Seismic performance of structures composed of T-shaped and L-shaped recycled concrete columns and lightweight infill walls

Cun Hui\textsuperscript{a}, Yongbo Zhang\textsuperscript{a}, Pan Liu\textsuperscript{a} and Ran Hai\textsuperscript{a}

\textsuperscript{a}School of Architecture and Civil Engineering, Zhongyuan University of Technology, Zhengzhou, China; \textsuperscript{b}Technology Research Center, China IPPR International Engineering Co. LTD, Beijing, China

ABSTRACT

In this paper, a structure composed of a T- or L-shaped recycled concrete column frame (RCCF) and lightweight infill wall (LIW) with a herringbone brace has been proposed to investigate its seismic behavior. Two T-shaped and two L-shaped full-size experimental models of the structure have been built and tested under low-cyclic loading conditions. Consequently, bearing capacity, stiffness, ductility, hysteretic behavior, energy dissipation, and failure characteristics of each test specimen have been analyzed, identifying structure parts prone to damage and failure, understanding their yield mechanisms, and lastly providing constructive recommendations for the structural design. Studies show that the bearing capacity, ductility, and comprehensive seismic behavior of T- or L-shaped lattice-reinforced RCCF-LIW structures with a herringbone brace have been significantly improved. For the engineering practice, it is necessary to strengthen the performance of the herringbone brace-frame beam joint.

1. Introduction

Although the rectangular section columns with great load-bearing capacity and high elastoplastic deformation capacity are used widely in the building structures, the main disadvantages of the rectangular section columns are their large cross section areas, which will greatly decrease the usable area of the building, especially the residences. Therefore, T-shaped and L-shaped columns will be used in the residence buildings. Because of the irregular cross section, the pure frame with T-shaped and L-shaped columns is not good at load-bearing and anti-seismic. To improve the seismic performance of the traditional special-shaped column structures, herringbone brace is added in the structures. Some research results have been obtained in related fields.

A special-shaped column frame-lightweight infill wall (LIW) structure has not only relatively lightweight due to the lightweight masonry used for filler walls, but also good seismic behavior and deformability (Chiou and Hwang 2015, 44). This type of structure has been widely applied. However, studies have shown that as weak parts, its lower column bases easily fail under a relatively strong earthquake loading due to the unique cross sections of its columns (Zovkic, Sigmund, and Guljas 2013, 42). One typical infilled masonry reinforced concrete frame buildings and their action in the course of an extreme load have been conducted by Brodsky and Yankelevsky (2017, 140) and the increasing frame deflection at the point of the missing column support under the action of gravity loading was restrained. Two 1/3 scaled, four-bay, two-story RC frame specimens were tested by Shan et al. (2016, 111) and it is concluded that the infill walls could affect the performance of RC frames against progressive collapse in different aspects. A new aseismic infill wall system composite steel plate deep beam was introduced by Jiang et al. (2014, 14), Zhou et al. (2012, 28) conducted low-cyclic tests on L-shaped reinforced concrete column frames and analyzed their seismic performance. In order to know the effects of simplified boundary element on seismic performance of high-rise and medium-height shear walls, the low frequency quasi-static cyclic loading experimental study was carried out by Zhang, Cao, and Yin (2009, 42) and Cao et al. (2010, 31). Zhou et al. (2015, 114) and Zhao et al. (2017, 133) experimentally investigated the seismic behavior of specially shaped columns composed of concrete-filled steel tube frames with or without chevron braces subjected to constant axial load and cyclically varying flexural load. Xiao, Sun, and Falkner (2006, 28) experimentally studied the seismic behavior of four half scale recycled aggregate concrete frames and analyzed their aseismic capabilities and energy dissipation characteristics. Similarly, based on the experimental results, Gonzalez and Moriconi (2014, 60) and Corinaldesi, Letelier, and Moriconi (2014, 25). analyzed the seismic performance of recycled concrete column frame (RCCF) structures. Xiao et al. (2012, 109) performed shaking table...
experiments on recycled concrete frames. The possible effects of openings in the infill wall on seismic behavior of RC frames is analytically studied through pushover analysis of several bare, partially and fully infilled frames having different bay and story numbers by Ozturkoglu, Ucar, and Yesilce (2017, 12). The seismic behavior of a 1/2 scaled, three-story three-bay RC frame with masonry infill walls was studied experimentally and numerically by Fenerci et al. (2016, 10). These research findings guide the study of this paper.

