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Experimental study of a self-centering prestressed concrete frame subassembly

Liang-Long Song, Tong Guo *, Yu Gu, Zhi-Liang Cao

Key Laboratory of Concrete and Prestressed Concrete Structures of the Ministry of Education, Southeast University, Nanjing 210096, PR China

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ABSTRACT

This paper presents the experimental investigation of the behavior of a 0.5-scale, one-bay, one-story self-centering (SC) moment resisting concrete frame subassembly under cyclic loading. The SC frame utilizes post-tensioned (PT) beam–column connections and column–foundation connections to provide the re-centering capabilities after earthquakes. Beam web friction devices are included in the beam–column connections to provide the adjustable energy dissipation capacity, and steel jackets are used at the beam ends and column bases to avoid concrete crushing at the contact interfaces. The influence of various parameters (i.e. initial PT forces in the beam and columns, friction forces, etc.) on the seismic behavior of the SC frame is experimentally examined, and it is observed that the frame subassembly performed well up to a 2.5% drift with only minor damage in concrete cover of the beam, and the column remained damage free. To avoid the stiffness reduction upon unloading and to achieve desirable energy dissipation, the stiffness of the friction devices and connecting bolts should be properly enhanced in future study.

1. Introduction

Reinforced concrete (RC) moment-resisting frames (MRFs) have been widely used as the primary lateral load-resisting system of buildings during the past decades in seismic areas. Conventional RC-MRFs designed under the prevailing capacity design philosophy are expected to be a ductile system under moderate to strong earthquakes, capable of developing inelastic deformations in well-detailed regions, so as to protect the structure from unfavorable instability or collapse. Recent earthquakes, e.g. the 1994 Northridge earthquake, the 2011 Canterbury earthquake, etc., have validated the adequacy of such a design philosophy in preventing structural collapse and loss of lives. However, it is also observed after these earthquakes that the buildings may experience extensive structural damage and residual deformations, resulting in considerable direct and indirect losses due to operation disruption and repair.

To reduce the residual deformations and eliminate the need for the post-earthquake retrofit, unbonded post-tensioned precast concrete (PC) MRFs have been developed and investigated. Previous study shows that unbonded post-tensioned PC-MRFs can have adequate strength, ductility and undergo large nonlinear displacements while maintain an excellent self-centering (SC) ability [1–3]. However, one major concern for unbonded post-tensioned PC-MRFs in seismic regions is that their lateral displacements during earthquakes may be larger than acceptable due to small energy dissipation. Therefore, it is necessary to adopt additional energy dissipators to reduce the displacement demands in earthquakes. Early form of dissipators provided energy dissipation through metallic yielding of mild steel bars grouted in ducts across the beam-to-column joints ("hybrid" connection) [4], which was developed in the US-PRESSS Program (PREcast Seismic Structural System) [5]. Subsequent research on "hybrid" precast structural systems brought other forms of dissipators, including tension-only mild steel devices [6], external mounted mild steel devices [7,8], lead-based damping devices [9,10], and friction-based devices on the top and bottom surfaces of the beam ends [11]. More recently, Chou and Chen [12] conducted full-scale cyclic tests on a one-story two-bay post-tensioned reinforced concrete steel (RCS) frame subassembly with reduced flange plates and demonstrated the excellent self-centering capacity of the PT frames. Shake table tests [13] on a 0.4-scale one-story, two-by-two bay post-tensioned concrete-filled tube (CFT) frame subassembly further confirmed the large deformation and self-centering ability of the PT frame systems. Along with the above developments in precast concrete (PC) MRFs, unbonded post-tensioning (PT) technology has also been applied to bridge piers [14,15], walls [16], bracing members [17] and beam–column connections of steel frames [18] to reduce the residual deformations of seismic resistant systems.

* Corresponding author. Tel./fax: +86 25 83795054.
E-mail address: guotong77@gmail.com (T. Guo).

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Recently, a new concrete PT connection, i.e. the self-centering prestressed concrete (SCPC) beam–column connection, has been proposed by Song et al. [19]. The novel connection uses the unbonded PT strands to provide the self-centering (SC) capacity and bolted web friction devices to dissipate earthquake energy. In addition, steel jackets are fabricated at the beam ends to prevent the concrete from damage. Previous tests on the beam–column connection have shown that, the SCPC connection can not only provide desirable energy dissipation capability but also prevent local concrete failure at the beam–column interfaces, without diminishing the self-centering capacity.

In order to further examine the performance of the MRFs with the proposed SCPC connection, six low cyclic tests were conducted on a 0.5-scale, one-bay, one-story frame subassembly. Post-tensioned column–foundation connections are also used in the frame system to eliminate the damage at the bases of columns and provide additional re-centering capacity. Primary design parameters (i.e. initial PT forces in the beam and columns, friction forces, etc.) were changed in these tests, so as to evaluate their influence on the seismic behavior of the SC frame.

