Experimental Study on Seismic Performance of Prefabricated Joints of Underground Utility Tunnel

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Abstract. In order to study the seismic performance of underground utility tunnel prefabricated joints anchored by button-head reinforcement, pseudo-static tests were carried out on two full-scale underground utility tunnel prefabricated joint specimens. The failure mode, bearing capacity, energy dissipation, and other seismic performance of the joints were analyzed. The results show that the bearing capacity, energy dissipation and other seismic performance parameter of these specimens can meet the seismic design requirements. However, the damage in the core area of the joint is severe under the pseudo-static loading. Therefore, when the button-head reinforcement is applied to practical engineering, necessary reinforcement measures should be taken to ensure that the joint has good seismic performance.

1. Introduction
The underground utility tunnel is a new type of underground space structure that concentrates municipal pipelines selectively such as electric energy, communication, water supply and drainage, and their ancillary facilities[1]. It is one of the key points of urban municipal construction and one of the research hotspots at present.

The button-head reinforcement anchorage technology is a new mechanical anchorage method of steel bars developed by China Metallurgical Construction engineering group Corporation (CMCC)[2]. It uses special equipment to thicken the ends of steel bars, and takes button-head as a mechanical anchorage measure to replace traditional bending hook anchoring form of the steel bar. Referring to the explanation of provisions of "Code for Design of Concrete Structures" (GB50010-2010)[3] for the mechanical anchorage of steel bars, the diameter of the circular anchor plates is not less than 2.24 times the diameter of the steel bars, and considering the method of making the button-head, the diameter of the button-head is determined to be about 2.4–3 times of the diameter of steel bars, as shown in figure 1.

Figure 1 Diagram of button-head anchorage steel bar

In the underground utility tunnel, the wall-slab joints are the
key parts of the structure to resist the load, and their mechanical properties and reliability are valued by scholars at home and abroad. Li jie et al.[4-7] carried out shaking table test on utility tunnel under uniform and non-uniform seismic excitations, and the results show that the maximum structural strain appears on corners of the structure and increases with the increase of input amplitude. Guo En-dong et al.[8] analyzed the seismic response of typical utility tunnel system, obtained the concrete damage strain nephogram of typical utility tunnel under different working conditions, and concluded that the connecting of the side wall and the bottom slab is the maximum damage position.

In the above study, the anchorage steel bars of the underground utility tunnel adopt traditional hooked steel bars, while the study on seismic performance of the underground utility tunnel joints with button-head reinforcement is rarely reported in the literature. Therefore, in order to study the seismic performance of prefabricated joint of underground utility tunnel, and to study the feasibility of applying button-head reinforcement to the joints, the pseudo-static tests were carried out on two full-scale utility tunnel prefabricated sandwich wall-slab joint specimens in this paper.

2. Test of the joint

2.1 Manufacture of specimens

In this paper, considering the different locations of joints, one exterior L-sectional joint and one interior T-sectional joint were designed. The sizes of the specimens are shown in figure 2 and figure 3.

The shop drawings of specimens are shown in figure 4. Referring to the requirements of “Code for Design of Concrete Structures”[3] for the anchorage of longitudinal steel bars at the interior joint in top storey of frame, the anchorage length of button-head steel bar is designed to be 0.5l_abE. Besides, considering the anti-seismic grade, the grade and diameter of steel bars, and the strength grade of the concrete, the anchorage length of button-head steel bar is designed to be 16d, that is, 256mm.
2.2 Material properties

The concrete strength grade of the specimens is C40. According to the "Standard for test method of mechanical properties on ordinary concrete" (GB 50081-2002)[9], the characteristic values of compressive strength of 150mm side length concrete cube ($f_{cu,k}$) were measured, as shown in table 1.

Table 1. Concrete compressive strength of specimens.

| Parts of the specimen | Prefabricated part | Cast-in-place part |
|-----------------------|--------------------|--------------------|
| Specimen number       | ZPBJD-1            | ZPZJD-2            |
| $f_{cu,k}$ (MPa)       | 43.25              | 43.25              |
|                       | ZPBJD-1            | ZPZJD-2            |
| $f_{cu,k}$ (MPa)       | 42.18              | 42.18              |

| Reinforcement specifications | Mean yield strength $f_y$ (MPa) | Ultimate tensile strength $f_u$ (MPa) | Elastic modulus $E_s$ (N/mm$^2$) | Yield strain $\varepsilon_y$ ($10^{-3}$) |
|-----------------------------|---------------------------------|--------------------------------------|---------------------------------|--------------------------------------|
| A8                          | 375.91                          | 506.67                               | 2.08$\times10^5$                | 1.81                                 |
| C16                         | 427.80                          | 584.62                               | 1.96$\times10^5$                | 2.23                                 |