In this study, a novel structure composed of T- or L-shaped lattice-reinforced RCCF and LIWs is proposed and low-cyclic loading tests with full size structural models are performed. Subsequently, bearing capacity, stiffness, ductility, hysteretic behavior, energy dissipation, and failure characteristics of the structure can be analyzed.

2. Experimental details
2.1. Design of specimens

Four full-size test specimens of the structure have been designed, i.e., TKJ-1 composed of T-shaped lattice-reinforced RCCF and fly ash block infill wall, TKJ-2 similar to TKJ-1 except for a recycled concrete herringbone brace added to the infill wall, LKJ-1 composed of L-shaped lattice-reinforced RCCF and fly ash block infill wall, and LKJ-2 similar to LKJ-1 except for a recycled concrete herringbone brace added to the infill wall. The detailed geometrical dimensions and reinforcement of each specimen are shown in Figure 1.

Details of the specimen TKJ-1 are described as follows: (1) the cross section of T-shaped columns has 150-mm long and long limbs, respectively, with a thickness of 150 mm. The clear height of the columns is 2650 mm. Longitudinal bars (5C16) are assembled with single-row reinforcement form and reinforcement rate of 0.89% and the single-row bar mesh is located exactly in the middle of the cross section of the special-shaped columns. The tie bars (A6@150/100) are installed as shown in section 1–1 of Figure 1(a); (2) the cross section of the frame beam is T-shaped with a height of 400 mm, flange width of 450 mm and flange thickness of 80 mm. Bi-directional double-row steel bars (A6@100) are assembled within the flanges. The single-row steel bars (1C16) are installed at the upper and lower portions of the 150 mm-wide web, respectively, with tie bars (A6@150/100) as shown in section 3–3 of Figure 1(b).

Compared to TKJ-1, the specimen TKJ-2 has a similar structure except for a recycled concrete herringbone brace in the infill wall. The construction process and testing have been performed at the “Key Laboratory of Engineering Anti-Earthquake and Structure Diagnosis and Treatment of Beijing City” of the Beijing University of Technology. The detailed process is illustrated in Figure 2.

The recycled coarse aggregate concrete comes from one 30 years old building in Xidan district of Beijing. It has been prepared with the following properties: C60 strength grade; proportions of the cement, water, sand and coarse aggregate mix (ratio of mass per unit volume) are 1: 0.36: 1.51: 2.17. The mean measured compressive strength for this concrete is 60.73 MPa and the mean elastic modulus is 32800 MPa. As manufactured by Qingzhou Jinju New Building Materials Co., Ltd, the fly ash aerated block B06 with measured compressive strength of 2.35 MPa are used for infill
walls of the specimens. Composite mortar M5 is selected for masonry mortar. Measured mechanical properties of reinforcing bars are shown in Table 1.

2.2. Loading scheme

During the test, constant vertical axial loads of 800 kN with an axial compression ratio of 0.17 are initially applied to the top of two frame columns of each test specimen by using a vertical jack – roller bearings – reaction frame – hydraulic control system. Then, lateral low-cyclic loads with different levels are applied to the center of the cross section of the specimen beam. This loading point is 2850 mm away from the top of the foundation beam. The test loading configuration is shown in Figure 3 and specimen photos are shown in Figure 4.

The hybrid control of force and displacement is adopted for the loading application process during the test. Before the specimen yields, the load controlling mode is used and each level of cyclic loading is applied one time. After the specimen yields, the displacement controlling load is used. Besides, the cyclic loading, each level of which is applied one time, continues until the test specimen presents significant damage and the loading cannot continue, or lateral loading drops to a value less than 85% of the peak load. The loading pattern is shown in Figure 5. $F_m$ is the
peak load and $\Delta_y$ is the displacement corresponding to the yield point. The yield point of the specimen is determined on the measured load-displacement curve as the point where it significantly deviates from a straight line.

3. Experimental results and analysis

3.1. Failure patterns

The failure modes of TKJ-1, TKJ-2, LKJ-1, and LKJ-2 are illustrated in Figure 6(a–d). From Figure 6 it can be observed that:

