Scientific paper

Seismic Damage of a Building Caused by Post-installed Anchors Intended to Increase Shear Strength of Structural Wall

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Abstract

A building was severely damaged during the 2011 Tohoku Earthquake in spite of having been strengthened before the event and having withstood a comparable event before with only minor damage. An experimental study was conducted to test the hypothesis that damage to the building may have been triggered by cone breakout failure of anchors installed during the strengthening program leading to a strain concentration at the third story. Specimens were tested which represented portions of shear wall boundary columns either before or after the strengthening program. Observed damage in the specimens representing the boundary columns after strengthening was similar to the damage observed in the building after the 2011 Tohoku earthquake. Projections of the test results suggest that the strengthening program may have reduced the drift capacity of the building.

1. Introduction

Building retrofits have been common in Japan since the 1995 Kobe Earthquake. Surveys conducted after the 2011 Tohoku earthquake show that most retrofits have proven effective (Kabeyasawa 2012). Nevertheless, there are exceptions. Built in 1969, the building of the Faculty of Architecture and Civil Engineering at Tohoku University is one such exception. This building was a nine-story steel-reinforced concrete (SRC) structure with a two-story podium (Fig. 1). In the north-south direction, lateral resistance was provided by reinforced concrete shear walls with boundary columns. In 1978, the building withstood the Miyagi-Ken-Oki earthquake with light structural damage (Sato et al. 1981). In 2001, the building was strengthened by replacing the shear wall webs from the third story up with wall webs consisting of higher strength concrete ($f'_c = 67$ MPa). The strengthening program also involved removing existing vertical reinforcement in the wall webs and replacing it with a larger portion of vertical reinforcement, increasing the reinforcement ratio from 0.25% to 0.68%. The removal of existing vertical reinforcement created a discontinuity at the beam-wall web joint. To provide continuity and anchor the new reinforcement, the replacement wall webs were connected to the original structure using post-installed anchors glued using epoxy.

The ground motion intensity during the 2011 earthquake was similar to the ground motion intensity during the 1978 earthquake. Furthermore, the drift demands on the structure – computed using measurements from accelerometers on the first and ninth floors – were within 40% of one another: 225 mm in 1978 compared with 310 mm in 2011 (Suzuki et al. 2013). Nevertheless, during the 2011 Tohoku earthquake the building sustained severe structural damage. This damage was most severe at the bottoms of the shear walls on the third story, particularly in the boundary columns (Suzuki et al. 2013; Wang et al. 2016). Examples of this damage are shown in

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Fig. 1 Overview of the building.
Fig. 2, with concrete disintegrated and longitudinal steel bars and angles buckled and/or fractured. In addition to this damage, the anchors installed in 2001 were observed to have experienced cone breakout failures. Wang et al. (2016) suggested that discontinuous web reinforcement as a result of the 2001 strengthening program led to a weak (nearly unreinforced) plane at the base of the third story webs where deformation concentrated. They postulated that concentration of deformations at this location led to strain concentration in the reinforcement, and subsequently decreased the drift capacity of the building. The present investigation provides additional insight into the effect of the building strengthening program on the performance of the building through the testing of eight specimens and the analysis of numerical models.

2. Estimated seismic response of building

The specimens tested in this investigation represented scaled portions of boundary columns. The loading protocol of the specimens was based on the expected axial deformations of the boundary columns during the 2011 event. Using measurements obtained from accelerometers on the first and ninth floors, the peak drift of the ninth floor was estimated to be 310 mm in 2011 (Suzuki et al. 2013). In addition, the peak of the acceleration response spectrum appears around 1 second of natural period, and as the increase of natural period, the acceleration response rapidly decreases (Suzuki et al. 2013). It indicates that the effects of higher modes were not relevant in this building. To estimate axial deformations in the boundary columns on the third story corresponding to this drift, the following assumptions were made:

1. From the third floor to the roof, the shear wall rotated as a rigid body (Fig. 3).
2. Because post-installed anchors in the wall panels [Fig. 4(a)] experienced cone failures [Fig. 4(b)], the tensile contributions from vertical reinforcement in the wall panels were neglected.
3. The gravity load supported by the wall in the third story was estimated to be 6100 kN (including finishing materials and live load) using data from the strengthening program.
4. Concrete compressive strengths of the column and the wall panel were assumed to be 14 MPa and 67 MPa from tests of drilled cores, respectively.

