REPAIRABLE PRECAST MOMENT-RESISTING BUILDINGS

A. Boudaqa¹ and M. Tazarv²

ABSTRACT

Reinforced Concrete (RC) moment-resisting frames (MRFs) are frequently used in seismic regions as lateral load resisting systems. Even though current design specifications ensure life safety for buildings, damage of structural components is allowed in extreme events. Minor damage may be repaired. Nevertheless, severely structurally damaged buildings are usually replaced, which imposes substantial economic and social costs to the owners and public. An innovative connection was developed for RC MRFs in which all components are precast and the damage is limited to replaceable reinforcement to eliminate the building total replacement. The new joint detailing incorporates (1) detachable external reinforcing steel bars restrained against buckling to develop plastic bending moments, (2) a steel pipe connecting the beam to the column through a pin connection to resist plastic shear forces, and (3) detachable mechanical bar splices to accelerate the construction of MRFs. The seismic performance of the proposed repairable RC connections was investigated through cyclic testing of a half-scale beam-column joint of a nine-story building designed for Los Angeles, which is a high seismic region. A reference cast-in-place beam-column joint was also tested for comparison. The test results showed that the repairable precast connection can withstand more than seven times the design level earthquake with minimal damage and ability to be repaired afterward. The specimen was repaired by simply replacing the detachable reinforcement and was retested with the same loading protocol. The test results were the same before and after the repair. Furthermore, the repairable precast connection showed higher drift capacity than that measured for the reference cast-in-place beam-column joint. The proposed repairable precast MRFs can be constructed as fast as steel MRFs and can be repaired in few hours after severe events without the need to replace the entire structure. The paper and the presentation discuss the new joint detailing and highlights the findings of the experimental and analytical studies.

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Repairable Precast Moment-Resisting Buildings

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ABSTRACT

Reinforced Concrete (RC) moment-resisting frames (MRFs) are frequently used in seismic regions as lateral load resisting systems. Even though current design specifications ensure life safety for buildings, damage of structural components is allowed in extreme events. Minor damage may be repaired. Nevertheless, severely structurally damaged buildings are usually replaced, which imposes substantial economic and social costs to the owners and public. An innovative connection was developed for RC MRFs in which all components are precast and the damage is limited to replaceable reinforcement to eliminate the building total replacement. The new joint detailing incorporates (1) detachable external reinforcing steel bars restrained against buckling to develop plastic bending moments, (2) a steel pipe connecting the beam to the column through a pin connection to resist plastic shear forces, and (3) detachable mechanical bar splices to accelerate the construction of MRFs. The seismic performance of the proposed repairable RC connections was investigated through cyclic testing of a half-scale beam column joint of a nine-story building designed for Los Angeles, which is a high seismic region. A reference cast-in-place beam-column joint was also tested for comparison. The test results showed that the repairable precast connection can withstand more than seven times the design level earthquake with minimal damage and ability to be repaired afterward. The specimen was repaired by simply replacing the detachable reinforcement and was retested with the same loading protocol. The test results were the same before and after the repair. Furthermore, the repairable precast connection showed higher drift capacity than that measured for the reference cast-in-place beam-column joint. The proposed repairable precast MRFs can be constructed as fast as steel MRFs and can be repaired in few hours after severe events without the need to replace the entire structure. The paper and the presentation discuss the new joint detailing and highlights the findings of the experimental and analytical studies.

Introduction

Moment-resisting frames (MRFs) have been extensively used in reinforced concrete (RC) buildings to withstand both gravity and lateral forces in low- to high-seismic regions. Although current codes ensure life safety performance level, data from past earthquakes showed that structural components of RC buildings may experience significant damage after severe earthquakes resulting in total replacement of buildings (EERI Special Earthquake Report, 2011 [1]).

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The structural damage of a well-designed RC MRF under a strong earthquake includes core concrete failure, yielding, and fracture of reinforcement in the plastic hinge region of a few beams and columns. Formation of plastic hinges thus damage is allowed at the ends of beams of a special MRF using current codes (e.g. ASCE 7-16 [2]). Retrofit, strengthening, and repair of existing RC buildings to resist strong earthquakes have been emphasized in the literature in the past two decades. Existing RC joint performance can be improved using fiber-reinforced polymer (FRP) wraps [3] or steel jackets [4]. Although these methods can effectively enhance the performance of RC beam column joints, they may lack the practicability if the architectural requirements of the existing buildings were considered. Moreover, RC structural elements may severely damage after extreme earthquakes which may make the repair impractical imposing financial hardship to the owners and public.

A new construction and design approach for RC moment-resisting frames was developed in which all structural components are precast, the frame is repairable by replacing damaged reinforcement, and the seismic performance of the precast building is better than that for a conventional cast-in-place RC building. First, the detailing of the proposed connection is illustrated for a beam-column joint. Second, the results of a proof-of-concept testing on a half-scale precast beam-column joint incorporating the proposed detailing under cyclic loading are presented. The precast beam was then repaired by only replacing the detachable reinforcement and tested again with the same loading protocol. Furthermore, a cast-in-place (CIP) specimen was tested as the reference specimen. The results of all three tests are compared to comment on the performance of the new joint. Finally, an analytical study was carried out to investigate the performance of cast-in-place and precast frames incorporating the new beam-column connection.

