Comparative Study on Seismic Response of Pile Group Foundation in Coral Sand and Fujian Sand

Qi Wu 1, Xuanming Ding 1,2,*, Yanling Zhang 1 and Zhixiong Chen 1,2

1 School of Civil Engineering, Chongqing University, Chongqing 400045, China; wuqi_cqu@163.com (Q.W.); ylz_cqu@163.com (Y.Z.); chenzhixiong@cqu.edu.cn (Z.C.)
2 National Joint Engineering Research Center of Geohazards Prevention in the Reservoir Areas, Chongqing 400045, China
* Correspondence: dxmhhu@163.com

Received: 12 January 2020; Accepted: 9 March 2020; Published: 11 March 2020

Abstract: The physical and mechanical properties of coral sand are quite different from those of common terrestrial sands due to the special marine biogenesis. Shaking table tests of three-story structures with nine-pile foundation in coral sand and Fujian sand were carried out in order to study the dynamic response characteristics of pile-soil-structure system in coral sand under earthquake. The influence of shaking intensity on the dynamic response of the system was taken into consideration. The results indicated that the peak value of the excess pore pressure ratio of coral sand was smaller than that of Fujian sand under two kinds of shaking intensities; moreover, the development speed of excess pore pressure ratio of coral sand was smaller than that of Fujian sand. The liquefaction of coral sand was more difficult than Fujian sand under the same relative density and similar grain-size distribution. The horizontal displacement, settlement, column bending moment, and pile bending moment of coral sand were smaller than those of Fujian sand, respectively. The magnification effect of column bending moment of buildings in coral sand was less than that in Fujian sand with increasing shaking intensity. This study can provide some supports for the seismic design of coral reef projects.

Keywords: coral sand; Fujian sand; shaking table test; dynamic response; pile group

1. Introduction

Coral sand is a special geotechnical medium that is rich in calcium carbonate and deposited by marine protozoan skeletons. Coral sand is widely distributed in the coasts and reefs of Australia, the Gulf of Mexico, and the South China Sea [1]. When compared with the common terrigenous sands, coral sand has special physical properties, such as multi-voids, irregular particle shape, and fragility. These properties lead to the difference of compressibility, shear strength, and permeability between the coral sands and the common terrigenous sands [2–5]. With the expansion of construction scale on coral reefs recent years, the seismic safety of coral reefs has attracted considerable attention [6–9]. Historically, a magnitude 8.1 earthquake struck Guam in the Pacific Ocean on August 8, 1993, which caused severe liquefaction of coral sandy sites in dredger fills and lake sediments [10]. Large-scale liquefaction of coral sand also occurred in the Hawaii earthquake in 2006 and Haiti earthquake in 2010, which caused irreparable losses to local infrastructure and people’s lives and property [11–13].

Xiao et al. analyzed the dynamic strength, cyclic deformation, and pore pressure of saturated coral sand and microbial reinforced coral sand by the dynamic triaxial test in order to study the dynamic response characteristics of pile-soil-structure system in coral sand under earthquake [1]. Salem et al. pointed out that the calcareous sand has higher dynamic strength than siliceous sands and suggested the dynamic resistance ratio-confining pressure-relative density relationship of calcareous sand from North Coast Dabaa [14]. Xu et al. studied the transmission law of explosive stress wave...
in saturated coral sand and quartz sand, and concluded that saturated coral sand has a stronger absorption and attenuation effect on explosive stress wave than quartz sand [15]. Sandoval et al. found the differences of dynamic response between Puerto Rico coral sand and Ottawa siliceous sand through the result of dynamic triaxial tests [16]. Other scholars have also carried out some research on the dynamic characteristics of coral sands [17,18]. However, the former researches mainly focused on the strength, deformation, and pore pressure characteristics of coral sands in small-scale models, and it does not involve the dynamic response while considering the interaction between structures and soil in large-scale coral sands sites.

The shaking table test is an important method for studying the dynamic response of liquefiable sites and the structures on them under earthquakes. Tang et al. performed the failure mode of pile foundation of bridge under earthquakes while considering the pile-soil interaction [19]. Dashti et al. carried out a series of liquefaction model tests of low-rise buildings through the centrifuge shaking table test; the mechanism of building settlement that was caused by liquefaction was discussed [20]. Chen et al. studied the dynamic characteristics and damage law of subway station [21]. Other scholars have also undertaken a lot of research on the dynamic response of various structures and foundations under earthquake using shaking table test and found that the depth and compactness of soil, the existence of structures, and the input parameters of shaking excitation have obvious effects on site liquefaction and dynamic response of structures and soil [22–30]. However, the above studies are aimed at the terrigenous sands, and there are few studies on coral sand at present.

