Research Article

Experimental Study on the Seismic Performance of Socket Bridge Piers

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In order to accelerate the construction of bridge substructure, a socket joint construction that does not require interfaces roughening between the precast columns and the reserved cavity of the precast foundation is raised in this paper. The seismic performance of such fabricated bridge piers was investigated by carrying quasistatic tests on socket circular pier specimens of different embedment depths with a compared cast-in-place pier specimen. The experimental results showed that the prefabricated piers with the embedment length larger than 1.0 times the column diameter, featuring smooth interfaces that was free of roughening, had a failure mode of bending damage as well as the cast-in-place component. As the embedment depth increases, the seismic performance indexes of the socket bridge pier, including bearing capacity, ductility, and energy dissipation capacity, are improved. The seismic performance indexes of a socket bridge pier specimen with an embedment depth of 1.5 times the column's diameter in the test are better than the cast-in-place one.

1. Introduction

The design of socket bridge piers is such that a precast column inserted into a reserved cavity of a precast foundation. The gap between the cavity and the column is filled up with semihard concrete or high-strength nonshrinkage grouting materials, and no reinforcement connection is selected between the column and the foundation. Comparing with the traditional cast-in-place bridge pier, socket piers, whose components are prefabricated in plants and assembled on site, have the advantages of being able to decrease various noises made during construction, alleviate dust pollution, enormously reduce erection time, and mitigate the influence on urban traffic and the lives of surrounding residents. In addition to improving the safety of constructors and generating both economic and social benefits, the adoption of socket bridge piers is also beneficial for boosting the quality of the prefabricated components and ensuring the durability of bridges since no reinforcing bars cross the column-to-footing connection. If compared with splice-type construction that selects reinforcing bars as

connecting elements, such as grout sleeves and grout bellows, rather large construction errors are allowed for socket bridge piers on the one hand; and on the other hand, construction procedures of such piers are simple, not to mention that the relevant workload on site is rather small. For these reasons, this is a competitive solution to the rapid construction of bridge substructures [1–4]. For example, prefabricated bridge piers in such an assembly form have been utilized in SR 520 highway bridge of Washington State (America), US 12 bridge of no. 5 interstate highway, US 101 Bone River bridge, the ramp bridge for Jiamin Overhead Viaduct in Shanghai (China), and Dianqi River Bridge Modification Project of China's Beijing-Hong Kong-Macao Expressway [5].

A joint connection between the column and the foundation is usually subjected to the maximum bending moment and nonelastic deformation reversion under seismic loads. Therefore, it is the most susceptible to failure. Because of this reason, the seismic performance of fabricated bridge piers has always been an issue that attracts extensive attention in the field of bridge engineering. Outside China, the

seismic performance of socket construction has been investigated both experimentally and theoretically [6].

As early as in 1996, Japanese scholars Osanai et al. [7] have performed quasistatic tests on socket joint construction between the column and the foundation in the architectural structure. Not only the load transfer mechanism of columns with different embedment depths was analyzed but also a computational stress formula was derived for column-foundation socket connection under the action of a horizontal force and an axial force. However, those are for a specific shear bond structure and are not universally applicable. Since 2004, Canha et al. [8, 9] carried out a series of experimental studies and theoretical analyses on stress performance of socket joint between the column and the foundation of architectural structures. Specific to the socket joint construction with or independent of shear connectors, they presented corresponding analytical formulas successively. Apart from analyzing the influence of shear connectors' sizes on the piers' shear capacity, the maximum and minimum sizes of the shear connectors were recommended by them.

With the increase in demand for accelerated bridge construction, socket joint construction has been increasingly applied in constructions of bridge structures. Through comparisons between prefabricated bridge piers in a bridge structure and columns in an architectural structure, not only can the distinct differences in their dimensions and section forms be found (e.g., most bridge piers are circular with large cross-sections, while columns in architectural buildings are I-shaped or rectangular with small cross-sections in most cases) but also the dynamic loads acting on piers of the bridge structure are also much higher and more complex. Consequently, the pier-foundation socket joint construction of architectural structures cannot be directly applied in bridge engineering. For this reason, several specific forms of socket bridge piers have been proposed in combination with the local requirements of practical bridge applications by some scholars, and the static and seismic performances have also been preliminarily investigated and analyzed.

