Research Article

Seismic Damage Behavior of Aeolian Sand Concrete Columns with an Inner Square Steel Tube

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In order to promote the application of aeolian sand in steel-concrete composite structures, the aeolian sand concrete columns with an inner square steel tube is proposed in this paper. This kind of column is composed of aeolian sand concrete, reinforcing steel, and an inner square steel tube. The seismic damage behavior of the column was studied through cyclic loading test and damage analysis on seven specimens with different structural forms. The seismic damage indices of the specimens in this study include the failure mode, bearing capacity, ductility, stiffness, hysteresis behavior, and energy dissipation. Then, a damage model of this kind of column is proposed. The study results show that installing an inner square steel tube can significantly improve the seismic damage behavior of aeolian sand concrete columns. This mode of construction can be used to enhance the replacement percentage of aeolian sand. In addition, the damage model proposed in this paper agrees well with the experimental results and can be used to evaluate the damage degree of the aeolian sand concrete columns with an inner square steel tube.

1. Introduction

In more than half a century, many countries and regions in the world will be seriously threatened by desertification, whose trigger is aeolian sand. Meanwhile, enormous quantities of ordinary engineering sand will be needed to fuel the increase in infrastructure construction taking place all over the world. In order to reduce the spending and environmental hazards resulting from the mass usage of ordinary engineering sand, many studies have been carried out on aeolian sand concrete. Chen et al. [1] investigated the effects of the water-binder ratio, fly ash content, and desert sand replacement rate on the compressive strength of high-strength desert sand concrete. The results show that it is feasible to make high-strength desert sand concrete, with the best relative replacement percentage of desert sand being 30%. Dong et al. [2] carried out freeze-thaw cyclic testing of concrete specimens mixed with aeolian sand. The test results showed that the mixing of aeolian sand could reduce freeze-thaw damage in concrete, with the best effective aeolian sand replacement percentage ranging from 20% to 30%. Taryal and Chowdhury [3] studied the water-binder ratio, compressive strength, and shrinkage behavior of concrete specimens mixed with desert sand. The study results showed that the water consumption of concrete increased with the addition of desert sand. In addition, the compressive strength of the concrete will not decrease when the replacement percentage of desert sand is controlled from 0% to 40%. Abudou [4] produced a batch of desert sand reinforced concrete beams where ordinary engineering sand was partially replaced by desert sand. These specimens were subjected to normal section failure tests, with the results showing that the desert sand concrete beams have sufficient load-carrying capacity to be applied in engineering construction. Wang et al. [5] investigated the seismic behavior of desert sand concrete columns and common concrete columns through cyclic loading tests. The skeleton curve, hysteretic curve, energy dissipation, ductility, and stiffness of
the two kinds of specimens were compared and investigated. The experiment results showed that the energy dissipation capacity of the desert sand concrete columns is higher than the ordinary concrete columns. This result provided a test basis for revealing the seismic mechanism of desert sand concrete columns.

Although many researchers have done a lot of research on the mechanical properties of aeolian sand concrete, there are very few studies on the seismic behavior of aeolian sand concrete components. Therefore, it is necessary to continue to develop the application of aeolian sand in steel-concrete composite structures. This paper proposes the development of an aeolian sand concrete column which consists of aeolian sand concrete, reinforcing steel, and an inner square steel tube. The seismic damage behavior of the columns was studied using cyclic loading tests and damage analysis on seven specimens with different structural forms.

2. Specimens

2.1. Specimen Design. The specimens include seven columns with the same geometric dimensions, shear span ratios, and axial compression ratios. The specimen numbers are PC1, SC1, SC2, SC3, TSC1, TSC2, and TSC3, respectively. Table 1 shows the characteristics of the specimens. As shown in Figures 1 and 2, the size and reinforcement details are the same for all the specimens. The main difference in the specimens is that there are square steel tubes installed in the middle of the reinforcement cages of specimens TSC1, TSC2, and TSC3. The external size of the steel tubes is 100 mm × 100 mm × 1300 mm. The thickness of the steel tubes is 3 mm. Some stud connectors were evenly welded on the square steel tubes to ensure proper bonding between the concrete and steel, as shown in Figure 3.