Important military buildings for large equipment and heavy machinery on coral sand sites often use the low-rise concrete structures that are supported by pile group foundations. A series of shaking table tests of buildings with nine-pile foundation in coral sand were carried out in order to study the seismic response of coral sand and the structures on it. Shaking table tests on quartz sand sites in the same situation were also performed as comparative tests. The differences and similarities of dynamic characteristics of coral sand and Fujian sand were compared and analyzed based on the results of pore water pressure, acceleration, displacement, and dynamic bending moment.

2. Shaking Table Test

The test was performed using ANCO shaking table, which is sourced from ANCO company, Los Angeles, America. The shaking table can simultaneously carry out horizontal and vertical shaking, with a dimension of 1200 mm × 1200 mm in plane, as shown in Figure 1. The maximum proof model mass, horizontal displacement, and base excitation is 1000 kg, 100 mm, and 2.0 g, respectively. A soil container with size of 950 mm in length, 850 mm in width, and 550 mm in depth was used. The soil container changes from a laminar shear type to a rigid type when the controls at the four corners of the soil container are tightened (Figure 1). Two DHDAS acquisition instruments with 96 channels were used to collect sensor signal data simultaneously in this test. The DHDAS acquisition instruments is sourced from Donghua Testing Technology Co., Ltd., Jingjiang, China.
The rigid soil container was used in the test to prevent the two sands from interacting with each other. The similarity ratio of elastic modulus quantities, such as time, acceleration, and displacement, were derived based on the similarity relations, the soil container to prevent the mix of the two kinds of sands. Coral sand that was used in the test was taken from a reef in the South China Sea, and the quartz sand used was Fujian sand foundation. The soil container was separated into two halves along the orientation of shaking, half of which was for preparing the coral sand foundation and the other half was for preparing Fujian sand foundation. Relatively compressible foam cushions with thicknesses of 100 mm were attached to the inner walls of the soil container perpendicular to the shaking direction to reduce the energy that was reflected by the container. The foam was made of polystyrene. The density, water absorption, and compressive strength of the foam were 30 kg·m$^{-3}$, 1%, and 150 kPa. A foam board was installed at the middle of the soil container to prevent the mix of the two kinds of sands. Coral sand that was used in the test was taken from a reef in the South China Sea, and the quartz sand used was Fujian standard sand.

2.1. Similitude Ratio

The geometric similarity ratio is set as 1:40 based on the maximum load of the shaking table and the size of the soil container. According to the Buckingham theory [31,32] and scaling laws [33], length $l$, elastic modulus $E$, and equivalent density $\rho$ were chosen as the basic physical quantities. The similarity ratio of elastic modulus $S_E$ is obtained by comparing the elastic modulus of the model building material and the prototype building material. The similarity ratio of equivalent density $S_\rho$ is calculated by Equation (1), when the preset acceleration similarity ratio $S_a$ is 1. The other physical quantities, such as time, acceleration, and displacement, were derived based on the similarity relations, as shown in Table 1.

$$S_a = \frac{S_E}{S_\rho S_l}$$ (1)

| Parameters       | Similitude Relation | Similitude Ratio |
|------------------|---------------------|------------------|
| Length $l$       | $S_l$               | 1:40             |
| Equivalent density $\rho$ | $S_\rho$        | 6:1              |
| Elastic modulus $E$ | $S_E$             | 3:20             |
| Acceleration $a$ | $S_a = S_E S_\rho^{-1} S_l^{-1}$ | 1                |
| Duration $t$     | $S_t = S_E^{0.5} S_\rho^{0.5} S_l$ | 0.158           |
| Frequency $\omega$ | $S_\omega = S_l^{-1}$ | 6.325           |
| Stress $\sigma$  | $S_\sigma = S_E$   | 3:20             |
| Linear displacement $r$ | $S_r = S_l$     | 1:40             |

