A POTENTIAL VULNERABILITY IN HIGH-STRENGTH REINFORCED CONCRETE SHEAR WALL RETROFITS

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Abstract

In reinforced concrete and steel frames deemed vulnerable to damage during earthquakes, one common retrofit is to fill the space between frames with reinforced concrete walls. These infill walls are meant to both stiffen and strengthen the structure. In some instances, walls can be removed and replaced with stronger walls. This was the case for the story Tohoku University Building of the Faculty of Architecture and Civil Engineering ("Tohoku CE Building"). In 2001, decades after having survived the 1978 Miyagi-Ken-Oki Earthquake, the nine-story Tohoku CE Building was retrofitted using this technique. Following the 2011 Tohoku Earthquake, this building experienced severe damage even though this earthquake caused demands similar to the 1978 earthquake. This performance indicated that high-strength infill panel retrofits may in fact introduce new vulnerabilities, and that detailing of the retrofits is particularly important to ensure satisfactory performance. To study the behavior of this retrofit and possible solutions to shortcomings, two infill-retrofitted frame specimens were tested in the laboratory at Tohoku University. In both specimens, the frames were designed in accordance with older Japanese seismic provisions, and the "retrofit" infill panels were designed in accordance with the Japan Building Disaster Prevention Association (JBDPA) Guidelines. The retrofit followed details similar to those used in the Tohoku University building per the JBDPA Guidelines. Under reversed cyclic loading, the first specimen experienced concentrated deformation at the base of the infill wall which led to failure at relatively small drift demands. The second specimen had the same frame and retrofit details, but also included additional external "bandages" designed to overcome this shortcoming. This specimen sustained larger drift demands before failure. The results of these tests caution engineers that: 1) retrofits that have been tested in one configuration should be approached with caution and thought, and 2) considering deformation compatibility is particularly important when designing retrofits. The results also demonstrate one solution to retrofits that may already possess this shortcoming.

Keywords: reinforced concrete; retrofit; high-strength concrete; infill

1. Introduction

Built circa 1969, the Building of Faculty of Architecture and Engineering at Tohoku University is a nine-story steel-reinforced concrete building with a two-story podium. Columns in this building are reinforced with a combination of reinforcing steel bars as well as structural steel sections connected to each other by rivets. An overview photo of one of the two north-south structural walls from after the 2011 Tohoku Earthquake is shown in Fig. 1 (left). The building survived the 1978 Miyagi-Ken-Oki earthquake, and was subsequently retrofitted in 2001. During this retrofit, the panels of the structural walls were removed and replaced. The new panels contained a larger amount of reinforcement and higher-strength concrete. Nevertheless, during the 2011 Tohoku Earthquake the building suffered severe damage to the structural wall boundary elements Fig. 1 (right). This damage consisted of buckling and fracture of vertical structural steel angles and reinforcement.

In 2014, Wang et al. speculated that the anchorage details used during the 2011 retrofit created a reinforcement discontinuity that led to a concentration of damage that ultimately caused severe damage to the wall boundary elements [1]. This hypothesis was subsequently tested by Wang [2], and Takahashi et al. [3] experimentally. Wang tested scaled structural walls containing deformed bars with vertical reinforcement discontinuities under reversed cyclic loading and concluded, among other things, that a discontinuity in longitudinal web reinforcement in a wall can increase the tensile strain in the boundary reinforcement, resulting in a larger accumulated strain and reinforcement buckling at smaller drift ratios. Takahashi later tested elements which more closely reflected the details of the retrofit walls and boundary columns and concluded that insufficient anchorage of panel reinforcement during the retrofit may have had a negative impact on the performance of the building.

Fig. 1 – (Left) Overview of Building of Faculty of Architecture and Engineering at Tohoku University (from Wang, 2014), and (right) damage observed to a boundary column after the 2011 Tohoku Earthquake.

The use of infill panels to retrofit vulnerable reinforced concrete structures is not uncommon, but the performance of the Tohoku CE building serves as a reminder that detailing of a retrofit is key to its success. The present investigation studies three new questions:

1. What may happen in an ordinary reinforced concrete building retrofitted in a similar manner? i.e., What if the boundary elements did not contain structural steel shapes with rivets?
2. What may happen if there are splices at the story being retrofitted, and what is the effect of the location of those splices?
3. What can be done to improve the performance of a reinforced concrete building already retrofitted by a technique like that used in the Tohoku CE building?