**Proposed Beam-Column Connection Detailing**

Figure 1 shows the main components of the proposed beam-column connection, which includes (1) fully precast beam and column with exposed longitudinal reinforcement, (2) a shear pin made of steel pipe, which was designed based on guidelines proposed by Zaghi and Saiidi [5], to be inserted into a steel cup placed in the column, (3) mechanical bar splices that can be detached, (4) external reinforcement restrained against buckling (Buckling Restrained Reinforcement, BRR) to connect the precast beam reinforcement to the column reinforcement, and (5) a steel plate between the precast column and the beam to prevent damage during rocking. The beam section has to be reduced at the end, which is referred to as “neck” region hereafter, to align BRR with the beam and column reinforcement. All components are designed as capacity protected members except replaceable BRR, which is allowed to yield and fracture. A BRR is a steel bar encased in a steel tube filled with non-shrink grout. The proposed connection is repairable since the exposed reinforcement, BRR, can be replaced after a severe event without the use of any other repair methods such as patching, jacketing, etc.

**Experimental Program**

Two half-scale beam-column joint specimens were constructed and tested at the Lohr Structures Laboratory at South Dakota State University: one CIP specimen as the reference model and one fully precast specimen (Fig. 1). The precast beam-column specimen (referred to as PBC-D) had dog-bone BRR in the connection. After the testing of PBC-D, the specimen was repaired by only replacing the dog-bone reinforcement and was rested (referred to as PBC-D-R). All specimens were tested under slow cyclic loading.
Test Specimen Details and Materials

The CIP specimen was designed and detailed based on a corner joint located in the first floor of a prototype nine-story RC special MRF building. The building was assumed to be located in Los Angeles, CA, which is a high seismic region, and was designed based on ASCE 7-16 [2] and ACI 318-14 [6]. The dimension of the beam and the column for the half-scale CIP specimen were 10×15 in. (254×381 mm) and 15×15 in. (381×381 mm), respectively. The test beam and column lengths were 45 in. (1143 mm) and 72 in. (1829 mm), respectively. ASTM A706 Grade 60 reinforcing steel bars with a yield and ultimate strength of 77.25 ksi (532.6 MPa) and 118.25 ksi (815.3 MPa) were used as the longitudinal and transverse reinforcement in both specimens. The test-day compressive strength of concrete for CIP and PBC-D was 5415 psi (37.3 MPa) and 4889 psi (33.7 MPa), respectively. More information regarding CIP can be found in Tuhin [7].

The precast specimen had the same geometry, reinforcement, and material properties as those in CIP but incorporating the new joint detailing. The depth of the neck was 8.25 in. (210 mm) while the beam depth was 15 in. (381 mm). All BRR were dog-bone made of ASTM A706 Grade 60 reinforcing steel bars encased with ASTM A513 Grade 1026 carbon steel tube and filled with 6226-psi (42.3-MPa) grout. Headed mechanical bar splices were used to connect BRR to the beam and column reinforcement. An ASTM A513 Grade 1026 steel pipe was used as shear pin, which was embedded in the beam to be extended into a steel socket placed in the column.

Test Setup, Loading, and Instrumentation

The specimens were tested under slow cyclic loading using a 22-kip (97.9-kN) actuator as shown in Fig. 2. The column was pinned at the base while the beam support was a roller. This was done to allow the frame to displace laterally. The cyclic loading was adopted from ACI 374.2R-13 [8]. Sixteen strain gauges were installed on CIP and 18 strain gauges were utilized in the precast specimen. An axial load of 68 kips (302.5 kN) were applied to the top of the column using two hollow-core jacks. Load cells and various displacement sensors were used to monitor forces, displacements, and rotations.

Test Results

Figure 3 shows the damage of the specimens at 3.64% drift ratio (equivalent to six times the design
level earthquake for CIP). The drift ratio is defined as the column tip displacement to the column height. It can be seen that CIP damage was significant in which the beam bottom reinforcement fractured at this drift while PBC-D had no damage in the plastic hinge region (one half of the beam depth). The damage of the precast beam where the depth changes was only cosmetic since the force-displacement behavior of the precast specimen remained essentially the same before and after the repair (Fig. 4). Although the precast specimen had lower initial stiffness compared to CIP, it exhibited higher drift capacity with no BRR fracture. CIP failed at 3.64% drift ratio by bar fracture while PBC-D and PBC-D-R withstood 4.18% drift ratio without any signs of failure. The lower initial stiffness of the precast specimens is because of the vertical gap between the steel pipe and the steel socket (cup) at the beam end (Fig. 1), and the shallower beam at the end. Note moment will be developed when the gap is closed. The cosmetic damage at the end of the neck region can be mitigated by a better design and detailing. It was initially assumed that the plastic moment will be fully resisted by BRR (Fig. 1) while a secondary moment, the product of the vertical reaction at the pipe-pin connection and the neck length, should be included in the design of the neck. A minimal longitudinal reinforcement was provided in the neck region of the precast beam while more reinforcement was needed to avoid neck cracking.