2.2. Preparation of the Model

The soil container was separated into two halves along the orientation of shaking, half of which was for preparing the coral sand foundation and the other half was for preparing Fujian sand foundation. The rigid soil container was used in the test to prevent the two sands from interacting with each other. Relatively compressible foam cushions with thicknesses of 100 mm were attached to the inner walls of the soil container perpendicular to the shaking direction to reduce the energy that was reflected by the container. The foam was made of polystyrene. The density, water absorption, and compressive strength of the foam were 30 kg·m$^{-3}$, 1%, and 150 kPa. A foam board was installed at the middle of the soil container to prevent the mix of the two kinds of sands. Coral sand that was used in the test was taken from a reef in the South China Sea, and the quartz sand used was Fujian standard sand.
Figure 2 presents the grain-size distribution of the two kinds of sands, which shows that the grain-size distributions of the two kinds of sands are similar. Table 2 illustrates the basic physical parameters of the two kinds of sands.

![Grain-size distribution curves of coral sand and Fujian sand.](image)

**Figure 2.** Grain-size distribution curves of coral sand and Fujian sand.

| The Category of Sand | Specific Gravity | Maximum Dry Density (g cm\(^{-3}\)) | Minimum Dry Density (g cm\(^{-3}\)) | Coefficient of Uniformity, \(C_u\) | Mean Grain Size, \(D_{50}\) (mm) |
|----------------------|------------------|--------------------------------------|-------------------------------------|-----------------------------------|----------------------------------|
| Coral sand           | 2.80             | 1.48                                 | 1.15                                | 2.67                              | 0.48                             |
| Fujian sand          | 2.63             | 1.64                                 | 1.35                                | 4.50                              | 0.60                             |

The model foundation consisted of two parts along the vertical direction, above and below the water level. The soil layer below the water level was prepared while using the water sedimentation method. The water surface is about 10 cm above the sand surface throughout the process. The main factor affecting the relative density of the model foundation that was prepared by the water sedimentation method is the fall distance [34]. The two kinds of sands fall to the water surface at the same height during sample preparation, and other influencing factors, such as the speed and flow of the sand ejection head, should be consistent, in order to make the relative density of the coral sand and quartz sand sites approximately the same. During the preparation of the model foundation, a calibrated aluminum box was used for sampling analysis in time to ensure that the uniformity and relative density of the two sand model foundations were approximately similar. The soil layer above the water level with a thickness of 30 mm was prepared to keep consistent with the actual engineering situation. The relative density of the whole model foundation is 0.67.

The prototype of the structures were three-story concrete frame buildings with a nine-pile foundation. The buildings are used to store large equipment and heavy machinery on coral sand sites. The organic glass was selected to prepare the model buildings and piles, because the geometric dimensions of the model building were too small after shrinking according to the similar law and there were practical operation difficulties in concrete pouring. The three-story model building with side length of 180 mm, net height of 100 mm for the bottom floor, and 90 mm for the other floors is made of organic glass, as shown in Figure 3. The organic glass plates with the thickness of 5 mm were used as slabs, under which rectangular organic glass bars with geometry of 5 mm × 5 mm × 160 mm were installed to simulate the beam. The cross-sectional dimension of the model column was 10 mm × 10 mm, and the outer edge of which was leveled with the outer edge of the model beam. The geometry of the model raft was 220 mm in length and 15 mm in thickness. The diameter and length of the model pile were 20 mm and 400 mm, respectively. The organic glass can be changed into liquid state by dropping acetone on it. Each component of the model building was dissolved in order to connect by the special adhesive of organic glass. In the connection process, the verticality among the components...
was ensured by the triangular rule and the integrity of model was guaranteed by the connection after the organic glass dissolved.

![Figure 3. Model structure details.](image)

While considering the gravity effect on the prototype structure, steel plates weighing 3.5 kg with geometry of 150 mm in length, 150 mm in width, and 20 mm in height were glued on each floor of the model structure, and steel plates weighing 6.2 kg with geometry of 150 mm in length, 150 mm in width, and 35.5 mm in height was glued on the model raft. A total additional mass of 16.7 kg or 81% of the enough artificial mass was placed on the model structure. The density of the model building is increased by adding enough artificial mass to meet the similarity rate. The gravity effect of pile foundation was ignored in this test.