The truss steel bar adopts HPB300, and the longitudinal reinforcement adopts HRB400. The mean yield strength ($f_y$), ultimate tensile strength ($f_u$), elastic modulus ($E_s$) and yield strain ($\varepsilon_y$) of the steel bars were measured, as shown in table 2.
2.3 Loading procedure of test
The test was carried out in the Earthquake Simulator Laboratory of Chongqing University. Low cyclic loading scheme and hybrid force/displacement controlling method were adopted in the test, as shown in figure 5.

![Loading procedure of test](image)

Figure 5 Loading procedure of test

According to the gravity load and earth pressure of the underground utility tunnel in practical engineering, the initial moment at the joint was calculated, and then the equivalent horizontal initial load applied to the joint specimen was calculated according to the initial moment of the joint. By calculation, the equivalent initial load of the L-sectional joint specimen is 146.13kN, and the equivalent initial load of the T-sectional joint specimen is 26.09kN. Before applying the initial equivalent load to the joint, pre-load the specimen, and a small load cycle is applied to the specimen. In this paper, the specimen is loaded with 20kN in the forward and reverse directions and unloaded to eliminate the uneven internal force of the specimens. Then, the equivalent horizontal initial load is applied to each nodal point, and the horizontal initial displacement of the nodal point is measured. On the basis of the horizontal initial displacement, displacement control loading is carried out step by step with a step of 10 mm, and the load is cyclically loaded twice at each stage.

At the end of loading, the load of the joint will decrease rapidly with the increase of loading displacement. The test shall be stopped when one of the following conditions is satisfied:

1. The load drops below 80% of peak load;
2. The concrete of specimens is obviously damaged;
3. Wallboard with large overall bending is not suitable for further loading.

3. Failure phenomena and failure mode of specimens
The observation shows that all the joint specimens have undergone four stages of cracking, yielding, ultimate and failure. The final failure mode of each joint specimen is shown in figure 6.

Taking the prefabricated joint ZPBJD-1 as an example, the failure process and phenomena of the specimen is as follows. Under the action of the equivalent horizontal initial load, the initial displacement at the loading point of the specimen is 3 mm, and there is no obvious cracking on the surface of the specimen. Subsequently, the displacement control loading is carried out. When the displacement of the wallboard is loaded to 8 mm, the ultimate tensile strain of concrete in tension zone near the root of the nodal point is reached, and horizontal crack appears. When the displacement is loaded to about 13 mm, longitudinal reinforcement of the specimen begins to yield at the root of the wallboard. As the loading displacement continues to increase, cracks on the surface of the wallboard extend and penetrate gradually. When the displacement is loaded to about 43 mm, the bearing capacity of the specimen reaches limit value. Subsequently, the concrete at the bottom corner of the wallboard is peeled under pressure, and the interface between prefabricated wallboard and cast-in-place part appears to be separated, and the bearing capacity of the specimen begins to decrease slowly. When the displacement is loaded to 83 mm, the concrete cover of the core area of wall-slab joint begins to fall off, and the core area of the joint suffers from flexural shear failure.

For the prefabricated joint ZPZJD-2, the crack on the side of the wallboard extends from
prefabricated wallboard to cast-in-place wallboard when the displacement is loaded to ±20 mm. At the same time, long "X" shaped cross crack appears in the cast-in-place concrete area of the nodal point. When the displacement is loaded to ±80 mm, the crack propagation at the interface between prefabricated wallboard and prefabricated floor slab is serious, and the concrete at the main diagonal crack in the joint core area is peeled off severely. Finally, the joint suffers from flexural shear failure.

![Figure 6 Failure modes of specimens](image)

**Figure 6 Failure modes of specimens**

4. Test results analysis

4.1 Bearing capacity of specimens

The cracking moment, ultimate moment, failure displacement and other parameters of the joints obtained from the test are shown in table 3.

| Specimen number | Cracking moment (kN·m) | Mu.t (kN·m) | Mu.m (kN·m) | Mu.t / Mu.m | Failure displacement (mm) |
|-----------------|------------------------|-------------|-------------|-------------|--------------------------|
|                 |                        | Positive direction | Reverse direction |             |                          |
| ZPBJD-1         | 267.5                  | 557.93       | 489.46      | 523.70      | 436.73                   | 1.20 | 83               |
| ZPZJD-2         | 252.2                  | 468.06       | 481.85      | 474.96      | 385.15                   | 1.23 | 80               |

Mu.t—The ultimate moment of the joints measured by the test, which is calculated by taking 85% of the peak load according to the "Specification of test methods for earthquake resistant building". Mu.m—The ultimate moment of the joints calculated by the measured strength of material

It can be seen from table 3 that the ultimate flexural capacity of ZPBJD-1 and ZPZJD-2 has a safety reserve of about 20%, indicating that the bearing capacity of the specimens anchored by button-head reinforcement meets the design requirements.

