast, for quartz sand, the same relative density increases from 0.4 to 0.8, and the liquefaction resistance increases by 200%. This indicates that both the widely graded coralline soils and the relatively narrowly graded coralline soils (i.e., coral sand) are much less sensitive to changes in relative density than the general quartz sand.

At present, for coralline soil liquefaction sites, biochemical reinforcement methods such as microbiological methods are still under research, and soil replacement of liquefiable layers is not feasible in sites such as island-reef projects. Therefore, traditional ground treatment methods such as dynamic compaction, rolling and vibroflotation with the goal of densification are still the current typical ground treatment methods for coralline soil sites to cope with liquefaction risks. However, from the results of this study, the sensitivity of coralline soils to liquefaction is much lower than that of general terrigenous sands, so this characteristic of coralline soils needed to be carefully considered in the design and implementation of anti-liquefaction reinforcement for coralline soil sites, and the previous experience of reinforcement for terrigenous soils needed to be used with care.

As shown in Fig. 5, the comparison between the test results in this paper and the liquefaction test results of typical terrigenous widely graded gravel soil sampled by freezing method shows that the liquefaction resistances of the two are basically consistent, which was the same as the historical earthquake liquefaction survey results from coralline soil sites by Yuan et al.[1]. It can be concluded that the ground motion conditions that trigger earthquake-induced liquefaction hazard in coralline soil sites are similar to those in terrigenous soil sites, and the earthquake conditions that can induce liquefaction in the general sites can still cause liquefaction in the coralline soil sites. The surface ground motion for this excitation condition is usually considered to be about 0.1g–0.5g, which is basically in accordance with the earthquake fortification requirement of VII intensity in China. We take the soil sampling site in this study as an example, its earthquake fortification level is 0.50g–0.53g, which shows that the site coralline soil has a very high risk of earthquake liquefaction if the site soil is not treated.

4.3 Excess pore water pressure development mode

The relationship between the pore pressure ratio and the ratio of the number of cycles is the basic model for the analysis of soil response in liquefiable sites by the effective stress method, and the most widely used model is arcsine model proposed by Seed et al.[24].

$$\frac{u}{\sigma_c'}=\frac{2}{\pi}\arcsin\left(\frac{N}{N_f}\right)^{\frac{1}{2\theta}} \tag{2}$$

where u is the residual excess pore water pressure; σ_c' is the effective consolidation pressure; u/σ_c' is the pore pressure ratio; N is the current number of cycles; N_f is the number of cycles to cause liquefaction, N/N_f is the ratio of the number of cycles; and θ is a test parameter, which can be obtained by fitting the test results.

On the basis of the model suggested by Seed et al.[24], Wang[2] proposed a two-parameter correction model according to the experimental study of coralline soils, as follows:

$$\frac{u}{\sigma_0'}=\frac{a}{\pi}\arcsin\left(\frac{N}{N_f}\right)^{\frac{1}{b}} \tag{3}$$

where a and b are the test parameters, and the other parameters are the same as the ones in Seed model. The model has good applicability to coralline soils containing large fractions of gravel or even larger grains.

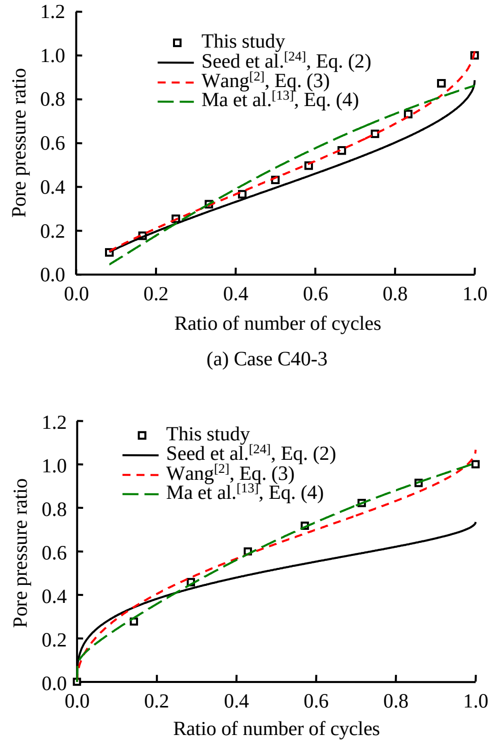
Ma et al.[13] developed a three-parameter correction model based on the experimental study of coralline soils as follows:

$$R_u=a\times\frac{2}{\pi}\arcsin\left(N/N_L\right)^{1/2\theta}+b\times\arctan\left(N/N_L\right) \tag{4}$$

where R_u is the pore pressure ratio; N_L is the number of cycles to cause liquefaction, which is the same as N_f in Eq. (2); and a, b and θ are the test parameters. This model can describe the pore pressure development more precisely for the stage with a higher pore pressure.