### 2.3. Instrumentation and Experimental Program

The coral sand and Fujian sand sites adopt the same sensor arrangement, as shown in Figure 4. Laser displacement sensors with heights of 150 mm and 330 mm from the ground of model foundation were installed on the shaking table using the rigid brackets, respectively, and the rigid targets point were installed on the floors of the structure. The horizontal displacement sensor was 280 mm from the vertical center line of the structure. The model foundation stood for 24 h before the test. The capillarity action is considered and the water level is consistent with Figure 4b during 24 h. The experimental program was arranged as a comparative study of the dynamic response of pile-soil-structure system in coral sand and Fujian sand sites, while considering the influence of shaking intensity. Table 3 summarizes the specific experimental program. Figure 5 shows the time history curves of sinusoidal wave excitation. The sinusoidal wave has a simpler law than the seismic wave, and it is easy to analyze the dynamic response of the model foundation and structure, many scholars have used sinusoidal wave as excitation, especially in the liquefaction condition [35,36]. The white noise with an amplitude of 0.02 g and a duration of 20 s was input before and after each sinusoidal wave excitation input.
(a) Plane layout sketch

(b) A-A profile sensor layout sketch

Figure 4. General configuration of model tests.
3. Macroscopic Phenomena of Soil and Structure

Figure 6a shows the surfaces of coral sand and Fujian sand sites. When the 0.1 g sinusoidal wave excitation was input, the building on the coral sand site began to shake slightly, and no water was discharged from the model soil. The phenomenon of Fujian sand site was similar to that of the coral sand site. Figure 6b presents the site condition after test. When the 0.2 g sinusoidal wave excitation was input, the building on the coral sand site began to shake slightly, and no water was discharged from the model soil. The phenomenon of Fujian sand site was similar to that of the coral sand site. Figure 6c presents the site condition after test. When the 0.2 g sinusoidal wave excitation was input, the shaking degree of buildings in the two kinds of sand sites increased and reached the maximum at about 4.8 s, and then the shaking degree suddenly decreased. With the input of shaking excitation, the shaking degree gradually increased again. The buildings subsided and inclined, and the soil on both sides of the building rose. For coral sand, the surface of model soil was gradually getting wet, and little water accumulated after test, as shown in Figure 6c. The water of Fujian sand site increased from the surrounding of the soil container and accumulated a little on the surface of the site (Figure 6d).
4. Result and Discussion

4.1. Pore Water Pressure Response

The excess pore pressure ratio ($r_{ua}$) was defined here as the ratio of the difference of pore water pressure in a specified stage and initial pore water pressure over the vertical effective stress to detect the occurrence of soil liquefaction.

Under 0.1 g shaking intensity, Figure 7 shows the time history curves of excess pore pressure ratio directly under the buildings (P1, P2, P3, and P4) of coral sand and Fujian sand. The signal of pore water pressure gauge was lost at P4 position in Fujian sand site. During the period of shaking (10 s), the excess pore pressure ratio of two kinds of sand sites gradually increased and the growth rate of coral sand was significantly less than that of Fujian sand. After 2 s of shaking, the excess pore pressure ratio of coral sand at P1 position was approximately 0.02, which of Fujian sand was about 0.04, and the excess pore pressure ratio of coral sand was less than that of Fujian sand. During the whole shaking period, Table 4 shows the peak values of excess pore pressure ratio. With the decrease of depth, the peak values of the two kinds of sand sites gradually increased. The peak values of excess pore pressure ratio of coral sand were less than that of Fujian sand. From top to bottom (P1–P3), the peak values of coral sand were about 0.86, 0.67, and 0.80 times of that of Fujian sand.

![Figure 7](image-url)

**Figure 7.** Excess pore pressure ratio time history curves under 0.1 g shaking intensity: (a) P1 position; (b) P2 position; (c) P3 position; and, (d) P4 position.

| Shaking Intensity | Sand Type      | P1  | P2  | P3  | P4   | P5   | P6   | P7   | P8   |
|-------------------|----------------|-----|-----|-----|------|------|------|------|------|
| 0.1 g             | Coral sand     | 0.06| 0.04| 0.04| 0.03 | 0.08 | 0.04 | 0.04 | 0.03 |
|                   | Fujian sand    | 0.07| 0.06| 0.05| Lost | 0.09 | 0.06 | 0.04 | 0.04 |
| 0.2 g             | Coral sand     | 0.94| 0.68| 0.62| 0.5  | 1.1  | 0.72 | 0.59 | 0.54 |
|                   | Fujian sand    | 1.2 | 1.09| 0.98| Lost | 1.5  | 1.24 | 1.12 | 0.81 |

Table 4. Comparison of peak excess pore pressure ratios.