2. Low cyclic tests

2.1. Test specimens

To evaluate the static cyclic performance of a self-centering concrete moment-resisting frame with PT beam–column and column–foundation connections, a 0.5-scale frame subassembly was constructed and tested at the Key Laboratory of Concrete and Prestressed Concrete Structures of the Ministry of Education at Southeast University, China. The test frame was designed according to the first story of a prototype self-centering concrete moment resisting frame designed by Guo and Song [20]. Fig. 1 shows the reinforcements and dimensions of the frame subassembly. The column had a height of 2300 mm and a cross section of 350 mm $\times$ 350 mm. The clear span of the beam was 2650 mm, and the beam was 200 mm wide by 350 mm deep. The beam and columns were expected to experience no damage in the tests and were repeatedly used. A tendon duct with a diameter of 70 mm was cast into the beam along its longitudinal axis and two tendon ducts with a diameter of 40 mm were cast in each column, as shown in Fig. 1(b) and (c). The beam was post-tensioned to the columns using two 15.2-mm diameter high strength PT strands, which passed along the mid-depth of the beam and were anchored outside of the exterior columns. The columns were connected to the foundations using two 15.2-mm diameter strands through posttensioning. To protect the concrete from damage during rocking, the ends of the precast beam and columns were armored with steel jackets and steel plates were embedded in the concrete column and foundations, as shown in Fig. 1(a). The thickness of the embedded steel plates and steel jacket was 8 mm. Two 8-mm thick shim plates were welded on the steel plate embedded in the column, as shown in Fig. 2(a), to ensure good contact between the beam and column [19]. Similarly, two shim plates were used at the column–foundation interfaces to maintain good contact between the column and foundation. Due to the existence of steel jackets, no shear reinforcements were used in the jacketed segments of the beam and columns. As shown in Fig. 2, web friction devices were installed at the beam ends to increase the energy dissipation capacity of the frame system. The web fiction device consisted of a steel channel and a steel connection plate and was bolted through the connection plate to the column using two 18-mm diameter high strength bolts, as shown in Fig. 2(a). Four bolt ducts, running perpendicular to the beam axis, were built in the beam, as shown in Fig. 2(b). A friction plate (i.e., 2-mm thick brass plates) was sandwiched between the inner surface of the steel channel and the steel jacket. One of the two surfaces of the friction plate was roughened and adhered to the steel channel using epoxy glue. Four 20-mm diameter high strength bolts passed through the channels, brass friction plates, and beam and produced normal force on the friction surfaces using a torque wrench. The diameter of the oversized circular bolt ducts in the steel jacket and concrete beam was 50 mm, which was determined considering that the friction bolts should not bear against the bolt ducts in the beam at a connection relative rotation of 3.5% rad.

2.2. Test setup and instrumentation

A photograph and an elevation of the test setup are shown in Fig. 3(a) and (b), respectively. First, the foundations were positioned to the strong floor. The precast columns were then temporarily post-tensioned to the foundation. The precast beam was then lifted and the strands were placed in the duct of the beam and anchored on the exterior surfaces of columns. After the beams had been installed, the PT strands in columns were post-tensioned to the designed stress level. Finally, the web friction devices were bolted to the column and the friction bolts were tensioned to their design force level using a calibrated torque wrench.

The lateral load was applied on the east column (EC) through a MTS servo-hydraulic actuator, at the story height (centerline dimension) of 1933 mm above the top of the foundation. The displacement of the east column at the loading point was used as the target displacement, and the inter-story drift was defined as the lateral displacement at this loading point divided by the story height.

The force and displacement of the actuator were recorded automatically by the loading system. Vibration string force transducers were installed on the top of columns and outside surface of the beam–column connection to measure the PT forces in the columns and beam. The effective length of the PT strands in the beam and columns were 3.56 m and 2.86 m, respectively. As shown in Fig. 3(c), the relative rotation of the PT beam–column connection was obtained from two displacement transducers located at the top and bottom of the beam at the beam–column interface and the distance between the two transducers. Similarly, two displacement transducers were placed on the east and west sides of the column at the column–foundation interface to measure the PT column–foundation connection rotation. Resistance strain gauges were adhered long the midline of the steel jackets to measure the strains of steel jackets in the beam and columns, as shown in Fig. 3(c). These strain gauges had a spacing of 3 cm, and were labeled as Gauge 1 to Gauge 7 (from the beam–column or column–foundation interfaces toward the far end of the jacket).

2.3. Material properties

The mean cubic compressive strength of the concrete was 35 MPa. The nominal diameter of the tendons was 15.2 mm, corresponding to a cross-sectional area of 140 mm^2. The tendons had a modulus of elasticity (E_{st}) of 1.95×10^5 MPa and the measured yield strength was 1730 MPa (with the yield strain of 1%). According to the typical double shear test results, the coefficient of friction between the brass and steel was around 0.33 [19]. The steel plates were Q235 steel with nominal yield strength of 235 MPa. The longitudinal reinforcing bars were HRB335 steel with the nominal yield strength of 335 MPa. The transverse reinforcement and cross ties were HPB235 steel with the nominal yield strength of 235 MPa [21].