2. Experiment Program

Structural walls and retrofits are often tested in laboratories under constant vertical compression and reversed lateral loads. The vertical compression represents gravity loads, and the lateral loads represent demands from strong ground motions. These conditions capture the conditions that squat structural walls experience in low-rise buildings, but mid-rise buildings like the Tohoku CE building experience more complex loading. Fig. 2 shows a typical equivalent lateral force distribution that may be assumed by an engineer for a mid-rise building. This lateral force distribution gives rise to a combination of shear as well as overturning demands that cause more complex demands in the first story as shown in Fig. 3. In particular, these overturning demands introduce uplift and compression in boundary elements, which are not reflected in laboratory tests of squat walls under constant vertical compression and reversed lateral loads. The testing program used in the present investigation was designed to emulate the more complex conditions a structural wall in a mid-rise building would experience under seismic loads.

Two specimens were tested under reversed cyclic loading. Boundary columns of the specimens were designed based on guidance available in Japan in the mid-1900s. Panels were detailed following modern JBDPA provisions [4]. Details of the specimens, test setup, instrumentation, and program follow.

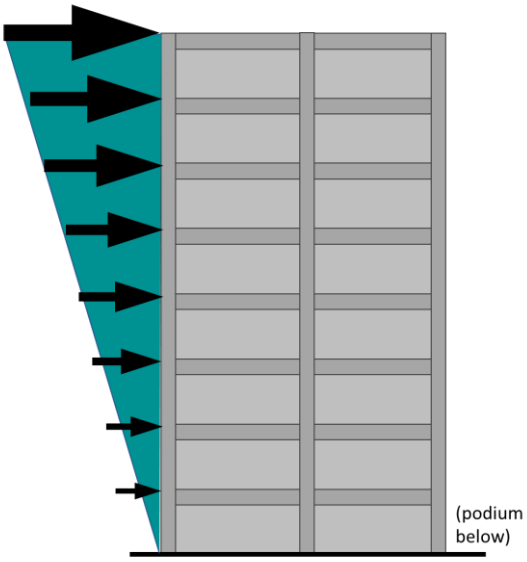


Fig. 2 – Typical assumption of lateral demands for a mid-rise building.

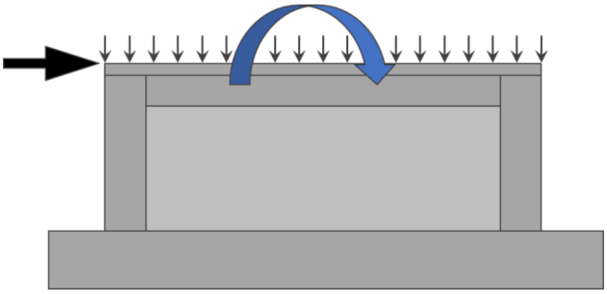


Fig. 3 – Expected demands on first-story wall.

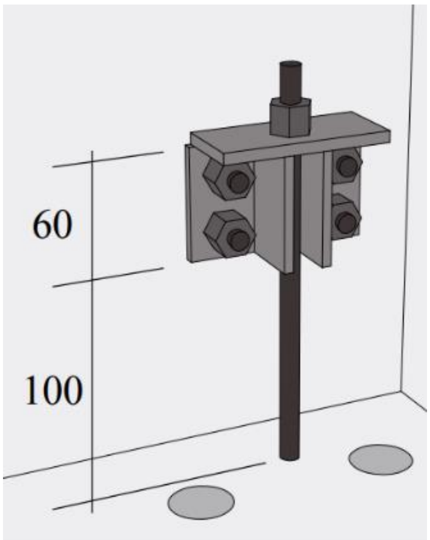
2.1 Specimens

Two specimens were built and tested. Both specimens were structural walls with nominally-identical dimensions, measuring 2000 mm long and 800 mm tall overall. Boundary columns were 200 x 200 mm. Infill panels were 1600 mm x 70 mm. Above and below the walls, enlarged stubs were used to connect the specimens to the loading frame.

The base design of the columns was modeled after a typical mid-1900s frame building in Japan. Towards that, concrete in the columns had a nominal compressive strength of 20 MPa. Unlike the Tohoku CE building which inspired the tests, the specimen did not contain embedded structural steel sections as reinforcement. Rather, conventional deformed reinforcing bars were used throughout to better represent construction elsewhere in the world. The boundary columns were reinforced with sixteen D6 bars. Hoops in these boundary columns consisted of D4 bars with 90-degree hooks, spaced at 80 mm. One additional variable also was studied: splice location in the boundary columns. In both specimens, one boundary column was spliced at its based whereas the other was spliced at mid-height. The splices were both forty bar diameters long.