Figure 3. Damage at 3.64% drift ratio for (a) CIP, (b) PBC-D, and (c) PBC-D-R.

Figure 4. Force-displacement relationships for CIP, PBC-D, and PBC-D-R.

The strains for both longitudinal and transverse reinforcement were measured using strain gauges. Figure 5 shows the strain profiles for the beam top and bottom reinforcement in PBC-D and CIP. It can be seen that the strain in the precast specimen is well and uniformly distributed while the CIP strains are higher at the beam-column interface. The precast beam longitudinal bar, BRR, yielded at 1.45% drift ratio while CIP beam longitudinal bar yielded at 1.0% drift ratio. The strain data showed no yielding of the column longitudinal and transverse reinforcement. Also, the beam transverse reinforcement did not yield.
An analytical study was carried out to investigate the performance of CIP and precast buildings incorporating the innovative beam-column connection detailing. A nine-story RC special MRF was designed for Los Angeles, CA. Then the proposed detailing was used to make the frame precast. The seismic performance of the CIP and precast frames was investigated through pushover and nonlinear dynamic analysis. The modeling methods and the analytical results are discussed herein.

**Modeling Methods for Proposed Beam-Column Connection**

A three-dimensional fiber-section finite element model was developed to simulate the behavior of the PBC specimens (Fig. 6). OpenSees [9] was used for modeling. The beam model can be generally divided into two submodules. The first module represents the beam neck where yielding and damage of replaceable reinforcement are allowed. The second module is the original beam section. The column was modeled as two elements, one starting from the base to the column mid height and another element starts from the mid-height to where the load was applied. A “forceBeamColumn” element with five integration points was used to model the column and the beam elements in both modules. The stiffness of the shear pin and the gap between the shear pin and the steel cup were modeled as a “gap element” at the beam-column interface. A uniaxial material model, “ReinforcingSteel”, was used to simulate all reinforcing steel bars. The concrete cover and core were modeled using “Concrete01” material model. Figure 7 shows the calculated and the measured force-displacement relationship for PBC. It can be seen that the model reproduced the test data with a reasonable accuracy. Next, this model was used in the nine-story precast frame for further analysis.

The key variable for a precast frame is the BRR fuse length which can be defined as the length of the machined portion of the within BRR. The fuse length was chosen to be 7.5 in. (191 mm) equivalent to 25% of the beam depth. The neck region length was 15 in. (381 mm) larger than the fuse length to accommodate couplers and dowel bars. The gap between the shear pin and
the steel cup is also an important parameter to control the performance. Two precast frame models were developed: one with 0.25-in. (6.5 mm) gap and another without any gap.

Analytical Results

Figure 8a shows the pushover relationship for the nine-story CIP and repairable precast frames. It can be seen that the proposed precast detailing significantly enhanced the displacement capacity of the frame. For example, the displacement capacity of the nine-story precast frame with a 7.5-in. (191-mm) BRR fuse length was 92% higher than the CIP frame. However, the precast frame showed lower initial stiffness. The CIP frame stiffness was 37.38 kip/in. (42.23 N/mm) while the stiffness of the precast frame without gap was 19.8 kip/in. (22.37 N/mm) (47% lower) and with gap was 19.42 kip/in. (21.94 N/mm) (48% lower). The pushover analysis showed that the gap effect on the capacity of the precast frame is negligible.

Figure 8b shows the story drift ratios versus the story number for both CIP and repairable frames. Figure 8c shows the roof displacement history for the CIP and repairable frames under a near-field ground motion (1992 Cape Mendocino Earthquake, Petrolia station) selected by Somerville et al. [10]. It can be seen that the displacement (drift) demands on the precast frames are higher than those for CIP due to lower initial stiffness for the proposed system. For example, the nine-story precast frame with 7.5-in. (191-mm) BRR fuse length and 0.25-in. (6.5-mm) gap had 64% increase in the story drift demand, on average. This increase was 36% when the gap was closed. Nevertheless, the increased demands did not exceed the ASCE drift limit indicating a sufficient design even without increasing the size of members or reinforcement. In summary, the proposed precast detailing can increase the drift capacity of the frame twice, can meet the ASCE drift demand limits, is low-damage, and is repairable by simply replacing the exposed reinforcement (BRR).
An innovative moment-resisting connection was developed for precast buildings with an additional benefit of reparability. The repair is done by simply replacing the damaged reinforcement. It was found that (1) the new beam-column connection can provide higher drift capacity compared to CIP, and (2) the precast buildings can withstand severe earthquakes with minimal damage and ability to be repaired afterward by replacing the damaged reinforcement. This will prevent total replacement of buildings incorporating this detailing. An analytical study was carried out to investigate the seismic performance of the proposed moment-resisting frames. The analytical findings showed that the proposed precast frames have significantly higher displacement capacities, have approximately 50% lower initial stiffness, and may meet the ASCE drift demand requirements without increasing the size of members and reinforcement.

References