Using these assumptions, upper and lower bound estimates were made of axial deformations in the boundary columns (Table 1). For the maximum elongation case (i.e. upper bound), the following additional assumptions and procedure were used:

1. Longitudinal steel angles and reinforcing bars were fractured and could not carry tension.
2. The maximum acceleration in the vertical direction was 0.64 g (Motosaka et al. 2012). To account for the effect of this vertical motion, an uplift force of 3800 kN (6100 kN × 0.64 g) was assumed. It is not probable that the maximum acceleration in both horizontal and vertical directions are recorded simultaneously. However, to estimate the upper bound, this assumption is used in this study.
3. The compression zone carried an axial force equal to the sum of: (a) the structure weight, (b) the tension in the columns, (c) resisting force in the orthogonal beams and, (d) uplift force from the vertical component of the ground motion:
The neutral axis depth was computed using this applied axial force (2300 kN).

It was assumed that shear walls above the third story rotated about this neutral axis until a displacement of 310 mm was reached at the ninth floor (Fig. 3). At this displacement, the corresponding elongation and shortening of each column at its centroid was computed (Table 1). A similar procedure was used to estimate the minimum elongation (i.e. lower bound). Differences in assumptions used when calculating minimum elongation are summarized in Table 1. Three key differences are:

1. Steel in the boundary columns was assumed to resist tension,
2. The direction of vertical acceleration was downwards, and
3. Orthogonal beams connecting exterior columns to the nearest row of interior columns resisted uplift of the wall. Assuming that these beams yielded at both ends, the total resisting force exerted on the exterior wall by the beams above the third story was estimated to be 2

\[ N_{min} = 6100 \text{ kN} + 0 \text{ kN} + 0 \text{ kN} - 3800 \text{ kN} = 2300 \text{ kN} \] (1)

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**Table 1 Elongation and compression of the boundary columns.**

| Case           | Sum of tensile force in columns | Effect of vertical motion | Compressive strength of Boundary column | Neutral axis to the center of column in comp. | Elongation (boundary) | Elongation (center) | Compression (boundary) |
|----------------|---------------------------------|---------------------------|----------------------------------------|-----------------------------------------------|-----------------------|---------------------|----------------------|
| Maximum elongation | 0                               | -3,800 kN                 | 9,100 kN                               | 0.16 m                                        | 204 mm (6.7%)         | 100 mm (3.3%)       | 2 mm (0.08%)         |
| Minimum elongation | 4,600 kN                         | +3,800 kN                 | 0                                      | 1.81 m                                        | 179 mm (6.0%)         | 75 mm (2.5%)        | 28 mm (0.92%)        |

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![Fig. 3 Assumed deformation of the building during the 2011 earthquake.](image)

![Fig. 4 Drawing and observed damage of post-installed anchors.](image)
Using this procedure, the elongation of the third-story boundary column was estimated to range between 180 mm to 200 mm (average strain of 6% to 7%), and the shortening was estimated to range between nearly zero and 30 mm (average strain of 0.1% to 0.9%).

### 3. Experiment

#### 3.1 Specimens

Because the components of interest in this investigation were the third-story boundary columns at the corners of the floor plan – which were expected to be in nearly uniform tension or compression – it was not deemed necessary to construct the entire wall. In addition, the effect of shear force can be assumed not so significant to the observed damage because no shear crack was observed on the wall panel [Fig. 4(b)] and discontinuity between the wall panel and the beam was not observed. Therefore, specimens representing portions of boundary columns were used to approximate the behavior of the full-scale columns. The longitudinal reinforcement in these specimens represented longitudinal reinforcement in both the boundary column and in the wall panels. It was essential to include vertical reinforcement from the wall panels in the specimens because insufficient anchorage of this reinforcement is thought to have led to strain concentrations at the base of the wall and subsequent failure of the boundary columns.

