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Seismic response of self-centering prestressed concrete moment resisting frames with web friction devices

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ABSTRACT

This paper examines the seismic response of self-centering prestressed concrete moment resisting frames (SCPC-MRFs) with web friction devices. Nonlinear dynamic analyses under ground motion ensembles corresponding to two seismic hazard levels are performed. The results of SCPC-MRFs are compared with those of monolithic reinforced concrete (RC) MRFs in terms of global and local responses. Seismic analyses results show that SCPC-MRFs generally experience similar peak story drifts, less beam end rotations, and much smaller residual deformations as compared with those of RC-MRFs. However, it is also found that SCPC-MRFs sustained larger column plastic rotations than RC-MRFs and are more sensitive to the formation of a soft story. In general, the 8-story SCPC-MRFs show better seismic performances than the 4-story ones. Mainshock–aftershock analyses show that the incremental effect of aftershock on damage extent is much smaller for the SCPC-MRFs than for the RC-MRFs.

1. Introduction

Reinforced concrete (RC) moment-resisting frames (MRFs) have been widely used as the main lateral force resisting systems in buildings. Although conventional RC-MRFs usually lead to attractive initial cost savings, they are likely to yield under moderate-to-strong earthquakes with significant structural damage and residual deformation, resulting in substantial costs associated with operation disruption and repair of damage. Considerable damage in RC frame buildings has been reported in past earthquakes, including the 1999 M7.6 Chi–Chi, Taiwan earthquake [1], the 2008 M8.0 Wenchuan, China earthquake [2], and the 2011 M6.3 Christchurch, New Zealand earthquake [3]. To reduce the residual deformation and structural damage after earthquakes, precast concrete moment-resisting frames with unbonded post-tensioned (PT) elements have been developed and investigated [4], in which notable recentering capacity was observed. To mitigate displacement demands and provide a certain level of energy dissipation, supplemental energy dissipation devices are usually adopted in these structures, including mild steel bars grouted in ducts across beam–column joints ("hybrid" connection) [5], mild steel devices [6–8], lead-based damping devices installed externally across beam–column joint or internally within beam-end-zone [9,10], and friction-based devices on top and bottom surfaces of beam ends [11]. In general, it is required that these energy dissipators should be easily installed, and be replaceable if damaged.

To investigate the seismic performance of unbonded post-tensioned RC-MRFs, analytical studies have been carried out. The analytical evaluation [12] on the seismic performance of RC-MRFs with "hybrid" connections showed that the seismic responses of hybrid concrete frames are similar to or better than RC concrete frames. Pampanin et al. [13] proposed a sectional analysis method for precast concrete members, and validated the method through the tests on hybrid beam–column subassemblages and a five-storey precast concrete building with hybrid connections. Morgen and Kuruma [14] analytically evaluated the seismic design and behavior of post-tensioned precast concrete frames with external friction dampers, and concluded that the prototype structures can attain a good energy dissipation capacity, while maintaining a large self-centering capability. Note that the above studies focused on the global displacement responses (i.e. roof drift and story drift); however, information regarding the local responses (i.e. the plastic rotation demands of structural members) of post-tensioned RC-MRFs when subjected to strong earthquake excitations were not addressed, which are important in evaluating the seismic performance of structures.

In addition, seismic sequences characterized by a main-shock followed by strong aftershocks have been observed in many seismically active regions. Field investigations have reported the cumulative damage of structural systems under repeated earthquakes. For instance, during the 2008 Wenchuan, China earthquake sequence, dozens of damaged reinforced concrete (RC)

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buildings weakened by the main-shock suffered from partial or complete collapse due to aftershocks. Numerical studies on the responses of RC moment frames [15,16] have also indicated that aftershocks may increase the damage state of structures at the end of the main-shock. Despite the fact that repeated earthquakes hazard is clearly threatening, the effect of repeated earthquakes on post-tensioned structures has not gained much attention. To the author's knowledge, no study has been reported in literatures regarding the effect of mainshock–aftershock seismic sequences on the structural damage of post-tensioned RC frame buildings.

Recently, a new concrete post-tensioned connection, i.e. the self-centering prestressed concrete (SCPC) beam–column connection, has been developed and investigated [17,18]. In the proposed SCPC connection, the self-centering ability is realized using post-tensioned tendons running parallel to the beam, and the energy dissipation capacity is mainly achieved through the friction devices located at the beam ends. In addition, the friction devices are located at the beam webs and thus have no interference with the floor slab, as compared with the connections developed by Morgen and Kuruma [11]. Moreover, steel jackets are shop fabricated at the beam ends, protecting the concrete from crushing when the beam rotates relatively against the column.

Based on the analytical and experimental studies on the SCPC connection [17], this paper aims at evaluating the seismic performance of the unbonded PT concrete frames with SCPC connections. Two SCPC-MRFs and two RC-MRFs (both having 4 and 8 stories) are designed according to the guidelines presented by Song et al. [17] and ASCE 7 and ACI 318 codes. Analytical models of the buildings are developed and nonlinear time-history analyses are conducted under two sets of ground motions corresponding to seismic hazard levels with 2% and 10% probabilities of exceedance in 50 years. Both the global responses (i.e. peak story drifts and residual deformation) and local responses (i.e. relative/plastic rotations at beam and column ends) under the ground motion ensembles are evaluated. Additional seismic analyses are also carried out to investigate the effects of mainshock–aftershock seismic sequences on the performance of the RC- and SCPC-MRFs.