Table 2 shows the components and mix proportions of the specimens. Considering the water absorption properties of the aeolian sand, a water-reducing agent was added to reduce water consumption. The cubic compressive strength of the concrete was obtained via a compression testing machine, and the mean of specimens is shown in Table 3. Table 4 shows the material performance of the steel tube and the reinforcing steel.

2.2. Column Specimen Loading. The experiments were carried out in the Key Laboratory of Civil Engineering Structure and Mechanics in the Inner Mongolia University of Technology. The loading device is shown in Figure 4. During the loading process, the vertical load is applied by a 200 T hydraulic jack, the axial pressure remains unchanged, and the horizontal low-cycle reciprocating load was applied by a 500 kN electrohydraulic servo actuator fixed on the reaction frame. Three LVDT displacement sensors are arranged at the top, middle, and bottom of the column to measure the displacement of the specimen. In order to record the test process conveniently, it is stipulated that the A side is pulled when the horizontal actuator is extended, the load and displacement directions are positive, and the B side is pulled when the horizontal actuator is contracted, and the load and displacement directions are negative.

A hybrid control method of force and displacement was adopted in this test [6, 7]. Before the specimens reach the yield stage, the force control method is adopted, the force control amplitude is 5 kN, and the cycle is three times per stage. When the specimens reach the yield point, the position control method is adopted, the value is an integer multiple of the yield displacement Δy, and the cycle is three times per stage. When the horizontal load drops to 85% of the maximum horizontal load, the test is terminated. The test live is shown in Figure 5.

3. Experimental Results

3.1. Experimental Observations

3.1.1. Column Specimen PC1. When the horizontal load reached 21.1 kN (the load when the specimen is cracked is shown in Table 5) and the load direction was positive, the first crack appeared in the tension zone, which was 120–210 mm away from the upper part of the foundation beam, and the drift ratio (the drift ratio is the displacement of the peak point of each cycle of the specimen divided by the height of the specimen) of the specimen was 0.006. With the increase in the horizontal load and cycle number, cracks gradually increased, but most of them appeared at the bottom of the column. When the specimens entered the yield state, there were few new cracks, the original cracks expanded obviously, the depth and length of the original cracks increased continuously, and the column foot concrete exhibited a spalling phenomenon. At the end of the test, a large amount of concrete can be observed falling from the foot of the column, and some longitudinal reinforcement bars were exposed and yielded. At this time, the drift ratio of the specimen was 0.039.

3.1.2. Column Specimen SC1. When the horizontal load reached 21.6 kN and the horizontal load direction was negative, the first crack appeared at the bottom of the specimen, and the drift ratio was 0.0054. With the increase in the horizontal load and cycle number, many new cracks had formed, and the original cracks continued to progress. When the specimens entered the yield state, there were few new cracks, and the concrete at the foot of the column had completely fallen off. At the end of the test, the concrete in the plastic hinge area was observed to have a large deformation; the drift ratio of the specimen at this moment was 0.042. At the same time, longitudinal reinforcement was exposed and bent.

3.1.3. Column Specimen SC2. When the horizontal load reached 23.2 kN and the horizontal load direction was positive, the first crack appeared at the bottom of the specimen. With the increase in the horizontal load and cycle number, many new cracks appeared, and the original cracks continued to expand. At the end of the test, similar to the specimen SC1, the concrete in the plastic hinge area had...
been crushed. Many concrete pieces were scattered on the surface of the foundation beam. The exposed longitudinal reinforcement also exhibited yielding.

3.1.4. Column Specimen SC3. The load when the specimen SC3 was cracked was 24.7 kN. With the increase in the horizontal load, cracks generally appeared in the lower half of the specimen. The original horizontal cracks expanded downward by 45 degrees, and the column foot concrete exhibited spalling. When the specimen failed, the concrete in the plastic hinge area was crushed, and many pieces fell off. The exposed longitudinal reinforcement was bent, and the drift ratio of the specimen at this moment was 0.0489.