The design basis for the specimens is shown in Table 2. The first column in Table 2 shows cross-sections of half the exterior shear wall, both before and after strengthening. Bars marked with an “X” are thought to have been insufficiently anchored as a result of the strengthening program. Making use of symmetry, the second column in Table 2 shows cross-sections of one-quarter of the boundary column and one-quarter of the wall panel reinforcing steel. The test specimens were one-third scale representations of these one-quarter sections (as shown in the third column in Table 3).

As shown in Table 2, four pairs of angles were provided in the building boundary column. Each pair of angles was connected to the opposite and adjacent pairs of angles using tie plates (opposite) and band plates.
adjacent). Tie plates were connected to the angles with rivets. Band plates were connected to the angles with welds. Tie plates were provided in the specimens because their effect on restraining buckling of steel angles was not negligible. In contrast, transverse hoops made with deformed reinforcing bars and band plates were not provided in the specimens because their capacity to restrain buckling of reinforcing bars was considered negligible, as discussed below.

Figure 5 compares the effectiveness at restraining reinforcing bar buckling of (a) concrete and (b) hoops made with reinforcing bars. To estimate the resistance provided by concrete, the tensile strength of concrete, \( \sigma_T \), was estimated as:

\[
\sigma_T = 0.33\sqrt{f'_c} = 0.33\sqrt{12.6} \approx 1.2 \text{ (MPa)}
\]

(2)

where \( f'_c \) is concrete compressive strength, which was taken as 12.6 MPa from tests of cores from the building. The lateral restraining force provided by the concrete, \( T_c \), was computed as:

\[
T_c = \sigma_T A_c = 1.2 \text{ (MPa)} \times 210 \text{ (mm)} \times 150 \text{ (mm)}
\approx 38 \times 10^3 \text{ (N)} = 38 \text{ (kN)}
\]

(3)

where \( A_c \) is the area of concrete, which was taken as 210 mm multiplied by the hoop spacing, 150 mm, for comparison with the restraining force from one hoop [Fig. 5(a)]. The restraining force provided by each hoop, \( T_s \), was computed as:

\[
T_s = A_v \sigma_{yv} \cos \theta \approx 200 \text{ (MPa)} \times \sqrt{2} \approx 18 \times 10^3 \text{ (N)} = 18 \text{ (kN)}
\]

(4)

where \( A_v \) is the area of a single leg of hoop reinforcement (\( \phi 9 \)), \( \sigma_{yv} \) is the yield strength of the hoops, and \( \theta \) is inclination angle shown in Fig. 4. These calculations suggest that the confining effect of concrete was more than twice that of the hoops.

Figure 6 shows a typical specimen without vertical reinforcing bars. The height of the column specimen (500 mm) is one-third the height of the lower half of the original column (broken line in Fig. 3). To anchor the tie plate in the concrete, two M6 bolts were used [Fig. 6(b)]. Bolts were also used in place of rivets to connect the tie plate to the steel angles [Fig. 6(b)]. The plate in Fig. 6(c) represents the steel plate in the original beam. These plates and bolts are provided in the specimen because strain may have concentrated at the rivet hole in the angle.