The wall panels were designed to emulate a retrofit like the one used in the Tohoku CE building, which may also be used to retrofit a bare RC frame. The nominal concrete compressive strength in the panels was 60 MPa. The panels contained two layers of D4 bars spaced at 60 mm in both the vertical and horizontal directions. Like the Tohoku CE building, these bars were not continuous into the adjacent columns or the stubs. To splice these bars, M8 threaded rods were used in between each pair with embedment lengths of seven bar diameters based on JBDPA.

Wall A was based on the details described above. Wall B was nominally identical to Wall A except for one aspect: Wall B contained external retrofits, dubbed here "bandages." The purpose of these bandages was to shift the critical section and engage vertical bars in the panel, similar to the idea of hinge relocation proposed by Paulay and Priestley [5]. An isometric of a typical bandage is shown in Fig. 4 alongside a schematic. Each pair of bandages (referring to opposite sides of the panel) was sized to carry the vertical load from six internal D4 bars in the panel. Each bandage consisted of the following: pairs of L40x40x5 steel angles fastened to opposite faces of the wall panel with four horizontal through-panel 9.2-mm steel threaded rods. A thin layer of epoxy was applied to the surface between the panel and these angles before tightening them to a pre-calculated torque. Between these pairs of angles, there was a single 9.2-mm diameter, high-strength vertical threaded rod anchored into the bottom stub. This vertical threaded rod was clamped down against the angles using a 90x40x6-mm steel plate and high-strength nut. The vertical distance from the stub to this plate was 160 mm, corresponding to forty times the bar diameter of the D4 steel bars used as vertical reinforcement in the panel. A total of nine pairs of these "bandages" were used across the entire bottom of the wall. Because of the moment gradient, it was not necessary to use the same number of bandages across the top: instead, four pairs bandages were used (two pairs at each end).



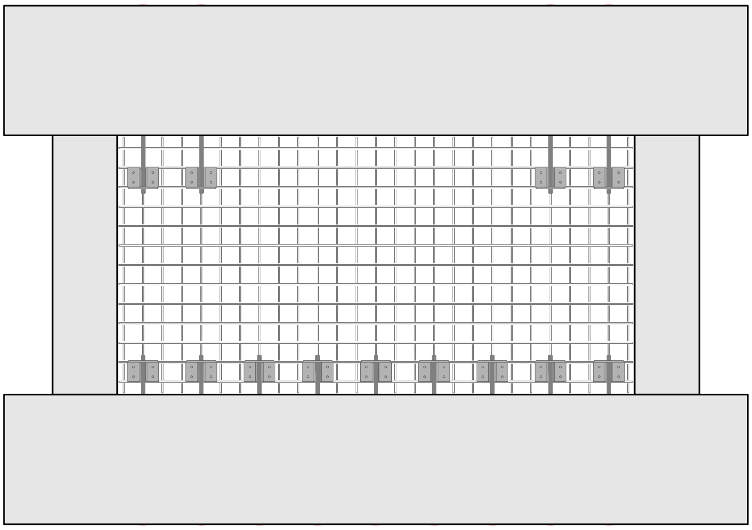


Fig. 4 – Retrofit used for Wall B: (left) photograph of retrofit, and (right) elevation view with panel concrete hidden.

2.2 Setup & Instrumentation

The specimens were tested in a laboratory at Tohoku University. The test setup was instrumented with displacement transducers and strain gages. Strain gages were located to measure: (a) vertical strain of column and panel reinforcement, (b) horizontal strain of panel reinforcement and column hoops, and (c) vertical strain of anchors used to transfer force to the panels. Displacement transducers were located to measure: (a) vertical elongation of both boundary columns along their bottom quarters and middle halves, (b) vertical elongation of the panel upper and lower halves approximately at quarter-points, and (c) relative horizontal and vertical movements of the top and bottom stubs. Displacement transducers intended to measure global displacements were affixed to an instrumentation truss connected to the lower stub as a reference point.