4.2 Hysteretic curves of specimens

The load-displacement hysteretic curves of each joint under quasi-static test are shown in figure 7. It can be seen from figure 7 that the general characteristics of the load-displacement hysteretic curves of joint specimens are as follows:

① Before longitudinal reinforcement of the wallboard yields, specimens are generally considered to be in an elastic state. The load-displacement hysteretic curves of specimens are long and narrow (about the first and second loops of hysteretic curves), and the residual deformation of the specimens is small after unloading.

② After specimens enter the elastic-plastic stage, the load-displacement hysteretic curve shows a significant nonlinear relationship. With the increase of loading displacement, the hysteretic curve begins to bend and the stiffness begins to degenerate. On the other hand, the residual deformation increases after unloading and the area enclosed by hysteretic curve increases gradually.

③ At the end of loading, with the bond slip between the longitudinal reinforcement and surrounding concrete during the loading process, the hysteretic curve appears "pinch" phenomenon and intensifies continuously. Finally, the hysteretic curve shows an “Anti-S” shape.
The skeleton curve can reflect the bearing capacity, yield load, ultimate load, ultimate displacement and strength degradation of the specimens. The skeleton curves of specimens are shown in Figure 8.

It can be seen from Figure 8 that the skeleton curves of two specimens have a relatively long smooth decline section, which indicates that the strength of specimens degenerates slowly and two specimens can maintain a stable energy dissipation stage. Therefore, the bearing capacity, energy dissipation capacity and displacement ductility of the prefabricated joints anchored by button-head reinforcement are good.

4.3 Energy dissipation capacity of specimens

The equivalent viscous damping coefficient \( \zeta_{eq} \) is used to evaluate the energy dissipation capacity of joint specimens. Referring to the "Specification of test methods for earthquake resistant building (JGJ/T101-2015)"[10], the equivalent viscous damping coefficient \( \zeta_{eq} \) is expressed as:

\[
\zeta_{eq} = \frac{1}{2\pi} \frac{S_{(ABC+CDA)}}{S_{(OBE+DOF)}}
\]

In equation (1):

- \( S_{(ABC+CDA)} \) — the area enclosed by the load-displacement hysteretic curve;
- \( S_{(OBE+DOF)} \) — the sum of the areas of the triangle BOE and the triangle DOF.

The \( \zeta_{eq} \) of the specimens ZPBJD-1 and ZPBJD-2 calculated by equation (1) are 0.387 and 0.372, respectively. It can be seen that the equivalent viscous damping coefficient of the specimens is about 0.38, while that of ordinary reinforced concrete joints is about 0.1[11]. Therefore, the prefabricated joints anchored by button-head reinforcement have good energy dissipation capacity.

5. Conclusion

In this paper, the pseudo-static tests were carried out on two full-scale utility tunnel prefabricated joint specimens anchored by button-head reinforcement, and the following conclusions can be drawn:

1. The test results of the underground utility tunnel prefabricated joints show that the bearing capacity, displacement ductility, energy dissipation capacity and other seismic performance of the specimens anchored by button-head reinforcement meet the seismic design requirements, indicating that applying the button-head reinforcement to practical engineering is feasible.

2. Under the pseudo-static loading, the crack propagation of prefabricated joints at the interface between prefabricated wall slab and prefabricated floor slab is serious, and damage in the core area of
the joints is severe. The ideal failure mechanism of “strong connection-weak member” can not be well realized. Therefore, necessary reinforcement measures of the joints should be taken in the practical engineering, such as strengthening the connection measures of the joints.

(3) Referring to the requirements of the "Code for Design of Concrete Structures"[3] for the anchorage of longitudinal stressed steel bars at the interior joint in top storey of frame, the anchorage length of button-head steel bar of specimens in this paper is designed to be 0.5\(l_{abE}\). The test results show that if the anchorage length of steel bar is higher than 0.5\(l_{abE}\), the specimens can meet the seismic design requirements.

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