Eight specimens were tested. All specimens had the same nominal dimensions. The concrete used had an average compressive strength of 22.5 MPa, an average tensile strength of 1.6 MPa, and an average Young’s modulus of 22.4 GPa. All specimens had two embedded L25x25 angles. These angles had a yield stress of 362 MPa, a strength of 477 MPa, and a Young’s modulus of 209 GPa. The specimens differed in the amount and anchorage of vertical reinforcing bars, as summarized in the lower part of Table 3. Each specimen was assigned an ID in the format CXY, where X denoted the number of vertical bars with sufficient anchorage and Y denoted the number of vertical bars with insufficient anchorage (shown in parenthesis in Table 3). Reinforcing bars with insufficient anchorage were embedded 30 mm into the lower stub (Fig. 7). As shown in Fig. 4(a), the embed-
ment length was 110 mm in the case of the actual building whereas the requirement of the Japanese guideline is 91 mm to resist shear force. In the scaled specimen, the required embedment length is almost 30 mm which was used in this experiment. All other reinforcement and steel angles were welded to plates in the stubs to ensure sufficient anchorage. All vertical bars were D6, with a yield stress of 340 MPa, a strength of 528 MPa, and a Young’s modulus of 181 GPa. In addition to C-series specimens, one specimen (S) was built with no concrete to determine the behavior of the steel alone.

3.2 Test Setup
During the earthquake, the boundary columns were expected to be in nearly uniform compressive or tensile strain. To simulate this, axial load reversals were applied to each specimen using two hydraulic jacks (Fig. 8). Specimen elongation was measured using four LVDTs placed in-line with these jacks (Fig. 8).

3.3 Test Results
Measured load-deformation envelopes are shown in Fig. 9 for mean tensile unit strains. Figure 9(a) shows specimens in which all bars had sufficient anchorage. Figure 9(b) shows specimens containing four bars with insufficient anchorage. The sudden drop in tensile force for all specimens corresponds to fracture of the steel angles, which typically occurred at the rivet hole. In specimens where all bars had sufficient anchorage, this occurred at a mean strain of 4% to 5%. In specimens containing bars with insufficient anchorage, this occurred at a lower mean strain (between 3% and 4%). These results suggest that the presence of bars with insufficient anchorage decreased the deformation capacity of the boundary column. Especially, the comparison between C40 and C44 indicates that insufficient anchorage just decreases elongation capacity. To explore this idea and the failure modes of the two specimen types, detailed results from two specimens C60 and C44 are discussed in the following sections because they represent the third story column before and after retrofit, respectively.

(1) Specimen C60
Specimen C60 represented the boundary column before strengthening. Figures 10(a) and 11(a) show the load-deformation relationship measured for this specimen. Circled numbers indicate the loading cycle. The loading rule is also shown in Fig. 10. The specimens were controlled by strain in the tensile side (positive side) of loading. In the compressive side (negative side) of loading, the specimens were controlled by the axial force. The axial force (300 kN) was determined by the axial force (10100 kN) in Fig. 3.

\[
P = N \times \frac{1}{4} \times \left(\frac{1}{3}\right)^2 = \frac{10100\text{kN}}{36} \approx 300\text{kN}
\]
where \( N \) is axial force in the third story building column, the factor 1/3 is because the specimen is a one-third scaled model, and the factor 1/4 is because the specimen is one-quarter of the boundary column. However, if the specimen reached an average compressive strain of \(-0.05\%\) before it reached an axial force of 300 kN, the axial force was reversed.

The dashed line in Fig. 10 shows the expected strength, which was calculated as the sum of the strength of specimen S (which consisted solely of the angle assemblies) and the yield strength of the longitudinal reinforcement in the column. Similarly, the dashed line in Fig. 11 indicates the calculated load-deformation relationship, which was computed as the sum of the load-deformation relationship of specimen S and the vertical reinforcing bars (assuming uniform strain in these bars). The dashed line in Fig. 11 in the compression side was obtained considering the contribution of concrete. The calculations agreed with the test results in terms of both strength and stiffness.

Strain gauges were installed to the angles and the bars, respectively, at the positions indicated by the solid and open squares in Fig. 12(a) which shows the strain distribution of specimen C60 at a mean strain of 0.67\%. The histogram in Fig. 12(a) shows the crack width of the specimen. In both the angles and reinforcing bars, strain concentrated near the rivet holes. Likewise, transverse cracks were widest in this region [Fig. 12(a)]. At this strain, a longitudinal crack was also observed (“a” in Fig. 12(a)). This type of crack is often associated with bond stress (Goto 1971), and suggests vertical reinforcement was experiencing high bond stresses at this time.