| Specimens     | PC1 | SC1 | SC2 | SC3 | TSC1 | TSC2 | TSC3 |
|---------------|-----|-----|-----|-----|------|------|------|
| Axial compression ratio | 0.2 | 0.2 | 0.2 | 0.2 | 0.2  | 0.2  | 0.2  |
| Shear span ratio         | 3.5 | 3.5 | 3.5 | 3.5 | 3.5  | 3.5  | 3.5  |
| Replacement percentage of aeolian sand (%) | No  | 10  | 20  | 30  | 10   | 20   | 30   |
| Inner steel tube        | No  | No  | No  | No  | Yes  | Yes  | Yes  |

| Specimens     | PC1 | SC1 | SC2 | SC3 | TSC1 | TSC2 | TSC3 |
|---------------|-----|-----|-----|-----|------|------|------|
| Axial compression ratio | 0.2 | 0.2 | 0.2 | 0.2 | 0.2  | 0.2  | 0.2  |
| Shear span ratio         | 3.5 | 3.5 | 3.5 | 3.5 | 3.5  | 3.5  | 3.5  |
| Replacement percentage of aeolian sand (%) | No  | 10  | 20  | 30  | 10   | 20   | 30   |
| Inner steel tube        | No  | No  | No  | No  | Yes  | Yes  | Yes  |

3.1.5. Column Specimen TSC1. With the increase of horizontal load, more cracks appeared at the bottom of the specimen. Concurrently, existing cracks extended. When the force reached 89.2 kN, cracks appeared one-third of the distance from the top of the column at angles ranging from 0° to 35°. When the cyclic load continued to act, the concrete at the bottom of the column began to peel slightly. At the end of the test, it was found that the concrete at the bottom of the specimen had a spalling phenomenon, but the spalling degree was significantly less than that of the specimens PC1, SC1, SC2, and SC3. Concurrently, the bottom areas of both longitudinal reinforcement and the inner steel tube were bent and yielded, the drift ratio of the specimen at this moment was 0.0751.
3.1.6. Column Specimen TSC2. When the force reached 28.1 kN, a few tiny, mostly horizontal cracks created in the tension zone at the bottom area of the specimen. In the next loading cycle, a few cracks appeared a third of the distance from the bottom of the column. When the horizontal force increased, there were many inclined cracks in the middle part of the column, and the original cracks at the bottom of the column continued to extend. As the force reached 92.1 kN in the positive direction, a few cracks occurred at the upper part of column, and this can be attributed to the fact that the strength of the upper concrete of the specimen cannot meet the demand of the lateral force under long-time low cycle cyclic loading, even though the moment demand here is much less compared to the bottom of the column. Compared with the cracks of other parts, those in the lower part of the column were more dense. In the end, the bottom concrete of the column was found to have peeled. Concurrently, the bottom areas of the longitudinal reinforcement and the inner steel tube were found to be bent and yielded. The drift ratio of the specimen at this moment was 0.0785.

3.1.7. Column Specimen TSC3. As the force reached 29.5 kN, few tiny and narrow cracks were developed in the lower part of the column. As the horizontal load increased, a few cracks occurred in the middle of the column. On further increase in load, similar to the specimen TSC2, a few cracks were...
formed in the upper part of the column. When the test was under deformation control, the cracks at the bottom part of the column extended and intersected with the other cracks. In the end, it could be seen that the longitudinal reinforcement was bent and yielded. Concurrently, the tension side of the steel tube was torn. Compared with the specimens TSC1 and TSC2, the concrete peeling in the bottom concrete of the specimen TSC3 was less.

Failure details of the seven specimens are shown in Figure 6.

4. Analysis of the Specimen Performance

4.1. Hysteresis Behavior and Skeleton Curves. The hysteresis curves of the specimens are shown in Figure 7. The skeleton curves of the specimens are shown in Figure 8. At the beginning of the experiment, the specimens were in an elastic state, and the hysteresis loops returned to the origin without any residual deformation. As the force increased, the residual deformation sharply increased. At this stage, the specimens were in a plastic and nonrecoverable state. At the end of the tests, the load-carrying capacities of the specimens decreased as the deformation increased. The loading continued until the bearing capacity of the specimens dropped to 85% of the maximum horizontal load. At this time, the specimens were considered to have failed.

By comparing the hysteresis and skeleton curves for all specimens, it can be seen that the specimen TSC3 has the largest displacement and the maximum horizontal load. The hysteresis loops for the specimens PC1, SC1, SC2, and SC3 are narrower and not as plump as the specimens TSC1, TSC2, and TSC3. This result indicates that the specimens TSC1, TSC2, and TSC3 exhibit better ductility, bearing capacity, and energy dissipation capacity than other specimens. In addition, when the replacement percentage of aeolian sand is the same, the specimens with inner steel tubes have a better load-carrying capacity, displacement capacity, and energy dissipation capacity than the other specimens. Therefore, the combination of aeolian sand concrete columns with inner steel tubes remarkably improves the seismic damage behavior of support columns. In other words, the researchers can enhance the replacement percentage of aeolian sand by installing inner steel tubes without reducing the seismic damage behavior of aeolian sand concrete columns.