1. The failure patterns of TKJ-1 are basically consistent with those of LKJ-1: at the initial loading stage, no visible cracks and damage are found due to the small loads; as the load increases, the infill wall firstly starts to crack, and then diagonal cracks throughout blocks and stepped cracks along vertical and horizontal mortar joints between blocks appear. Consequently, after the stiffness of the infill wall gradually degrades and the structure significantly yields, flexural cracks throughout the entire cross section of the beam emerge and are mainly distributed within one-third of the beam span away from both beam ends. Besides, those cracks are mainly caused by the bending stresses and the development of a plastic hinge. With the lateral loading continually increasing, the infill wall is gradually separated from the frame and significant dislocation occurs along the diagonal cracks throughout the blocks and stepped cracks appear along the vertical and horizontal mortar joints between blocks; lastly, as the load increases, the infill wall is completely separated from the frame and crushed and it cannot continue to work. This is followed by the formation of plastic hinges at the upper ends of the columns. Therefore, joint failure refers to the situation where the concrete at the boundary of the joint zone becomes crisped and crushed due to the concrete of the beam and upper ends of the columns crisped, while there are only several oblique cracks appearing at the core area of the joint. Moreover, the crack growth can be inhibited by the flanges of the special-shaped columns. Failure of the core area of the joint occurs after the beam and columns fail, and the deformation of the entire structure is well developed. Thus, the failure mechanism of the structure is the “strong column-weak beam-stronger joint” type.

2. The failure patterns of TKJ-2 are basically consistent with those of LKJ-2: at the initial loading stage, no visible cracks and damage are observed due to the small loads; as the load increases, the
followed by the cracking of concrete herringbone brace which is the first line of defense against earthquake loading; after the specimens reach their peak loads, the loading exhibits an abrupt drop because, at the later loading stage, shear diagonal cracks appear nearby the frame beam-herringbone brace joint (FB-HBJ) resulting in the apparently weaken stiffness and significantly reduced bearing capacity; shear cracks initially emerge at the FB-HBJ leading to a rapid decrease of the lateral stiffness and a significant reduction of bearing capacity of the entire structure while no visible plastic hinge is developed on both ends of the beam. For the brace, although tensile-compressive stresses are dominant, the bending moment exists causing corresponding cracks. The segmented beam shear failure occurs nearby the FB-HBJ since the supporting longitudinal bars are slipping. Therefore, in real engineering design, the longitudinal bars at the joint should be strongly anchored and the shear capacity of the cross section of the beam nearby the joint should be strengthened to ensure that the joint, as a part of the beam, has sufficient shear strength even if the brace fails.

(3) In real engineering design, the supporting longitudinal bars at the FB-HBJ should be strongly anchored and the shear capacity of the cross sections of the frame beam nearby the joint should be strengthened to ensure that the herringbone brace can play its role in energy dissipation as the first measure to resist earthquake loadings, and the joint, as a part of the beam, has sufficient shear strength even if the brace fails.

3.2. Hysteretic behavior

Figure 7 shows the load-displacement hysteretic curve of each test specimen, where $F$ is the lateral load at the horizontal loading point of the specimen and $\Delta$ represents the corresponding horizontal displacement. It can be observed from Figure 7 that:

(a) specimen TKJ-1

(b) specimen TKJ-2

Figure 4. Specimen photos.

Figure 5. Loading pattern.
(1) at the early stage of loading, the specimens are in the elastic state and the load-displacement curve changes linearly due to the small loads. Therefore, the specimens can recover to their original state after the unloading.

(2) as the load increases, the load-displacement curve exhibits certain nonlinear changes resulting in a residual deformation of the specimen's structure after the unloading. The certain pinching phenomenon appears on the load-displacement hysteretic curve at the later stage of loading.

(3) the hysteresis loop exhibits good symmetry under positive and negative loads. Along with the loading times, the hysteresis curve of the specimens initially changes from arch to reverse S-shape, and then change back again, which indicates the specimens have a strong energy dissipation capacity.

(4) for TKJ-1, the load declines slowly after it reaches the peak value, while for TKJ-2 with a herringbone brace, the load drops significantly after it reaches the peak value. This phenomenon is because at the later stage of loading, shear diagonal cracks appear at the FB-HBJ resulting in a significant reduction of the stiffness and bearing capacity.

(5) for LKJ-1, it still involves energy dissipation and its load also decreases slowly even after the load reaches the peak value. The area enclosed by the hysteresis loop is further increased. For LKJ-2, the load shows an abrupt drop after it reaches the peak value because at the later stage of loading, shear diagonal cracks appear nearby the FB-HBJ leading to a significant decrease of the stiffness and bearing capacity.

3.3. Load-bearing capacity and backbone curve

The bearing capacity values of the different specimens at various loading stages are listed in Table 2, where $F_c$ is the cracking load which refers to the load when visible cracks firstly appear on the special-shaped columns; $F_y$ is the yield load; $F_m$ is the peak load; and $F_u$ is the failure load referring to 85% of the peak load in this test. The mean value of the positive and negative yield loads is taken as the yield load. Similarly, the mean value of the positive and negative ultimate loads is taken as the ultimate load. The load-displacement
backbone curve of each specimen is illustrated in Figure 8.