In 2006, Paolo [10] from Italy probed into the seismic performance of piers with socket joints and grout sleeve connection. During investigation, scabbling was fulfilled for the concrete surface of the embedment portion of the pier to make the surface rough enough. It was proven by relevant results that the hysteresis response of the socket pier was comparatively stable and no obvious strength degradation took place. In 2013, Haraldsson et al. [11–13] of the University of Washington conducted an experimental study on the seismic performance of socket prefabricated bridge pier with anchor head structure and the cast-in-place foundations. During the study, the column segment embedded into the foundation was an octagonal section with dents on the surface designed for the purpose of improving the mechanical behavior of the pier-foundation connection [14]. According to their experimental results, the failure of the structural components was caused by the failure of the column itself under the circumstance that the embedment depth of the column was greater than 1.1 times of its diameter; under the same scenario, the construction of the

socket joints remained intact. Mashal and Palermo [15–17] conducted an experimental study on the socket connection of the precast column and the precast foundation. In the test, the surface concrete of the precast column embedded into the precast foundation was chiseled, and the results showed that the seismic performance of the socket pier with an embedment depth of 1.0 times of the column diameter was similar to that of the monolithic pier. Mohebbi et al. [18] conducted shake table studies and analysis of a novel posttensioned precast bridge column connected to a precast footing with a pocket connection incorporating CFRP tendons and UHPC. Results showed that the pocket connection is effective in forming the plastic hinge in the column with no connection damage. Jones et al. [19] tested two large-scale models of bridge systems-utilized socket connection and pocket connection, respectively, on shake tables, and the results indicated that both exhibited satisfactory seismic behavior and are appropriate for use in high seismic areas.

With the goal of further accelerating the bridge construction and simplifying the procedures of assembly and construction, a socket joint construction that does not require roughening of the joint interface between the precast column and the precast foundation is raised in this paper. Through quasistatic tests on socket pier specimens of different embedment depths and a compared cast-in-place pier specimen, hysteresis curves were acquired and compared. From the view of bearing capacity, displacement ductility, energy dissipation property, and residual displacement, seismic performances of prefabricated piers with socket connection and cast-in-place piers were comparatively investigated. This research was performed with the expectation of providing a reference basis for the design optimization of socket bridge piers.

2. General Conditions of the Experiment

2.1. Model Design and Production. Considering that the literature [3, 11] suggested the embedment depth of the socket connection should be 1.1–1.5 times the cross-sectional size of the column, while the research of literature [20] pointed out that the seismic performance of the socket bridge pier is closest to that of the monolithic one when the embedment depth is 0.7 times the cross-sectional size of the column, three socket bridge pier specimens with embedment depths equal to $0.5D$, $1.0D$, and $1.5D$ (D refers to the diameter of the column), respectively, and one specimen of the cast-in-place pier were designed in the study. To facilitate the discussion later in this paper, these specimens were denoted as CC- $0.5D$, CC- $1.0D$, CC- $1.5D$, and ZT, respectively. The total heights of all specimens are 2.15 m. With regards to the cast-in-place pier specimen, its external dimension was same as that of CC- $1.0D$ (a socket pier specimen with an embedment depth equal to 1.0 times of the column diameter), and their only difference was the reinforcing bars in the foundations (no reinforcing bars were reserved to insert into the cavity), as shown in Figure 1. As for the three socket bridge piers, they were all columns with 1900 mm high and 300 mm diameter circular sections; their reinforcement

FIGURE 1: Dimensions and reinforcement drawings of the cast-in-place pier specimen (in mm).