4.2. Characterized Points of Specimens. The characterized points measured during the test are shown in Table 5. In this table, \( F_{cr} \) is the load when cracks appear in the specimen, \( F_y \) is the load when the specimen enters the yield state, \( F_{max} \) is the maximum value of lateral load during the experiment, \( \Delta_{cr} \) is the displacement of the specimens when cracks begin to appear, \( \Delta_y \) is the displacement when the specimen enters the yield state, and \( \Delta_{max} \) is the displacement of the specimen when it reaches the maximum value of lateral load during the experiment. When the lateral load reaches 85% of the maximum lateral load, the specimen is considered to be failed, the load at this point is \( F_{cr} \), and the displacement at this point is \( \Delta_{cr} \).

Table 5 shows the displacement ductility coefficients of the specimens. The ductility coefficient is defined as the ratio of \( \Delta_y \) to \( \Delta_{cr} \). Researchers used different definitions to calculate the yield displacements; in this paper, the definition of equivalent elastoplastic energy absorption [8] is adopted, as shown in Figure 9.

From Tables 5 and 6, we see that the bearing capacities of the seven specimens successively increase. The bearing capacities of the specimens TSC1–TSC3 are markedly higher than the other specimens. For instance, \( F_{max} \) of the specimen TSC1 is 73.9% higher than that of the specimen SC1, \( F_{max} \) of the specimen TSC2 is 78.5% higher than that of specimen SC2, and \( F_{max} \) of the specimen TSC3 is 77.2% higher than that of SC3. In addition, the ductility coefficients and ultimate drifts of the specimens TSC1–TSC3 are markedly higher than the other specimens. For instance, the ductility coefficient of the specimen TSC1 is 16.1% higher than that of the specimen SC1, the ductility coefficient of the specimen TSC2 is 15% higher than that of the specimen SC2, and the ductility coefficient of the specimen TSC3 is 8.3% higher than that of SC3.

From this analysis, the following conclusions can be drawn. When the replacement percentage of aeolian sand is the same, the specimens with inner steel tubes have better bearing capacity and ductility than the other specimens. This phenomenon is due to the mechanical properties of the specimens being obviously improved by the installation of the inner steel tubes. First, the inner steel tubes and interior concrete play an essential role in resisting the lateral force. Second, the external reinforced concrete can provide a certain restraint on the inner steel tubes. Third, there is a hoop effect from the steel tube on the interior concrete; this causes the core concrete to exist in a three-phase
4.3. Accumulative Energy Dissipation. The amount of energy dissipated during the loading process is reflected by the area of the hysteresis loop. Thus, the accumulation of each hysteresis loop area is the cumulative energy dissipation of the specimen. The accumulative energy dissipation of the seven specimens is shown in Figure 10. Here, the accumulation energy dissipation of the specimens TSC1–TSC3 is significantly larger than the other specimens; this indicates that the installation of the inner steel tube can improve the energy dissipation capacity of the specimens.

4.4. Stiffness Degradation. The secant stiffness degeneration coefficient curves of the specimens are shown in Figure 11. The abscissa represents the drift ratio (the drift ratio is the displacement of the peak point of each cycle of the specimen divided by the height of the specimen) of the specimens. The ordinate represents the secant stiffness. Here, the secant stiffness [9] of all the specimens decreased as the drift increased. More significantly, the stiffness degradation velocity of the specimens TSC1, TSC2, and TSC3 is slower than that of the other specimens. As the force and displacement increase, the longitudinal reinforcing steel yields, which causes the secant stiffness of the specimens PC1–SC3 to decrease significantly. However, the specimens TSC1, TSC2, and TSC3, which have inner steel tubes, have sufficient stiffness reserves to resist the lateral loads; this results in a slower stiffness degradation. In summary, installing inner steel tubes can significantly improve the stiffness degradation behavior of the specimens.

5. Damage Analysis

Damage models can quantitatively describe the damage degree in the specimens. In this paper, two kinds of damage....
models are used to analyze the entire process of the seismic damage evolution of the specimens with inner steel tubes.