2.3 Testing Protocol

The loading protocol was designed to mimic what the first story of a mid-rise structure may experience during a strong ground motion. It included a combination of variable lateral demands, constant vertical demands from gravity, and variable vertical demands from overturning. Vertical demands from gravity were approximated as 0.15FcAg, where Fc is the compressive strength of the boundary columns and Ag is the gross area of the boundary columns. Vertical and lateral demands were calculated assuming the specimen represented the first story of a multi-story wall with an aspect ratio of 3 (height/width). The loading concept is illustrated in Fig. 5. Additional details are available in Okada et al. [6].

The specimens were tested under increasing demand to failure. The parameter driving the testing protocol was the rotation of the upper stub relative to the lower stub, as measured by vertical displacement transducers between the two. The loading program was modeled after FEMA 461 [7], with the amplitude of loading starting below a value at which damage would be expected and increasing by a factor of 1.4 with subsequent loads. Each amplitude was repeated before moving onward to a larger amplitude.

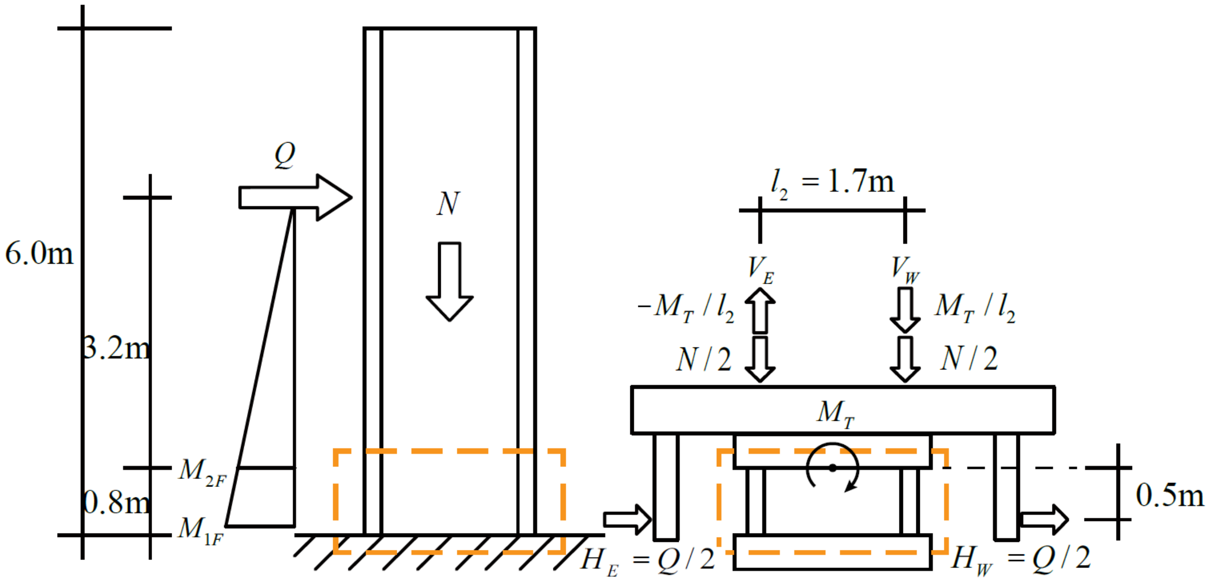


Fig. 5 – Loading concept (from Okada et al. [6]).

3. Results & Discussion

3.1 Wall A

Fig. 6 shows a photograph of Wall A in the testing setup, with locations of the splices in the boundary columns indicated. Positive loading was defined such that the boundary column with the bottom splice was in in tension. Negative loading put the middle splice in tension.