distribution was entirely identical to each other, as shown in Figure 2(a). The symmetrical distribution of the longitudinal reinforcement consisted of 12 bars with a diameter of 12 mm along the circumference, resulting in a reinforcement ratio of 1.92%, which is typical for this type of piers in China. For the transverse reinforcement of the column, spiral stirrups with a diameter of 6 mm spaced 50 mm were adopted. To prevent local failure at the top of the precast pier due to stress concentration under the axial load, 10 mm thick round steel plates were provided on top of each column. All precast foundations have the same top view dimensions of 1300 mm $\times$ 750 mm, and their respective heights were 400 mm, 550 mm, and 750 mm, respectively. Diameters at the bottom and the top of circular reserved cavities in the corresponding foundations were 360 mm and 400 mm, respectively. With regards to the embedment depths, denoted by h_i , the columns were set as 180 mm, 330 mm, and 480 mm, respectively. Reinforcement drawings of the precast foundations are given in Figures 2(b) and 2(c).

The columns and the foundations were cast separately; after 28 days of curing, high-strength nonshrinkage grouting materials were used to connect them in a laboratory, as presented in Figure 3. To ensure the correct position and perpendicularity of the precast columns in the reserved cavity of the foundation, an independently designed positioning steel frame (composed of a base, a supporting component mounted on the base, connecting linkage provided on the support component and several jackscrews) was first erected around the preformed cavity in the precast foundation during the connection. After that, a laboratory crane was adopted to hoist the precast column and move it to the center of the steel frame. By adjusting the jack in length of the jackscrews on the positioning steel frame and taking certain measures such as manual rotation, the center of the column can be concentrically positioned in the reserved cavity in the foundation. Moreover, the perpendicularity of the column was observed by two total stations which had

been arranged orthogonally on a plane; all jackscrews were fastened after the column had been ensured to be completely perpendicular. Subsequently, high-strength nonshrinkage grouting materials (a cementitious grout with high strength, early strength, high self-flow, microexpansion, and nonshrinkage characteristics) were poured into the gaps of the cavities in which the foundation and the column had been connected to each other. Three days after curing of grouting materials, the positioning steel frame was dismantled. The general structural section of the assembled socket test specimen is shown in Figure 4.

2.2. Properties of the Material. Three reinforcing bars specimens with lengths equal to 600 mm were reserved at random for each rebar diameter during manufacturing the reinforcement skeletons of the column and the foundation. The average yield strength and tensile strength were obtained by the tension test, 499.9 MPa and 662.2 MPa for longitudinal reinforcing bars and 316.3 MPa and 449 MPa for transverse reinforcing bars, respectively.

While casting the pier specimens, six concrete blocks with the dimension of 150 mm $\times$ 150 mm $\times$ 150 mm were cast at the same time. When using high-strength nonshrinkage grout to connect the precast column and the precast foundation, six grout blocks with the dimension of 100 mm $\times$ 100 mm $\times$ 100 mm were made. The measured average test day strength of concrete and grout are listed in Table 1. Strength of concrete on the test day in all specimens was between 26.5 MPa and 28.8 MPa. The strength of grout on the test day in all specimens was between 48.7 MPa and 52.8 MPa.

2.3. The Test Scheme. The quasistatic test loading scheme was selected for this study. The loading method of force-displacement hybrid control was adopted in the experiment. A displacement sensor was placed opposite to the horizontal

FIGURE 2: Dimensions and reinforcement drawings of the socket pier specimens (in mm). (a) Precast column, (b) front view of the precast foundation, and (c) top view of the precast foundation.

FIGURE 3: Connection of precast column and precast foundation.

loading location on a column. Considering that the axial compression ratio (the axial compression ratio herein is defined as the ratio of the axial load to the product of the concrete compressive strength and the gross section area) generally ranges from 0.1 to 0.3 as far as the bridge pier is concerned [21], a 3000 kN servo oil jack was used to exert axial loads vertically on the top of the column according to the axial compression ratio of 0.2; once the axial load reached 202.2 kN, it stopped and remained unchanged. Horizontal cyclic loads were applied by a 500 kN tension-compression type jack in a mode of force-displacement hybrid control. Before the loading was initiated formally, preloading in three cycles was first carried out by virtue of a 5 kN horizontal force, and bolts of the component, the

FIGURE 4: General structural section of the assembled socket specimen.

linkage, and the actuator were all fastened to ensure firm connection. Afterward, gradation loading was performed at 5 kN, 10 kN, 15 kN, and so on; loading at each grade was repeated once before the specimen yielded. After the yield, the loading would be under control in line with the displacement. To be specific, the displacement increased at each grade would be a multiple of yield displacement; subjected to the same displacement, it would be repeated for 3 cycles. In addition, the test would not be ended until the horizontal bearing capacity of specimens dropped below 85% of the maximum horizontal load or until longitudinal bars ruptured. During the test, the yield was judged by the obvious

TABLE 1: Material properties.