Equations (2)–(4) were applied to fit the excess pore water pressure measurement results of this test, the goodness-of-fit results R^2 are shown in Table 3, and the typical fitting analysis curves are shown in Fig. 6.

As seen in Fig. 6 and Table 3, the one-parameter model generally underestimate the development rate of the pore pressure ratio and has low goodness of fit. However, the two-parameter and three-parameter models can reasonably estimate the development of the pore pressure ratio for both the sample group CFT and sample group NFT, in which the three-parameter model simulates the group NFT slightly better than the two-parameter model. Thus, it is deemed that the Seed's pore pressure model modified by the two-parameter or three-parameter models can simulate the development pattern of excess pore water pressure in the liquefaction process of coralline soils containing fine grains in a more reasonable way.



(a) Case C40-3

(b) Case N40-2

Fig. 6 Typical results of fitting analysis of the relationship between pore pressure ratio and ratio of number of cycles

5 Conclusion

Based on an actual engineering project of a coralline soil site, large-scale dynamic triaxial tests were conducted to investigate the liquefaction resistance characteristics of coralline soils containing fine grains, and the following conclusions were obtained through comparative analysis:

(1) The broadly graded coralline soils containing fine grains are liquefiable soils, and the present study refines the category of liquefiable coralline soils. For the earthquake action of magnitude 6.5 or higher, the presence of fine grains reduces the liquefaction resistance of coralline soils by 10%–20%, but this reduction is not as significant as that of terrigenous soils. Based on this work, subsequent research can be carried out to study the influence of the variation of fine grain contents on the liquefaction resistance of coralline soils.

(2) The studied coralline soils is comparable to the general terrigenous gravel soil in liquefaction resistance. It is suggested that if the earthquake fortification intensity is larger than VII (ground acceleration is greater than 0.15g), ground treatment should be considered to resist seismic liquefaction .

(3) As the relative density increases from 0.4 to 0.8, the liquefaction resistance of coralline soils containing fine grains only increases by less than 20%. However, according to previous research results, the liquefaction resistance of general sand increases by more than 200% when the relative density increases, and the liquefiable soil can even be changed into non-liquefiable soil. Thus, it is concluded that the treatment of

coralline soil by compacting the soil layer is not economically reasonable for the improvement of liquefaction resistance, and other methods should be introduced for the anti-liquefaction treatment of coralline soil.

(4) Both two-parameter correction and the three-parameter correction for the Seed pore pressure model can estimate the pore water pressure development during the liquefaction of coralline soils reasonably. The former has fewer parameters and the latter has better goodness of fit at the high pore pressure ratio stage. However, the physical meaning of the parameters of two model and the relationship with the physical and mechanical parameters of the coralline soils are not clear and can only be obtained through experiments at present.

Referecnces

[1] YUAN Xiao-ming, ZHANG Wen-bin, DUAN Zhi-gang, et al. Analysis for characteristic of seismic liquefaction in engineering sites of coralline soils[J]. Chinese Journal of Rock Mechanics and Engineering, 2019, 38(10): 3799–3811.

[2] WANG Luan. Liquefaction characteristics of coralline soils[D]. Harbin: Institute of Engineering Mechanics, China Earthquake Administration, 2021.

[3] SWAN S W, HARRIS S K. The island of Guam earthquake of August 8, 1993[R]. New York: State University of New York at Buffalo, 1993.

[4] MEDLEY E W. Geological engineering reconnaissance of damage caused by the October 15, 2006 Hawaii earthquakes[J]. International Journal of Geoengineering Case Histories, 2007, 1(2):89–135.

[5] CHOCK G, KINDRED T, ROBERTSON I, et al. Compilation of observations of the October 15, 2006 Kiholo Bay (Mw 6.7) and Mahukona (Mw 6.0) earthquakes, Hawaii[R]. Hawaii: Earthquake Engineering Research Institute, 2006.