5.1. Single-Parameter Damage Model. First, this paper used a stiffness-based damage model revised by the American scholars Roufaiel and Meyer [10]. The expression for this model is

\[
D_1 = \frac{K_{x,i} - K_y}{K_{m} - K_y} 
\]

where \( K_y \) is the secant stiffness corresponding to the moment when the specimens are at the yielding point and \( K_m \) is the secant stiffness corresponding to the moment when the specimens are at the destruction point. When \( D_1 > 0 \), the specimens begin to incur damage. When \( D_1 = 1 \), the specimens have completely failed. However, Roufaiel assumes that there is no damage occurring in the specimens before yielding, which is inconsistent with the actual test results. In this paper, this model is only used as a reference basis for the two-parameter model. Table 7 shows the calculated results of \( D_1 \) for the specimens TSC1–TSC3; these 4 data correspond to various drift ratios.

5.2. Two-Parameter Damage Model. The entire test process shows that the stiffness degradation reflects the damage degree of the specimens. Some scholars [11] have verified the
The rationality of stiffness degradation as a damage parameter. However, the stiffness degradation cannot reflect the damage degree of the elements completely. Therefore, Wu et al. [12] considered that the stiffness degradation reflects the passage damage of the elements, and the hysteretic energy dissipation reflects the cumulative damage of the elements; the S-E (stiffness degradation and energy dissipation) damage model was proposed. This model uses the stiffness degradation as a failure index to reflect the maximum force response of the elements. Simultaneously, the cumulative hysteretic energy is used as a reflection of the damage indicator of the elements. The formula for this is as follows:

\[
D_2 = \frac{K_o - K_i}{K_o} + \beta \times \frac{\sum E_i}{E_u} \times \frac{K_i}{K_o} \tag{2}
\]

where \(K_o\) is the secant stiffness corresponding to the moment when the structural elements are at the yielding point, \(K_i\) is the secant stiffness corresponding to the peak displacement point of the elements, \(E_i\) is the cumulative hysteretic energy of the elements for each loading segment, \(E_u\) is the ultimate hysteretic energy of the elements under monotonic loading, and \(\beta\) is the energy dissipation factor and is fitted by the physical geometry parameters of the elements. The variable \(\beta\) can be calculated from

\[
\beta = 0.045 - 0.042 \times \frac{\delta}{\ln \lambda} + \frac{0.071}{1 + [(n/\ln \lambda) - 0.176]/0.021}]^2 \tag{3}
\]
where \( \lambda \) is the slenderness ratio of the elements, \( n \) is the axial compression ratio of the elements, and \( \delta \) is the component hoop coefficient, which can be calculated from

\[
\delta = \frac{f_sA_s}{f_cA_c},
\]

(4)

where \( f_s \) is the yield strength of the steel tube, \( f_c \) is the compressive strength of the filled concrete, \( A_s \) is the cross-sectional area of the steel tube, \( A_c \) is the cross-sectional area of the concrete, and \( \lambda \) is the shear span ratio.

Using equations (2)–(4), the calculation results \( D_2 \) cannot reflect the actual situation of the test, so the model needs to be revised. First, the damage index corresponding to the end of the test is set at 1; that is, equation (2) is equal to 1, and the secant stiffness and the cumulative hysteresis energy value are substituted to calculate the \( \beta \) value. Second, according to the mathematical model of equation (3), the statistical fitting analysis is carried out on the obtained \( \beta \) value and the axial compression ratio, slenderness ratio, and hoop coefficient of the specimen. The revised expression for parameter \( \beta \) is

\[
\beta = 0.055 - 0.042 \times \frac{\delta}{\ln 3.5} + \frac{0.802}{1 + [((n/\ln \lambda) - 0.176)/0.021]^2}.
\]

(5)

Using equations (2), (4), and (5), the calculation results for \( D_2 \) are shown in Table 7. Compared with the single-parameter damage model, the two-parameter damage model considers the effect of cumulative damage on the specimens. Therefore, using this model to analyze the damage process of the aeolian sand concrete columns with inner steel tubes is more accurate.

**6. Conclusions**

The aeolian sand concrete column with an inner square steel tube is proposed in this paper. The seismic damage behavior of the column was studied through cyclic loading test and damage analysis. The following conclusions can be drawn:

(1) For the same load, the damage degree of the aeolian sand concrete specimens with an inner steel tube is less than that of other specimens.