Figure 8 shows the time history curves of excess pore pressure ratio under 0.2 g shaking intensity, and the signal of pore water pressure gauge was lost at P4 position in the Fujian sand site. The development patterns of the excess pore pressure ratio of the two kinds of sand sites were the same with time during the shaking period (10 s). The excess pore pressure ratio reached peak value after a sharp increase of about 4 s, and then began to decrease. The growth rate of excess pore pressure ratio of coral sand was less than that of Fujian sand, when the excess pore pressure ratio reached 0.5 at the P1 position, the coral sand uses 2.73 s, and the Fujian sand uses 2.22 s. Table 4 shows the peak values of the excess pore pressure ratio of two kinds of sand sites. The peak values of excess pore pressure ratio of coral sand were less than that of Fujian sand. From top to bottom (P1–P3), the peak values of excess pore pressure ratio of coral sand were 0.78, 0.62, and 0.63 times of that of Fujian sand. With the
increase of depth, the peak values of the excess pore pressure ratio of two kinds of sand sites gradually decreased. The liquefaction degree of coral sand site is less than that of the Fujian sand site.

**Figure 8.** Excess pore pressure ratio time history curves under 0.2 g shaking intensity: (a) P1 position; (b) P2 position; (c) P3 position; and, (d) P4 position.

### 4.2. Acceleration Response

Figure 9 shows the acceleration time history curves of the coral sand and Fujian sand site. Under 0.1 g shaking intensity, the shape of acceleration time history curves of two kinds of sand sites was similar to that of the input sinusoidal wave excitation, which indicated that the soil was basically in an elastic state, and there was a near-linear amplification effect on the input sinusoidal wave excitation. The acceleration amplification factors got larger when the depth decreased, as shown in Figure 10. The acceleration amplification factors of coral sand were less than that of Fujian sand, which were approximately 0.54–0.90 times of that of Fujian sand. The difference of peak values of the acceleration between two kinds of sand sites was the greatest at the A1 position and the smallest at the A3 position.

**Figure 9.** Acceleration time history curves: (a) A1 position under 0.1 g intensity; (b) A2 position under 0.1 g intensity; (c) A3 position under 0.1 g intensity; (d) A1 position under 0.2 g intensity; (e) A2 position under 0.2 g intensity; and, (f) A3 position under 0.2 g intensity.
acceleration excitation with the onset of liquefaction. Under 0.1 g shaking intensity, the variation law of horizontal displacement was in a high pore pressure state at this time, but there was no obvious liquefaction phenomenon, the attenuation effect of soil on input acceleration excitation was less obvious than that of Fujian sand. Figure 10 shows acceleration amplification factors at different depths of two kinds of sand sites. Acceleration amplification factors increased with the decrease of depth, which is consist with the acceleration response of general liquefaction sites. The acceleration amplification factors of coral sand were less than that in Fujian sand site, which was approximately 0.79–0.90 times of that of Fujian sand.

Figure 11 shows the spectrum analysis with a damping ratio of 0.05 for the white noise at the A1 position. The dominant frequencies of the coral sand and Fujian sand sites were 10 Hz before the sinusoidal wave excitation, and the spectrum distribution of two kinds of sand sites was similar, which indicated that the initial state of two kinds of sand sites was close. After 0.2 g sinusoidal wave excitation, the high frequency component attenuation and low frequency component amplification occurred in both coral sand and Fujian sand sites although the dominant frequency of the two kinds of sand sites were still 10 Hz, which illustrated that the two kinds of sand sites had softened and the stiffness of model foundation was reduced when compared with the initial state.

![Figure 10](image1.png)

**Figure 10.** Acceleration amplification factors at different depth.

Under 0.2 g shaking intensity, the acceleration of Fujian sand gradually increased with time, reaching peak values at about 4.5 s, and then suddenly decreased. The acceleration change of coral sand with time was not that obvious, like Fujian sand. The shape of time history curves of coral sand was basically similar to that of input sinusoidal wave excitation. The difference of shape of acceleration time history curves between coral sand and Fujian sand was due to the liquefaction of the Fujian sand site. The shear strength of Fujian sand site sharply decreased, and the soil had an obvious attenuation effect on input acceleration excitation with the onset of liquefaction. The coral sand site was in a high pore pressure state at this time, but there was no obvious liquefaction phenomenon, the attenuation effect of soil on input acceleration was less obvious than that of Fujian sand. Figure 11 shows acceleration amplification factors at different depths of two kinds of sand sites. Acceleration amplification factors increased with the decrease of depth, which is consist with the acceleration response of general liquefaction sites. The acceleration amplification factors of coral sand were less than that of Fujian sand, which were approximately 0.79–0.90 times of that of Fujian sand.