[6] GREEN R A, OLSON S M, COX B R, et al. Geotechnical aspects of failures at Port-au-Prince seaport during the 12 January 2010 Haiti earthquake[J]. Earthquake Spectra, 2015, 27(Suppl.1): 43–65.

[7] RATHJE E, BACHHUBER J, COX B, et al. Geotechnical engineering reconnaissance of the 2010 Haiti earthquake[R]. Haiti: Geotechnical Extreme Events Reconnaissance, 2010.

[8] SALEM M, ELMAMLOUK H, AGAIBY S. Static and cyclic behavior of North Coast calcareous sand in Egypt[J]. Soil Dynamics and Earthquake Engineering, 2013, 55: 83–91.

[9] YU Hai-zhen, WANG Ren. Experimental study on dynamic strength of calcareous sand[J]. Rock and Soil Mechanics, 1999, 20(4): 6–11.

[10] MENG Qing-shan, QIN Yue, WANG Ren. Liquefaction characteristics and mechanism of coral reef sediments[J]. Soil Engineering and Foundation, 2012, 26(1): 21–24.

[11] GAO Ran, YE Jian-hong. Experimental investigation on the dynamic characteristics of calcareous sand from the reclaimed coral reef islands in the South China Sea[J]. Rock and Soil Mechanics, 2019, 40(10): 3897–3908.

[12] SANDOVAL E A, PANDO M A. Experimental assessment of the liquefaction resistance of calcareous biogenous sands[J]. Earth Sciences Research Journal, 2012, 16(1): 55–63.

[13] MA Wei-jia, CHEN Guo-xing, LI Lei, et al. Experimental study on liquefaction characteristics of saturated coral sand in Nansha Islands under cyclic loading[J]. Chinese Journal of Geotechnical Engineering, 2019, 41(5): 981–988.

[14] WANG Luan, WANG Yun-long, YUAN Xiao-ming, et al. Experimental study on liquefaction resistance of hydraulic fill coralline soils in artificial site based on large-scale dynamic triaxial apparatus[J]. Rock and Soil Mechanics, 2021, 42(10): 2819–2829.

[15] Ministry of Housing and Urban-Rural Development of the People’s Republic of China. GB 50011－2010 Code for seismic design of buildings[S]. Beijing: China Architecture and Building Press, 2010.

[16] YOUD T L, IDRISS I M. Proceedings of the NCEER Workshop on evaluation of liquefaction resistance of soils[R]. Buffalo, NY: NCEER, 1997.

[17] DONG Lin, WANG Lan-min, XIA Kun, et al. Improvement of SPT-based liquefaction discrimination methods for fines-containing sandy soils[J]. Chinese Journal of Geotechnical Engineering, 2015, 37(12): 2320–2325.

[18] POLITO C P, MARTIN II J R. A reconciliation of the effects of non-plastic fines on the liquefaction resistance of sands reported in the literature[J]. Journal of Geotechnical & Geoenvironmental Engineering, 2012, 19(5): 408–415.

[19] LIU Xue-zhu, CHEN Guo-xing. Experimental study on influence of particle content on liquefaction of Nanjing fine sand[J]. Earthquake Engineering and Engineering Vibration, 2003, 23(3): 150–155.

[20] WU Yang, CUI Jie, LI Neng, et al. Experimental study on the mechanical behavior and particle breakage characteristics of hydraulic filled coral sand on a coral reef island in the South China Sea[J]. Rock and Soil Mechanics, 2020, 41(10): 3181–3191.

[21] Ministry of Water Resources of the People’s Republic of China. GB/T 50123－2019 Standard for geotechnical testing method[S]. Beijing: China Planning Press, 2019.

[22] WANG Luan, SUN Rui, LIU Hui-da, et al. New method to compensate for membrane compliance in dynamic triaxial liquefaction tests on gravelly soils[J]. Chinese Journal of Geotechnical Engineering, 2020, 42(12): 2281–2290.

[23] LIU Hui-da, YUAN Xiao-ming, WANG Luan, et al. Research and new calculation formula of membrane penetration in wide graded gravel soils sample[J]. Chinese Journal of Rock Mechanics and Engineering, 2020, 39(4): 804–815.

[24] SEED H B, MARTIN P P, LYSMER J. Pore-water pressure changes during soil liquefaction[J]. Journal of the Geotechnical Engineering Division, 1976, 102(4): 323–346.