During the sixth cycle, starting at a mean strain of 5\%, the applied load began to decrease gradually. In specimen S, this corresponded to the strain at which cracks began to form at the edges of rivet holes. At the peak strain of the next cycle (6.6\%), wide transverse cracks were observed along the height of the specimen, and the longitudinal crack first observed at 0.67\% strain began to propagate [Figs. 13(a) and 13(b)]. The propagation of this longitudinal crack suggests that the concrete may have lost its confining effect at this time. During the subsequent compressive load cycle, the vertical reinforcement buckled although the specimen had a net tensile strain of 5\% [Figs. 13(c) and 10(a)]. This observation highlights that reinforcement bar restraint is not the only factor affecting bar buckling: repeated cycles in tension and compression also play a role. A similar phenomenon was observed in previous research (Mayor and Kowalsky 2003). Fractured steel angles and buckled reinforcing bars were both visible in specimen C60 at the end of the test [Figs. 13(d) and 13(e)].
(2) Specimen C44
Specimen C44 represented the boundary column after strengthening. Figure 10(b) shows the load-deformation relationship of specimen C44. The dashed line is calculated strength. When calculating this value, the contribution of vertical reinforcement with poor anchorage was neglected. This is because bars with poor anchorage were expected to pull out before yielding. The calculated strength showed good agreement with measured strength, suggesting this assumption was reasonable.

Figure 12(b) shows the strain distribution and crack pattern at 0.67% average tensile strain. Unlike specimen C60, where strain concentrations were observed at the top, bottom, and around rivet holes, in specimen C44 strain concentration was observed only at the bottom.

During the sixth cycle, tensile axial load began to decrease before the peak axial load of the previous cycle was reached [Fig. 10(b)]. Soon after, the steel angles fractured at an average strain of 3.8%. At an average strain of 5%, the load decreased again, indicating fracture of a reinforcing bar. At an average strain of 6.6%, the load decreased again, indicating fracture of another bar. Figure 14(a) shows specimen C44 at an average strain of 10%. The specimen uplifted as a result of strain concentration at the bottom. Fracture of the steel angles is also visible at this strain [Fig. 14(b)]. During the subsequent load reversal, the steel angles and vertical reinforcement buckled.

Specimen C44 at the end of the test is shown in Fig. 15. Buckled steel angles and buckled vertical reinforcement spalled the concrete [Figs. 15(b) and 15(c)]. This damage was similar to what was observed in the building (Fig. 2). Based on the similarities in damage, it is plausible that the failure observed in the building was caused by a process similar to the one observed in specimen C44. This process can be summarized as: (1) cone failure of post-installed anchors, (2) strain concentration at the column base, (3) fracture of steel angles at rivet holes in the beam-column joint, (4) pull-out of angles and fracture of bars, and (5) buckling of steel angles and longitudinal reinforcement.

3.4 Uniformity of Elongation
The elongation of all specimens except C00 and S was measured along three gauge lengths as shown in Fig. 16(a), where the number in each parenthesis represents...
the initial gauge length. The elongation of the three lengths are denoted $e_1$, $e_2$, and $e_3$. For specimens C44 and C04, the elongation of the lower length, $e_3$, could not be measured because of cracks on the surface of the lower stub, and was instead estimated by subtracting $e_1$ and $e_2$ from the total elongation $\Delta L$.

The circles in Figs. 16(b) and 16(c) are measured peak elongation in that region at each loading cycle. The dashed lines show elongation calculated assuming strain was uniform in that region. The plots show that the strain distribution in specimen C60 was nearly uniform, while strain concentrated at the bottom of C44.