![Figure 11](image2.png)

**Figure 11.** Fourier analysis of white noise: (a) coral sand; and, (b) Fujian sand.
4.3. Displacement Response

Figure 12 shows the horizontal displacement time history curves of buildings in coral sand and Fujian sand sites. Under 0.1 g shaking intensity, the variation law of horizontal displacement oscillation amplitude of buildings in the two kinds of sand sites was basically similar with time. Both of the building horizontal displacements of coral sand and Fujian sand experienced a rapid increase in a short time (about 1.5 s) and remained stable. The variation law of building oscillation amplitude with time was consistent with the input sinusoidal wave excitation. At this time, there was no liquefaction in the two kinds of sand sites, no obvious reduction of soil stiffness and shear strength, and the horizontal displacement oscillation amplitude of the building changed with the input sinusoidal wave excitation. When the shaking ended, the horizontal displacement of building in coral sand site was 0.02 mm, and that in the Fujian sand site was 0.22 mm. The horizontal displacement in coral sand site was less than that in Fujian sand site, which was approximately 0.09 times of that in the Fujian sand site.

![Horizontal displacement time history curves of building.](image)

The horizontal displacement of buildings in the two kinds of sand sites changed with time similarly under 0.2 g shaking intensity, while the horizontal displacement oscillation amplitude in the coral sand site was obviously smaller than that in the Fujian sand site. The horizontal displacement oscillation amplitude in the two kinds of sand sites decreased abruptly at around 4.8 s, at this moment, the excess pore pressure ratio reached its peak value (Figure 8). The coral sand site was in high pore pressure state, and the research of Chen et al. [37] showed that the soil under that condition exhibited shear thinning non-Newtonian fluid characteristics, even if the soil did not liquefy, so the attenuation
of horizontal displacement oscillation amplitude of buildings was related to a certain reduction of soil to input shaking excitation and horizontal force of pile that was caused by stiffness degradation of the coral sand site. The attenuation of building oscillation amplitude in the Fujian sand site was due to the sharp decrease of shear strength of soil that was caused by liquefaction. When shaking ended, the horizontal displacement of building in coral sand site was 0.53 mm, and that in Fujian sand site was 2.96 mm. The horizontal displacement of building in coral sand site was less than that in Fujian sand site, which was approximately 0.18 times of that in the Fujian sand site.

Figure 13 shows the horizontal displacement time history curves of buildings in coral sand and Fujian sand sites. Under 0.1 g shaking intensity, the building settlement in two kinds of sand sites increased slowly with time. The building settlement in coral sand site was 0.07 mm, which in the Fujian sand site was 0.43 mm. The building settlement in coral sand site was less than that in the Fujian sand site, which was about 0.16 times of that in Fujian sand site. Under 0.2 g shaking intensity, the building settlement in coral sand site increased linearly with time. The building settlement development trend in the Fujian sand site was similar to that in coral sand site within 4.8 s from the beginning of shaking, while the building settlement rate in Fujian sand site suddenly increased at 4.8 s, when compared with the results of the excess pore pressure ratio of Fujian sand, as illustrated in Figure 8, it could be seen that the excess pore pressure ratio of Fujian sand reached 1 at this time and the soil was in the initial liquefaction state. The effective stress between the soil particles of Fujian sand was close to 0, which led to the soil having almost no shear strength and the bearing capacity of the foundation decreasing, consequently, the building subsided sharply. After shaking, the building settlement in coral sand site was 1.29 mm, which in the Fujian sand site was 7.98 mm. The building settlement in coral sand site was less than that in Fujian sand site, which was approximately 0.16 times of that in the Fujian sand site.