2. SCPC connections

2.1. Configuration and mechanical behavior

A schematic illustration of a MRF subassembly with SCPC beam–column connections is shown in Fig. 1(a). The post-tensioned tendons running horizontally precompress the precast concrete beam to the column to resist moment and are anchored outside the exterior columns. Shear forces at beam ends are taken by friction forces at beam–column interfaces generated by PT tendons and at the web friction devices (WFDs). The details of a typical exterior SCPC beam–column connection are shown in Fig. 1(b), where the beam end is casted inside a steel jacket which protects the concrete at the beam–column interface from spalling and crushing. Located at mid-depth of the beam is a WFD which also contributes to the moment and shear capacity at the beam end and dissipates energy under seismic loading. Each WFD consists of two steel channels which are bolted (or welded) to steel plates embedded in the column. Sandwiched between the outer channels and the steel jacket are two friction plates (i.e., brass or aluminum plates), which are attached to the two inner surfaces of the steel channels. High strength friction bolts with washers compress the friction device and the steel jacket together and produce controlled normal forces on the friction surfaces. The diameter of the holes through the steel jacket and beam is much larger than that of the friction bolts to accommodate the relative motion of friction bolts when the beam rotates relative to the column, as shown in Fig. 1(c). Two steel shim plates are welded to the steel plate in the column at the beam column interface to obtain good contact between the beam and column.

Fig. 2(a) shows the free body diagram of a typical connection subassembly, where P is the resultant force of all tendons forces; F_f is the friction force resultant on the web friction device; N_c and V_c are the axial compressive force and shear friction force on the beam–column bearing interface, respectively; d_2 is the distance from the beam section centroid to the center of rotation (COR); and r is the distance between F_f and the COR; The connection moment M (at the beam–column interface) is controlled by two parts, M_{PT} and M_{Ff}, representing the contributions of PT tendons and web friction device, respectively, as follows:

$$M = M_{PT} + M_{Ff} = P \cdot \mathrm{d}_2 + \mathrm{F_f} \cdot r \quad (1)$$

The conceptual connection moment-relative rotation (M–θ_r) behavior of the SCPC connection under cyclic loading is further illustrated in Fig. 2(b). The beam–column interface is in complete contact until the applied external moment exceeds the imminent gap-opening moment M_{IGO} (point 1 in Fig. 2(b)), which can be calculated from Eq. (1) using $P = T_0$, where T_0 is the initial resultant PT forces. As the load increases, gap-opening occurs and the PT tendons elongate as θ_r increases. After gap-opening occurs, the connection rotational stiffness is mainly provided by the PT tendons and can be determined by the depth of the beam, the axial stiffness and arrangement of the PT tendons, and the axial stiffness of the beam [17]. If unloading (point 2) occurs before yielding of the tendons (point 3), the friction force changes direction and dissipates energy until the gap closes at point 5.

The energy dissipation required for the SCPC connection can be quantified using the hysteretic energy dissipation ratio, β_E, which is the ratio of the energy dissipation of the self-centering system to the energy dissipation of a bilinear elastic–plastic system with the same force capacity [19]. Based on Fig. 2(b), β_E can be calculated for design purposes as follow:

$$\beta_E = M_{Ff} / M_{IGO} \quad (2)$$

It can be seen that a larger β_E means that more energy is dissipated by the WFDs. To maintain self-centering ability of the SCPC connection, β_E can range from 0 to 0.5. ACI Innovation Task Group [20] recommends that β_E should be no less than 0.125.

3. Prototype buildings and numerical modeling

3.1. Prototype buildings

In this study, the 4- and 8-story, 3-bay RC-MRFs (referred as RC4 and RC8 here after) created by Haselton and Deierlein [21] are selected as the prototype structures for analyses. The two prototype buildings are designed as special RC moment frames to meet the requirements defined by 2003 International Building Code [22], ASCE 7-02 [23] and ACI 318-02 [24], and they are located on the stiff soil (site class D) in Los Angeles, California. The mapped spectral accelerations for this location are S_s = 1.5 g and S_1 = 0.6 g (g is the acceleration of gravity). Beam strengths were controlled by gravity and lateral force demands, and column strengths were governed by the strong-column weak-beam requirements. Beam transverse reinforcement was controlled by shear capacity design, and column stirrups were controlled by both shear capacity design and confinement requirements. Fig. 3(a) and (b) shows the plan and elevation views of the two buildings, respectively. The building has two identical premier MRFs in two primary directions and the structural responses in the north–south direction are examined in this study. The heights of the 4- and 8-story buildings are 16.6 m and 32.6 m, respectively, with a story height of 4.6 m for the first story and 4.0 m for the others. Major design parameters