4. Projection of Results to Third Story Boundary Column

Assuming the properties of the materials, welds, and rivets were similar in the test specimens and the studied building, the test results were projected to the building to study the hypothesis that reinforcement discontinuity caused by the strengthening program led to strain concentration that ultimately led to the failure of 2011. The specimens represented the lower half of the third story boundary columns, so to estimate the axial load-deformation relationship of the entire third story column, the deformation of the upper half of the column needed to be estimated and added to the measured deformations. For simplicity, the strain in the upper half of the column was assumed to be equal to the strain in the upper length of each specimen. Considering the scale of
the specimen, the total elongation of the third-story building column, $\delta$ was estimated as:

$$\delta = \frac{e_1}{220} \times 1500 + 3 \times (e_1 + e_2 + e_3)$$

(6)

where $e_1$, $e_2$, and $e_3$ are elongations in the top, middle, and bottom lengths of each specimen, $e_1/220$ mm is the average strain in the upper length of each test specimen (assumed to represent the strain in the upper half of the building column), 1500 mm is the length of the upper half of the building column, and the multiplier 3 is the geometric scale factor used. To estimate axial load in the building column, the left part of equation 5 was solved for $N$:

$$N = 36P$$

(7)

where $P$ is the axial force applied to the test specimens. Using equations 6 and 7, the estimated load-deformation relationship of the third-story column was generated (Fig. 17).

Figure 17(a) was projected from specimens containing bars with sufficient anchorage (i.e. pre-strengthening). Figure 17(b) was projected from specimens containing bars with insufficient anchorage (i.e. post-strengthening). The top x-axes show estimated drift corresponding to that axial elongation. These drifts were calculated assuming that the length between the center of the boundary column (in tension) and the neutral axis was 12 m which is almost the average of the two cases shown in Table 1. The gray regions are the estimated upper and lower bound elongations of the third-story column (from Table 1). The deformation capacity of the column was almost 0.5% and the estimated elongation of the center and the boundary columns were beyond this value. It is not known with certainty the extent to which the details of the specimens were representative of details in the building, but the numbers suggest that it is plausible that the drift capacity decreased as a result of the strengthening program. To examine this idea in more detail, numerical models of the building were made.

4.1 Static Numerical Analyses

A static, three-dimensional analysis was carried out using Super Build/SS3 (v1.1.1.34) (UNION SYSTEM Inc. 2014). In this analysis, each beam and column was represented by a line element with nonlinear rotational springs at both ends and a nonlinear shear spring at the center. Each shear wall with boundary columns was represented by an element with three vertical components, as proposed by Kabeyasawa et al. (1983). Properties of the springs were determined according to Japanese guidelines (MLIT 2007).

Three analysis cases were considered: before strengthening (model B), after strengthening (model A), and as intended in the strengthening program (model P). For all models, the strengths and elongation capacities of the third story columns were inferred from the test results shown in Fig. 18. In the case of the model A, the vertical
reinforcements in the wall panel were neglected because they will not work due to the pull-out of the post-installed anchors. The following procedure was used to idealize the effects of fractures:

1. Lateral force was applied to the models so that the story shear force was proportional to $A_i$:

$$A_i = 1 + \left( \frac{1}{\alpha_i} - \alpha_i \right) \frac{2T}{1 + 3T}$$

(8)

where $\alpha_i$ is the ratio of the weight above the $i$-th story to the weight of the whole building, and $T$ is the fundamental natural period of the building estimated as $0.02h$ ($h$: building height), with $h$ in m and $T$ in sec (Ishiyama 1988).

2. If elongation of any third story column reached 45 mm in model A [Fig. 18(b)] or 180 mm in model B and P [Fig. 18(a)], the steel angles in the column were assumed to fracture and the model was updated to exclude the fractured angles. If column elongation exceeded 80 mm in model A, vertical reinforcing bars were assumed to fracture.

(1) Model A Results
The sequence of damage inferred from this analysis is illustrated in Fig. 19. Failures of the boundary columns were inferred to have occurred on the west side of the building first, causing torsion. The observed difference in damage between the east and west façades may be attributable to such torsion.