![Figure 13. Settlement time history curves of building.](a) 0.1 g shaking intensity

![Figure 13. Settlement time history curves of building.](b) 0.2 g shaking intensity

4.4. Dynamic Bending Moment Response

The bending moment of the column and pile foundation is obtained by the following equation:

\[
M = \varepsilon_t \cdot h - \varepsilon_c \cdot h \quad (2)
\]

where \(M\) is the bending moment, \(\varepsilon_t\) and \(\varepsilon_c\) are tensile and compressive strain, respectively, \(h\) is the length of the section side for square cross-section columns, and \(h\) is the diameter of the pile for circular cross-section piles.
4.4. Dynamic Bending Moment Response

The bending moment of the column and pile foundation is obtained by the following equation:

\[ M = \frac{E I (\varepsilon_t - \varepsilon_c)}{h} \]  

(2)

where \( M \) is the bending moment, \( \varepsilon_t \) and \( \varepsilon_c \) are tensile and compressive strain, respectively, \( h \) is the length of the section side for square cross-section columns, and \( h \) is the diameter of the pile for circular cross-section piles.

Figure 14 shows the peak values of the dynamic column bending moments in the coral sand and Fujian sand sites. The dynamic column moments in two kinds of sand sites were the largest at the bottom of column, followed by the second story column, which was consistent with the general law of dynamic moment response of building columns under earthquake. Under 0.1 g shaking intensity, the peak column moments in the coral sand site were smaller than that in the Fujian sand site. From top to bottom (S1–S4), the peak column moments in coral sand site were approximately 0.98, 0.88, 0.82, and 0.98 times of that in Fujian sand site. The peak column moments in the coral sand site were also smaller than that in Fujian sand site under 0.2 g shaking intensity. When compared with 0.1 g shaking intensity, the dynamic column moments in the coral sand site increased by 3.11–3.61 times, and by 4.41–5.93 times in Fujian sand site under 0.2 g shaking intensity. The dynamic moment amplification effect of building columns in coral sand site was smaller than that in Fujian sand site when the shaking intensity increased.

Figure 14. Peak column bending moments at different height: (a) 0.1 g shaking intensity; and, (b) 0.2 g shaking intensity.

Figure 15 shows the peak values of the dynamic pile moments in coral sand and Fujian sand sites. The dynamic moments of corner pile, edge pile, and center pile similarly varied with buried depth. The peak values of moments were the largest at the top and the smallest at the bottom of pile. When 0.1 g shaking excitation was input, the peak moments of corner piles in coral sand site were less than that in the Fujian sand site, which were approximately 0.69–0.94 times of that in the Fujian sand site. The peak moments of edge piles and center piles in coral sand site were also less than that in Fujian sand site, respectively. When 0.2 g shaking excitation was input, the moments of pile groups in coral sand site were less than that in Fujian sand site, which were about 0.62–0.93 times of that in Fujian sand site.
Acknowledgments:
The authors appreciate the assistance of our group members in the experiments.

Author Contributions:
Conceptualization, Q.W. and X.D.; Data curation, Q.W.; Formal analysis, Y.Z.; Funding acquisition, X.D.; Investigation, X.D.; Methodology, Q.W.; Project administration, X.D.; Resources, X.D.; Supervision, X.D.; Validation, Q.W., Y.Z. and Z.C.; Visualization, Y.Z.; Writing—original draft, Q.W.; Writing—review & editing, Z.C. All authors have read and agreed to the published version of the manuscript.

Funding:
This research was funded by National Natural Science Foundation of China (grant number 51622803, grant number 41831282 and grant number 51878103).

Acknowledgments:
The authors appreciate the assistance of our group members in the experiments.

Conflicts of Interest:
The authors declare no conflict of interest.

5. Summary and Conclusions
Shaking table tests of three-story buildings with nine-pile foundation in the coral sand and Fujian sand sites were carried out in this research. The similarities and differences of dynamic responses of coral sand and Fujian sand sites were studied through testing and analyzing the physical quantities, such as pore water pressure, acceleration, displacement, and dynamic bending moment. The following conclusions are drawn:

1) The peak values of excess pore pressure ratio of coral sand and Fujian sand were far less than 1 under 0.1 g shaking intensity, there was no liquefaction in two kinds of sand sites. The peak values of excess pore pressure ratio of coral sand were basically less than that of Fujian sand, which were approximately 0.67–1.00 times of that of Fujian sand.

2) The development rate of excess pore pressure ratio of coral sand was smaller than that of Fujian sand and the peak values of excess pore pressure ratio of coral sand were less than that of Fujian sand, which were about 0.53–0.78 times of that of Fujian sand. The coral sand sites were more difficult to liquefy than Fujian sand sites under the same relative density and similar grain-size distributions.