Table 2 and Figure 8 demonstrate that:

(1) for each specimen, its backbone curve experiences elastic, elastic-plastic, and decedent segments. The slopes of the specimen’s backbone curves at the elastic segment are larger than slops of the elastic-plastic and decedent segments, and have a similar ascent trend, indicating the initial stiffness of the structures can resist the initial deformation.

(2) for TKJ-2, its cracking load decreases, while its mean yield and peak loads increase by 23.63% and 27.7%, respectively, compared to those for TKJ-1, which illustrates the concrete herringbone brace can significantly improve the yield and peak loads for integral structure.

(3) the mean yield and peak loads of LKJ-2 increased by 34.98% and 39.25%, respectively, compared to those of LKJ-1, indicating the concrete herringbone brace can significantly...
(4) at the later stage of loading, the bearing capacities of TKJ-2 and LKJ-2 rapidly decline because the shear failure occurs at the FB-HBJ resulting in an abrupt change on the specimens’ stiffness.

### 3.4. Displacement and ductility

The measured displacements of the specimens are listed in Table 3, where $\Delta_c$, $\Delta_y$, $\Delta_m$, and $\Delta_u$ are the displacements corresponding to $F_c$, $F_y$, $F_m$ and $F_u$ respectively; $\theta_u$ is the inter-layer drift angle corresponding to $\Delta_u$; and $\mu$ is the displacement ductility factor (DDF) which corresponds to the ratio of the ultimate displacement $\Delta_u$ to the yield displacement $\Delta_y$.

From Table 3 it can be observed that:

The DDF of TKJ-2 significantly decreased by 42.82% compared to that of TKJ-1, while the DDF of LKJ-2 is reduced by 37.45% compared to that of LKJ-1, indicating that despite the concrete herringbone brace can enhance the initial stiffness and bearing capacity of the whole structure, shear failure occurs nearby the FB-HBJ at the later stage of loading, therefore, the brace has not effectively played its role as the first line of defense against earthquake loading and the structure has not allowed its energy dissipation mechanism to act resulting in a reduction in the structure’s ductility. According to the

### 3.5. Stiffness degradation

The secant stiffness at the peak points of a hysteresis loop represents lateral stiffness of the specimen under cyclic loading. The stiffness degradation curve of the specimens is shown in Figure 9, where $K$ is the secant stiffness at peak points at different loading times as presented in Table 4 (herein, $K_0$, $K_y$, $K_m$ and $K_u$ are the mean secant stiffness of the hysteresis loop as the specimens are under initial load, yield load, peak load, and ultimate load, respectively).

It can be noted that:

1. The stiffness degradation of the specimens can be divided into four stages, namely, initial to cracking stage, cracking to yield stage, yield to ultimate stage, and ultimate to failure stage.

2. The stiffness degradation of all the specimens exhibits a fast to slow trend: the stiffness rapidly decreases as the infill wall starts to crack due to the large initial stiffness of the specimens; after the wall is seriously damaged and cannot work, the cracks on frame beam and columns further develop leading to a continuous decrease in the stiffness with a small degradation rate.

3. The initial stiffness, yield stiffness, and peak stiffness of TKJ-2 are increased by 59.48%, 30.53%, and 62.68%, respectively, compared to those of TKJ-1, demonstrating the concrete herringbone brace can significantly enhance the stiffness of the specimens at the various loading stages as well as the overall lateral displacement-resisting ability.

4. The initial stiffness, yield stiffness, and peak stiffness of LKJ-2 are increased by 61.76%, 18.32% and 69.01%, respectively, compared to those of LKJ-1, which indicates the concrete herringbone brace can significantly enhance the initial and

### Table 3. Measured displacements and ductility of the specimens.

| Specimen | Loading direction | $\Delta_c$ (mm) | $\Delta_y$ (mm) | $\Delta_m$ (mm) | $\Delta_u$ (mm) | $\theta_u$ | $\mu$ |
|----------|-------------------|-----------------|-----------------|-----------------|-----------------|-----------|------|
| TKJ-1    | Positive          | 5.09            | 12.06           | 31.07           | 57.40           | 1/50      | 4.39 |
|          | Negative          | 5.13            | 13.09           | 31.02           | 52.66           | 1/54      |      |
| TKJ-2    | Positive          | 2.83            | 12.86           | 27.22           | 32.77           | 1/87      | 2.51 |
|          | Negative          | 2.10            | 11.16           | 22.69           | 27.69           | 1/    |      |
| 103      |                   |                 |                 |                 |                 |          |      |
| LKJ-1    | Positive          | 1.58            | 9.20            | 30.21           | 49.67           | 1/57      | 4.70 |
|          | Negative          | 3.08            | 14.61           | 34.41           | 58.47           | 1/49      |      |
| LKJ-2    | Positive          | 2.78            | 12.73           | 22.83           | 35.84           | 1/80      | 2.94 |
|          | Negative          | 0.94            | 13.41           | 30.89           | 41.08           | 1/70      |      |