Material property	ZT measured	CC-1.5D measured	CC-1.0D measured	CC-0.5D measured
Concrete compressive strength (MP)	26.5	27.6	28.3	28.8
Grout compressive strength (MP)	48.7	50.6	49.9	52.8

inflection point of the horizontal loading-displacement curves at the top of the pier. Besides, an isolated measurement pod (IMP) data collector was used to record test data. Based on these data, the hysteresis curves were generated.

3. Test Results and Discussion

3.1. Failure Modes. As shown during the test, the failure modes of socket bridge pier specimens with embedment depths equal to 1.0D and 1.5D were identical to that of the cast-in-place specimen. The failure occurred out the embedment region and with the yielding of the longitudinal reinforcement in the tension side. Since the bending failures were caused by the insufficient flexural capacity of the columns, they were characterized by the slow process going into the failure, while the grout connecting the precast column and the precast foundation showed no obvious damage. In the test, neither longitudinal bars nor transverse bars were fractured. In Figures 5(a)–5(d), the final failure conditions of the components are shown. As for the socket bridge pier specimen with an embedment depth of 0.5D, its failure mode significantly differed from that of other specimens. To be specific, only a few microcracks could be found on the surface of the precast column, and there were no obvious failures, such as concrete spalling. However, not only was the top surface of the foundation uplifted but also large amounts of radial microcracks were formed on the grouting material, which are shown in Figure 6. In terms of the longitudinal reinforcing bars of the specimen, they had not yielded when the specimen reached its maximum bearing capacity.

3.2. Hysteresis Curves. The load-displacement hysteresis curves for all test specimens are shown in Figure 7.

From Figure 7, it could be seen that the fullness of hysteresis curves for specimens CC-1.5D and CC-1.0D were similar to that of specimen ZT. With regard to specimen CC-0.5D, a pinching phenomenon can be observed from its hysteresis curves. This signifies that the energy dissipation capacity of the prefabricated pier may be similar to that of the cast-in-place one if its embedment depth is large enough. However, both ductility and energy dissipation capacity of the socket bridge piers decrease as the embedment depth increases. At a later stage of unloading, hysteresis curves of specimens CC-1.0D and CC-1.5D jittered. The possible reason may be that

grouting materials between the precast column and the precast foundation slipped slightly.

3.3. Backbone Curves. The backbone curves of the various specimens are presented in Figure 8, while the characteristic points of these backbone curves are listed in Table 2. In the table, both the yield load and yield displacement were defined by means of the Park method [22], while the ultimate load P_u and ultimate displacement were determined under the precondition that the maximum bearing capacity P_{max} declined by 15%, i.e., $P_u = 0.85P_{max}$.