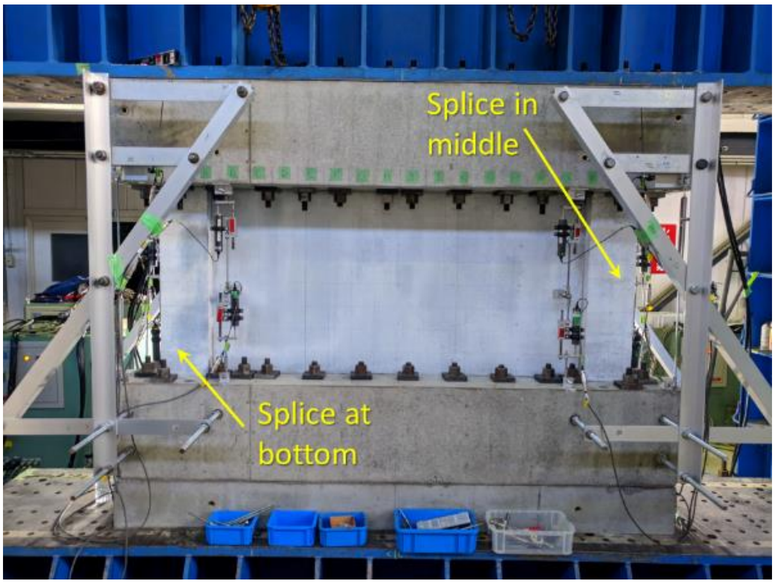
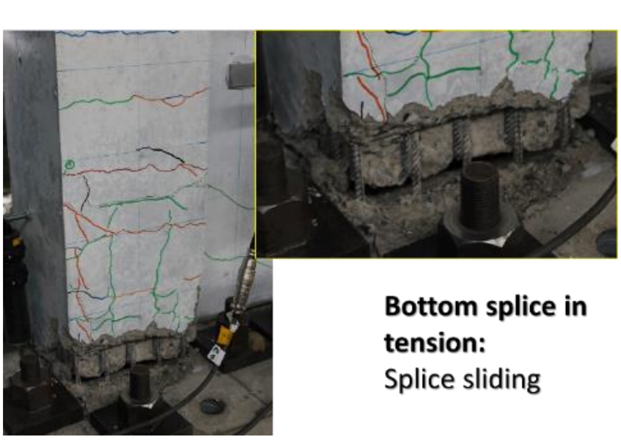


Fig. 6 – Wall A in testing rig, with locations of splices indicated.

Wall A showed a concentration of damage at its base. Minimal cracking was observed along the height of the wall panel, with largest cracks observed at the base near the stub-wall interface. In boundary columns, there was cracking along the height but the largest cracks concentrated at the base. Fig. 7(a)(left) shows an overview of positive loading of Wall A, where light is visible beneath the left side of the wall panel as a result of concentration of damage at the base. This light was visible beneath the panel at top rotations as little as 0.27%. Concentration of damage at the base of the wall panel and boundary element ultimately led to the anchor pullout as shown in Fig. 7(a)(right) in both directions of loading.

Photographs of the boundary elements of Wall A during the latter stages of positive loading are shown in Fig. 7(b) and Fig. 7(c). Fig. 7(b), where the bottom splice is in tension, shows a general sliding failure of the upper vertical bars past the lower vertical bars, which limited the strength of the wall. Fig. 7(c) shows the opposite side in compression, where bars were observed to have buckled as the specimen was loaded from +0.5% to +0.7%. This damage was like what had been observed of the Tohoku CE building following the 2011 Earthquake (Fig. 8). Under negative loading, Wall A continued to exhibit a concentration of damage at its base. For the column with the splice at mid-height, bars which previously buckled [Fig. 7(c)] elongated and eventually fractured (Fig. 9), again causing a decrease in the strength of the wall.

(a) (left) Overview of positive loading of Wall A, (right) showing pullout of anchors.



(b) Tensile side during positive loading.

(c) Compressive side during positive loading.

Fig. 7 – Positive loading of Wall A.

Fig. 8 – Comparison of Wall A and Tohoku CE Building.

Fig. 9 – Negative loading of Wall A, which led to fracture of vertical bars.

Wall B was subjected to the same loading protocol as Wall A. The only difference between the two was the presence of “bandages” on Wall B. These bandages were intended to shift the critical section of Wall B from the stub to a height at which the vertical reinforcement in the panels would be engaged, as illustrated in Fig. 10. Wall B performed better than Wall A. Rather than concentrating solely at the base, cracking distributed over more of the boundary columns and infill panel. As a result, steel in the panel helped resist applied demands. A comparison of force versus top rotation curves is presented in Fig. 11. Wall A is shown in black and Wall B in red. This figure shows that not only did Wall B develop larger strengths, but it maintained its strength at larger top rotations than Wall A.