### Table 4. Stiffness at characteristic points of the specimens.

| Specimen | $K_0$ (kN·mm$^{-1}$) | $K_y$ (kN·mm$^{-1}$) | $K_m$ (kN·mm$^{-1}$) | $K_u$ (kN·mm$^{-1}$) |
|----------|----------------------|----------------------|----------------------|----------------------|
| TKJ-1    | 63.99                | 35.64                | 21.26                | 9.94                 | 4.75      |
| TKJ-2    | 102.05               | 60.62                | 27.75                | 16.17                | 11.33     |
| LKJ-1    | 60.93                | 34.35                | 20.47                | 8.26                 | 4.20      |
| LKJ-2    | 98.56                | 59.56                | 24.22                | 13.96                | 8.20      |

### Table 5. Measured energy dissipation of the specimens.

| Specimen | Energy dissipation $E$ (kN·mm) | Relative value |
|----------|--------------------------------|----------------|
| TKJ-1    | 14751.80                      | 1.0000         |
| TKJ-2    | 9574.94                       | 0.6990         |
| LKJ-1    | 12662.46                      | 1.0000         |
| LKJ-2    | 11975.26                      | 0.946          |
peak stiffness of the specimens, while its effect on yield stiffness is relatively small.

3.6. Energy dissipation capacity

The area enclosed by the hysteresis loop reflects the elastic-plastic dissipation energy of the structure. Due to the difference in the loading process for the specimens, the area enclosed by the coordinate axes and the backbone curve of each specimen before it reaches failure is taken as a representative value of energy dissipation. The measured value of specimens’ energy dissipation is given in Table 5, where the value refers to the mean representative value of the positive and negative energy dissipation of each specimen. Noticeably, compared to TKJ-1, TKJ-2 reduces its energy dissipation by 35.1%; similarly, LKJ-2 lowers its energy dissipation by 5.4% compared to LKJ-1. This is since at the later stage of loading, segmented shear failure occurs at the beam of the specimen with a concrete herringbone brace leading to an abrupt decrease in stiffness and a reduction in the specimen’s energy dissipation capacity. Therefore, in real design, the FB-HBJ and lateral shear capacity of the beam should be enhanced to ensure the specimen’s stiffness at the later loading stage.

4. Conclusions

With this experimental study and analysis, it can be concluded that:

(a) The T- or L-shaped lattice-reinforced RCCF-LIW structures proposed in this study have large plastic deformation capacity and better energy dissipation capacity.

(b) The initial stiffness of the whole structure can be enhanced by using a herringbone brace which serves as the first line to resist earthquake loadings and meets the seismic conceptual design. Moreover, it also plays an important role in improving the weak parts of the structure base.

(c) The T- or L-shaped lattice-reinforced RCCF-LIW structure not only can save steels and simplify the reinforcement forms but also has better energy dissipation capacity and well-developed deformation due to its “strong column-weak beam-stronger joint” failure mechanism. Besides, the use of recycled concrete to construct rural residential projects allows achieving energy-saving and aseismic integration.

(d) The FB-HBJ should be greatly strengthened to prevent the segmented shear beam failure due to the insufficient shear capacity resulting from the herringbone brace. Otherwise, the whole structure might fail due to its insufficient lateral shear resistance.

(e) Reasonable matching of the frame beam stiffness, columns, and the brace is needed to achieve an effective energy dissipation mechanism, i.e., the brace initially failing followed by the failure of beam and columns.

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Notes on contributors

Cun Hui is an associate professor at Zhongyuan University of Technology. He focus on the research of seismic performance of steel-concrete composite structures.

Yongbo Zhang is a senior engineer at China IPPR International Engineering Co. LTD. He focus on the research and design of the complex and tall building structures.

Pan Liu is a master student at Zhongyuan University of Technology. She focus on the research of mechanical property of the high performance concrete.

Ran Hai is a professor at Zhongyuan University of Technology. She is focus on the research of seismic performance and mechanical property of the high performance concrete structures.

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