(2) Comparison of All Models
Comparisons of the deformed shapes of building models A, B, and P at similar roof drifts are presented in Fig. 20. Also shown in this figure are the failures that took place throughout the building models at this drift. In model B, shear dominated the response. In model P, which represented the intended behavior of the building after strengthening, failures in the shear walls were prevented and flexural deformation of the shear walls was observed instead (on axes Y2 and Y3). In model A, which represented the expected behavior of the building after strengthening using results of this study, damage was most prominent in the third story boundary columns, with no damage above this story. This is consistent with what was observed in the field after the 2011 earthquake (Wang et al. 2016).

Load-displacement relationships of all models are shown in Fig. 21. Based on the record of the accelerometer on the ninth floor, Wang et al. (2016) constructed...
the relationship between the acceleration and the displacement at the accelerometer and showed that the relationship included sudden decreases of the acceleration. The solid line in Fig. 21 illustrates the load-displacement relationship produced by Model A representing the structure after the strengthening. The vertical axis of Fig. 21 represents the acceleration of the ninth floor defined by the lateral force divided by the weight of the floor. The horizontal axis represents the displacement at the accelerometer. The black circles in Fig. 21 show the fractures in corner column X3Y2. After the angles fractured in the column, the bars fractured at smaller displacement. This phenomenon is caused by the decrease of the elastic deformation illustrated in Fig. 22, where Spring A represents the deformation of the bottom part of the wall and Spring B represents that of the remaining part of the wall. The fractures in column X3Y3 also occurred at displacement smaller than that of the first fracture. In contrast, models B and P (the broken lines in Fig. 21) did not experience such a sudden failure (fracture of the angles or the bars). Overall, the numerical models considered here indicated that the deformation capacity of Model A, which represented the building after strengthening, was smaller than Models B and P and smaller than the measured demand (310 mm).

5. Conclusions

During the 2011 Tohoku earthquake, a nine-story steel-reinforced concrete (SRC) building with shear walls was damaged severely in spite of having been strengthened beforehand. In the strengthening program, shear wall webs were replaced and anchors were installed using epoxy. After the earthquake, damage was observed to concentrate on the third story (above a two-story podium) at the bottoms of boundary columns and shear wall webs (where anchors were installed). In this investigation, experiments were conducted to explore the idea that the strengthening technique used on the building led to a strain concentration at the third story and an unintended reduction in drift capacity. Observations from the experimental investigation are as follows:

(1) In specimens representing the boundary column and wall panel before the strengthening program, tensile strain distributed nearly uniformly along the full height of the specimens. The steel angles in these specimens fractured at an average strain of 5%, and buckled during subsequent load reversals.

(2) In specimens representing the boundary column and wall panel after the strengthening program, some reinforcing bars were deliberately anchored insufficiently. This was to represent cone breakout failures of post-installed anchors in the wall panels, which were observed after the 2011 earthquake. This insufficient anchorage led to strain concentration at the bottom of the specimens, which paralleled the damage observed at the bottoms of third-story columns in the building. This strain concentration led the steel angles to fracture at a lower average strain than pre-retrofit specimens (3% versus 5%). The fractured angles pulled out during the tension loading cycle, and then buckled during the subsequent load reversal.

These experimental results suggest that insufficient anchorage of wall vertical reinforcement may have led to strain concentration at the bases of the boundary columns, which may have led in turn to fracture and buckling of the embedded steel angles. Assuming the properties of the materials, welds, and rivets were similar in the test specimens and the studied building, the experimental results from each specimen were projected to the building. For cases representing the building before strengthening, the projected results show failure at drift ratios larger than 1%. For cases representing the building after strengthening, the projected results show failure at drift ratios smaller than 0.5%. Numerical models were built in Super Build/SS3 to examine the response of the building in more detail. Both the models and the experimental results show that it is plausible that the
strengthening program decreased the drift capacity of the building. This unintended reduction in drift capacity can be attributed to poor anchorage of wall vertical reinforcement as a result of the strengthening program, causing damage to concentrate around the third floor.

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