3) The acceleration amplification coefficient of coral sand and Fujian sand sites increased with the decrease of depth, and the acceleration amplification factors of coral sand was smaller than that of Fujian sand.

4) The building horizontal displacement in the coral sand site was smaller than that in the Fujian sand site. The building horizontal displacement in coral sand site was 0.09 times of that in the Fujian sand site under 0.1 g shaking intensity and 0.18 times of that in Fujian sand site under 0.2 g shaking intensity. The building settlement in coral sand site was also smaller than that in the Fujian sand site.

5) The dynamic bending moments of building columns in coral sand site were smaller than that in the Fujian sand site, and the magnification effect of increasing shaking intensity on the building column moment in coral sand site was smaller than that in the Fujian sand site. The peak values of dynamic bending moments of pile groups in the coral sand site were smaller than that in the Fujian sand site. The buildings in coral sand will withstand earthquakes better than buildings in silica sand.

**Figure 15.** Peak pile bending moments at different depth: (a) corner pile; (b) edge pile; and (c) center pile.
25. Luan, L.; Ding, X.; Zheng, C.; Kouretzis, G.P.; Wu, Q. Dynamic response of pile groups subjected to horizontal loads. *Can. Geotech. J.* 2019. [CrossRef]

26. Lv, Y.; Liu, J.; Zuo, D. Moisture effects on the undrained dynamic behavior of calcareous sand at high strain rates. *Geotech. Test. J.* 2018, 42, 725–746. [CrossRef]

27. Lv, Y.; Wang, Y.; Zuo, D. Effects of particle size on dynamic constitutive relation and energy absorption of calcareous sand. *Powder Technol.* 2019, 356, 21–30. [CrossRef]

28. Ni, P.; Song, L.; Mei, G.; Zhao, Y. Predicting excavation-induced settlement for embedded footing: Case study. *Int. J. Geomech.* 2018, 18, 5018001. [CrossRef]

29. Cui, C.Y.; Meng, K.; Wu, Y.J.; Chapman, D.; Liang, Z.M. Dynamic response of pipe pile embedded in layered visco-elastic media with radial inhomogeneity under vertical excitation. *Geomach. Eng.* 2018, 16, 609–618. [CrossRef]

30. Wu, W.; Liu, H.; Yang, X.; Jiang, G.; El Naggar, M.H.; Mei, G.; Liang, R. New method to calculate apparent phase velocity of open-ended pipe pile. *Can. Geotech. J.* 2020, 57, 127–138. [CrossRef]

31. Iai, S. Similitude for shaking table tests on soil-structure-fluid model in 1g gravitational field. *Soils Found.* 1989, 29, 105–118. [CrossRef]

32. Chen, S.; Tang, B.; Zhao, K.; Li, X.; Zhuang, H. Seismic response of irregular underground structures under adverse soil conditions using shaking table tests. *Tunn. Undergr. Space Technol.* 2020, 95, 103145. [CrossRef]

33. Zhou, Z.; Lei, J.; Shi, S.; Liu, T. Seismic response of aeolian sand high embankment slopes in shaking table tests. *Appl. Sci.* 2019, 9, 1677. [CrossRef]

34. Kheradi, H.; Morikawa, Y.; Ye, G.; Zhang, F. Liquefaction-Induced Buckling Failure of Group-Pile Foundation and Countermeasure by Partial Ground Improvement. *Int. J. Geomech.* 2019, 19, 4019020. [CrossRef]

35. Ebeido, A.; Elgamal, A.; Tokimatsu, K.; Abe, A. Pile and Pile-Group Response to Liquefaction-Induced Lateral Spreading in Four Large-Scale Shake-Table Experiments. *J. Geotech. Geoenviron. Eng.* 2019, 145, 4019080. [CrossRef]

36. Teparaksa, J.; Koseki, J. Effect of past history on liquefaction resistance of level ground in shaking table test. *Geotech. Lett.* 2018, 8, 256–261. [CrossRef]

37. Chen, Y.; Liu, H.; Shao, G.; Zhao, N. Laboratory tests on flow characteristics of liquefied and post-liquefied sand. *Chin. J. Geotech. Eng.* 2009, 31, 1408–1412.

© 2020 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (http://creativecommons.org/licenses/by/4.0/).