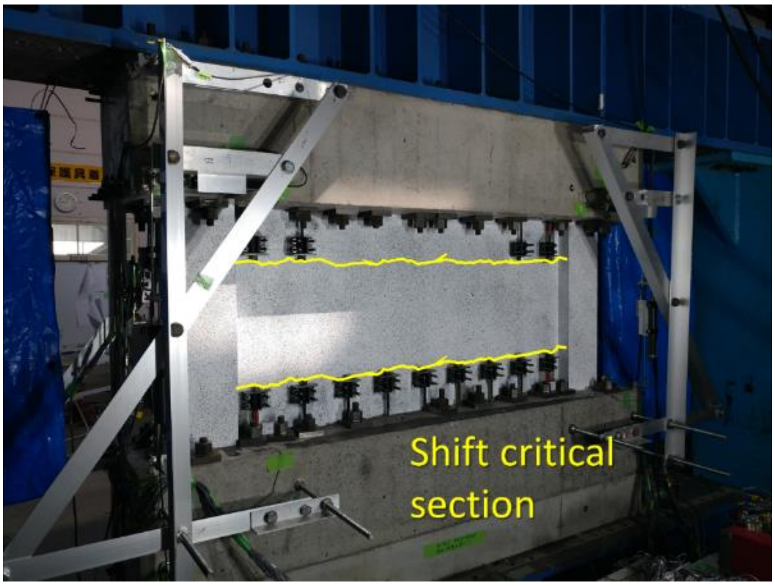


Fig. 10 – Concept of shifted critical section with Wall B.

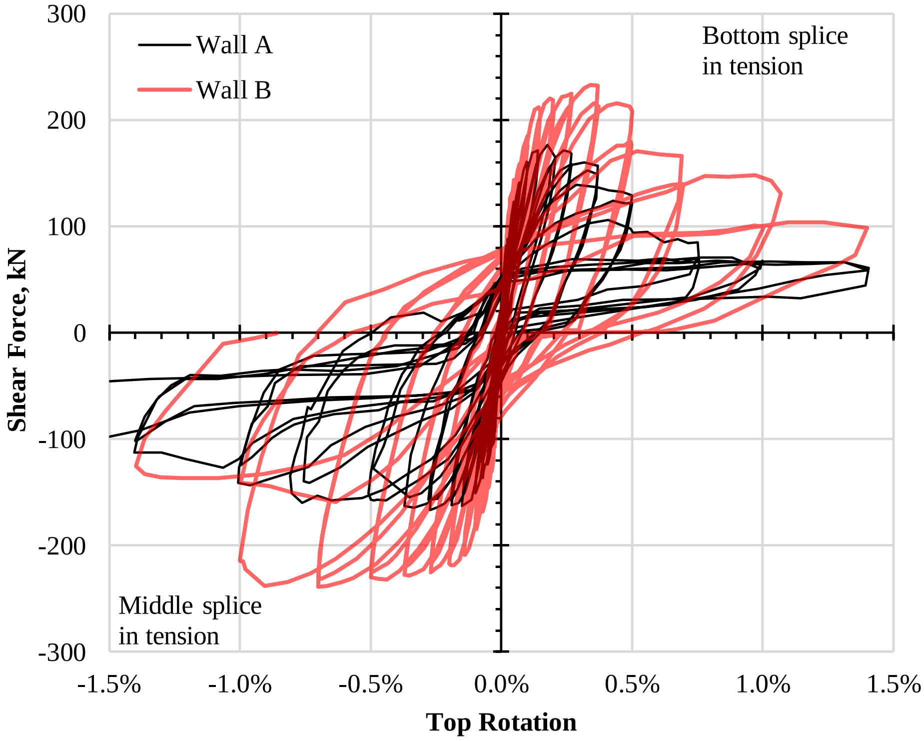


Fig. 11 – Force versus top rotation for Wall A and B.

4. Conclusions

Reinforced concrete buildings constructed in the past are often retrofitted to increase their capacity against seismic actions. These retrofits are intended to increase strength and deformation capacity. When designing retrofits for such buildings, special attention needs to be paid to detailing to ensure that they provide continuity. Codes and specifications are targeted towards doing so, but failures such as the Tohoku CE building in 2011 show that improvements can be made to details in these Codes. These failures also highlight the need for testing of specimens under conditions which represent better the loading conditions actual buildings will experience. In this investigation, two structural wall specimens representing a retrofitted first-story of a mid-rise reinforced concrete building were tested under reversed cyclic loading to failure. The first wall (A) contained conventional retrofit details, but the second wall (B) contained additional retrofits intended to ensure continuity of reinforcement. Under the same loading conditions, the second wall was 30-40% stronger and had larger displacement capacity. This wall also avoided issues with strain concentration at its base, which has been cited as a reason for the severe damage observed for the Tohoku CE building. The study highlights one possible retrofit to buildings facing similar issues, as well as the importance of tailoring testing protocol to the actual conditions in which a retrofit is expected to be used.

5. References

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