From Figure 8 and Table 2, it is clear that

- (1) Despite the differences in construction approaches, backbone curves of specimen CC-1.0D (embedment depth equal to 1.0 times the column's diameter) bear a resemblance to those of the cast-in-place specimen. Compared to specimen ZT, specimen CC-1.0D performed slightly worse in ductility. Regarding their ductility coefficients, they were, respectively, 4.81 for the former and 4.47 for the latter, i.e., the ductility coefficient of specimen CC-1.0D is about 7.1% lower than that of specimen ZT. Considering that their ductility coefficients were both higher than 4, they can be said to have demonstrated good ductility. Furthermore, their peak loads were rather close to each other. To be specific, average peak loads of specimens CC-1.0D and ZT reached 70.69 kN and 71.78 kN, respectively, indicating that the former is only 1.5% lower than the latter.
- (2) As the embedment depth increased, the parameters of the socket specimens all increased, including their yield loads, yield displacements, peak loads, peak displacements, ultimate loads, ultimate displacements, and ductility coefficients; thereupon, they were capable of acclimatizing themselves to higher seismic displacement and greater seismic loads. It is can be seen from Table 2 that the above parameters of specimen CC-1.5D are better than that of specimen ZT. This implies that a reasonable embedment depth should be selected for the socket piers to postpone yielding and improve their bearing capacity, which is beneficial for their seismic performance and ductility improvement.
- (3) The peak displacements of all specimens during the pull loading were greater than those during the push loading. The reason is that push loading was carried out first during the test, causing the specimens to have been damaged before processing the pull loading. In addition, it can be seen that the yield displacement and ultimate displacement of the cast-in-place bridge pier are smaller than that of specimen CC-1.5D. This is because the load acting on the socket pier is transferred to the foundation via grout, and the compressive strength of the grout used in the test was much higher than that of the concrete. As mentioned in Section 2.2, the measured average compressive strength was 26.5 MPa to 28.8 MPa for

FIGURE 5: Final failures conditions of the specimens. (a) Specimen ZT. (b) Specimen CC-1.0D. (c) Specimen CC-1.5D. (d) Specimen CC-0.5D.

FIGURE 6: Radial microcracks of grouting material of specimen CC-0.5D.

the concrete and 48.7 MPa to 52.8 MPa for the grout. Since the compressive strength of the grout is higher than the concrete, as long as the embedment length is sufficient, the socket bridge pier got a stronger restraint effect on the column base to adapt to larger loads and displacements.

3.4. Energy Dissipation Capacity of the Specimens. The energy dissipation capacity of the specimen is generally measured by the graphic area enclosed by the load-displacement hysteresis curve envelope. The fuller the hysteresis curve envelope, the stronger the energy dissipation capacity and

the better seismic performance of the specimen. The energy consumption coefficient λ_D is introduced to indicate the energy consumption of the test specimens [23, 24], which is defined as follows:

$$\lambda_D = \frac{S_{ABC} + S_{CDA}}{S_{OBE} + S_{ODF}}, \quad (1)$$

where S_{ABC} and S_{CDA} represent the area of the upper and lower halves of the hysteresis curve, respectively; S_{OBE} and S_{ODF} represent the area of the triangle respectively, as shown in Figure 9.

The second hysteresis loop corresponding to the load level of the yield load, peak load, and ultimate load were

FIGURE 7: Load-displacement hysteresis curves of the test specimens. (a) Specimen ZT. (b) Specimen CC-1.0D. (c) Specimen CC-1.5D. (d) Specimen CC-0.5D.

FIGURE 8: Backbone curves of specimens.

selected to calculate the energy dissipation coefficient λ_D of each specimen, and the results are listed in Table 3.

From Table 3, the following summary can be made:

- (1) Energy dissipation coefficients of the peak hysteresis loops for all specimens were greater than those of their yield hysteresis loops, which were also the case for the relationship of energy dissipation coefficients between the ultimate hysteresis loops and the peak hysteresis loops. This proves that the energy dissipation capacity of the specimens enhances as the load increases.
- (2) The energy dissipation coefficients of socket specimens with different embedment depths can be arranged in the following order: CC-1.5D > CC-1.0D > CC-0.5D, which indicates that energy dissipation capacity improvement of these specimens can be observed with an increase in their embedment depths. This is because the vertical and horizontal loads acting on the socket pier top are transmitted to the foundation through the adhesion, friction, and

TABLE 2: Characteristic points of specimens.