(2) The study results show that installing an inner steel tube can achieve a significant improvement in ductility, hysteresis behavior, stiffness degradation, load-carrying capacity, and energy dissipation capacity of an aeolian sand concrete column. Therefore, the researchers can enhance the replacement percentage of aeolian sand by installing inner steel tubes without compromising the seismic damage behavior of aeolian sand concrete columns.

(3) The damage model revised in this paper is more consistent with the damage process associated with the specimens in these tests. It is feasible to use this model to analyze the entire damage process for an aeolian sand concrete column with inner steel tubes.

**Data Availability**

The data, models, and code generated or used to support the findings of this study are included in the article.

**Conflicts of Interest**

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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**Supplementary Materials**

(1) Skeleton curves: the file named “Coordinates of skeleton curve” shows the coordinate points of skeleton curves corresponding to Figure 8. In this file, “F” represents the lateral load and “A” represents the displacement. (2) Accumulative energy dissipation: the file named “Coordinates of accumulative energy dissipation” shows the coordinate points of cumulative energy consumption corresponding to Figure 10. In this file, “E” represents cumulative energy consumption and “A” represents displacement. (3) Stiffness degradation: the file named “Coordinates of secant stiffness” shows the coordinate points of secant stiffness corresponding to Figure 11. (Supplementary Materials)

**References**

[1] Y. L. Chen, J. R. MA, H. F. Liu, and J. X. Song, “Influence of fly ash and desert sand content on the compressive strength of high strength concrete,” *Concrete*, vol. 7, no. 7, pp. 80–84, 2014.

[2] W. Dong, X.-d. Shen, H.-j. Xue, J. He, and Y. Liu, “Research on the freeze-thaw cyclic test and damage model of Aeolian sand lightweight aggregate concrete,” *Construction and Building Materials*, vol. 123, pp. 792–799, 2016.

[3] M. S. Taryal and M. K. Chowdhury, “Effect of superfine sand on workability and compressive strength of cement mortar and concrete,” *International Journal for Housing Science and Its Applications*, vol. 7, no. 2, pp. 165–174, 2012.

[4] Abudou, *Experimental Study on Flexural Behavior of Desert Sand Concrete Beams*, Xinjiang University, Ürümqi, China, 2015.

[5] G. Q. Wang, Z. Q. Li, S. Yang, and G. N. Ju, “Experimental study of the desert sand concrete frame columns under low cyclic loading,” *Concrete*, vol. 6, no. 6, pp. 18–21, 2018.

[6] W. Zhu, J. Jia, J. Gao, and F. Zhang, “Experimental study on steel reinforced high-strength concrete columns under cyclic lateral force and constant axial load,” *Engineering Structures*, vol. 125, pp. 191–204, 2016.

[7] S. Zhen, Q. Qin, Y. Zhang, L. Zhang, and W. Yang, “Research on seismic behavior and shear strength of SRHC frame columns,” *Earthquake Engineering and Engineering Vibration*, vol. 16, no. 2, pp. 349–369, 2017.

[8] S. A. Mahin and V. V. Bertero, “Problems in establishing and predicting ductility in asismatic design,” in *Proceedings of the International Symposium on Earthquake Structural Engineering*, p. 613, St Louis, MO, USA, August 1976.
[9] Y. H. Wang, Z. Y. Gao, Q. Han, L. Feng, H. Su, and N. Zhao, “Experimental study on the seismic behavior of a shear wall with concrete-filled steel tubular frames and a corrugated steel plate,” *The Structural Design of Tall and Special Buildings*, vol. 27, no. 15, p. e1509, 2018.

[10] M. S. L. Roufaiel and C. Meyer, “Analytical modeling of hysteretic behavior of R/C frames,” *Journal of Structural Engineering*, vol. 113, no. 3, p. 429, 1978.

[11] H. X. Yu, J. H. Wu, and G. B. Zhang, “A new earthquake damage model for RC structure,” *Journal of Chongqing Jianzhu University*, vol. 10, 2004.

[12] Y. Wu, J. M. Huang, W. Lee et al., “Stiffness degradation and hysteretic energy dissipation based damage model of concrete-filled circular steel tube columns,” *Earthquake Engineering and Structural Dynamics*, vol. 5, pp. 172–179, 2014.