Specimens no.		Yield load (kN)	Yield displacement (mm)	Peak load (kN)	Peak displacement (mm)	Ultimate load (kN)	Ultimate displacement (mm)	Ductility coefficient
ZT	Push	62.78	16.21	72.57	35.13	55.56	77.77	4.80
	Pull	-61.59	-16.75	-70.99	-42.2	-59.59	-80.75	4.82
CC-1.5D	Push	66.75	24.25	76.96	71.83	65.95	100.3	4.14
	Pull	-65.87	-20.05	-77.09	-78.98	-64.19	-110.6	5.52
CC-1.0D	Push	60.49	20.4	73.43	32.02	60.17	89.78	4.40
	Pull	-54.97	-19.65	-67.94	-35.08	-58.1	-89.16	4.54
CC-0.5D	Push	37.63	13.67	39.02	22.89	31.88	43.22	3.16
	Pull	-39.37	-14.62	-42.58	-28.26	-29.78	-44.62	3.05

FIGURE 9: Schematic diagram of energy consumption coefficient calculation.

TABLE 3: Energy dissipation coefficients of the specimens.

Specimens no.	ZT	CC-1.5D	CC-1.0D	CC-0.5D
Yield hysteresis loop	0.82	0.94	0.68	0.47
Peak hysteresis loop	1.30	1.69	0.86	0.81
Ultimate hysteresis loop	1.82	1.99	1.47	1.32

lateral resistance between the concrete interfaces of the column to grout and grout to foundation. The greater the embedment depth, the greater cohesion, friction, and lateral resistance are provided, so the greater the energy dissipation capacity of the pier as the embedment depth increases.

- (3) Energy dissipation coefficients for yield, peak, and ultimate hysteresis loops of specimen CC-1.0D were, respectively, 19.5%, 33.8%, and 19.2% lower than those of the cast-in-place specimen. Whereas, in terms of specimen CC-1.5D, the same coefficients were 14.6%, 30%, and 9.3% higher than those of the cast-in-place specimen. Thus, it is clear that socket specimens may perform as good as and even better than the cast-in-place one as long as the proper embedment depth has been set for the former.

3.5. Residual Displacement of Specimens. Residual deformation refers to the irrecoverable deformation of a structure or component after unloading, and it is important in seismic

FIGURE 10: Comparison chart of residual displacement for test specimens.

design. Some bridge codes such as Japanese require consideration of residual displacement in design [25]. The lower the residual deformation is, the more beneficial it is for postdisaster renovation and operation of bridges. Based on hysteresis curves of socket specimens with different embedment depths and the monolithic construction, the relationship curves can be portrayed for the residual displacement vs. the loading-displacement of the four test specimens, which are shown in Figure 10.

The following aspects are illustrated in Figure 10:

- (1) Once the test was completed, the residual displacement of specimen CC-0.5D was the smallest of all components in the test. This is consistent with the case where socket joint failure took place while no failure of the column was incurred during the test.
- (2) For specimen CC-1.0D, its residual displacement curve basically coincides with that of the cast-in-place specimen, which demonstrates that they have the similar residual displacement.
- (3) It can be seen that for the same lateral displacement of the column, the residual displacement for all specimens is comparable, except CC-0.5D. And the

final residual displacement of the socket pier specimens increases as the embedment depths increase.

4. Conclusions

A quasistatic test was conducted on the cast-in-place pier specimen and the socket pier specimens without roughening the joint interface between the precast column and the precast foundation. The following conclusions were made based on the observed damage and test data:

- (1) The failure mode of the socket pier specimens with embedment depth of no less than 1.0 times the column's diameter is the same as that of the cast-in-place pier specimen, that is, the bending failure was caused by the flexural capacity insufficiency of the column and occurred out of the embedment region
- (2) The seismic performance of the socket bridge piers, including bearing capacity, displacement ductility, and energy dissipation capacity, are being improved as the embedment depths increasing. Thus, it can be adapted to greater seismic displacement and loads.
- (3) The bearing capacity of the socket specimen CC-1.0D is basically the same as that of the cast-in-place member, but its energy dissipation capacity and displacement ductility are inferior to that of the latter. Whereas, the bearing capacity, ductility, and energy consumption of specimen CC-1.5D are all better than that of the cast-in-place one. Therefore, it is suggested that, for socket joint constructions with no roughening on the joint interface between the precast column and the precast foundation, the embedment depth should be set at a value no less than 1.5 times the diameter of the corresponding column.

Data Availability

The data generated or analyzed during this study are included within this article.

Conflicts of Interest

The authors declare that there are no conflicts of interest.

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