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VERIFICATIONS ON SEISMIC STRENGTHENING OF THE EXISTING RC BUILDING

Kazushi SHIMAZAKI [1]

ABSTRACT

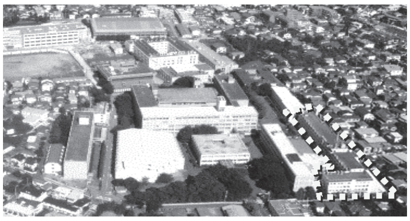
It was decided to demolish the three-story seismically strengthened “Building 4” at Kanagawa University, which was a five-story building when it was originally built in 1963. Presented with this opportunity, we planned two series of full-scale tests to evaluate the actual strength of this seismically strengthened building. One was an in-situ test of the frame, which had been seismically strengthened with a new steel brace, and the other involved member tests on two columns and a beam-column joint. During the in-situ test, the maximum horizontal force applied was 1.1 times the calculated horizontal strength based on the actual strength of the materials. In the member tests, one column experienced shear failure; the others suffered bending failure along the connecting beams. The beam-column joint specimen experienced shear failure on the beam. The obtained strengths can be estimated on the safe side by the common evaluation formulas.

Keywords: seismic strength, reinforced concrete building, steel brace, in-situ test, actual member test

INTRODUCTION

In Japan, much work has been done to evaluate seismic capacity, and many public buildings have been seismically strengthened. In the disaster investigation report (AIJ 2011) of the Tohoku-Chiho Taiheiyo-Oki Earthquake of 2011, it is reported that seismically strengthened buildings mostly only suffered small damage. Some reports (e.g. AIJ Tohoku 2013) indicate that the degree of damage is more closely related to the seismic index of the structure, such as its Is values (JBDPA 2001). However, only a few full-scale in-situ tests have been performed to directly identify the actual strength of strengthened structures. School buildings are intended for use as shelters when subjected to sever seismic movement, so it is necessary to evaluate the actual strength of such buildings, and whether they can ensure safety.

A seismic diagnosis on the then five-story “Building 4” at Kanagawa University, built in 1963 and shown in Photo 1(a), was conducted after the Great Hanshin-Awaji Earthquake of 1995. This revealed that seismic strengthening was difficult, so the building was reduced in scale by about a quarter by removing the eastern wing and two uppermost stories of the existing five-story structure. The remaining three-story section was seismically strengthened with a new steel brace as shown in Photo 1(b). The seismic capacity evaluation after strengthening indicated that its Is values were 0.7 or more, with q values of at least 1.1 on all floors. This implied sufficient earthquake resistance. As it has been decided to demolish this building, we planned to conduct two series of full-scale tests to evaluate the actual strength of this seismically strengthened building. One is an in-situ test, and the other involved member tests on two columns and a beam-column joint.


(a) In 1988

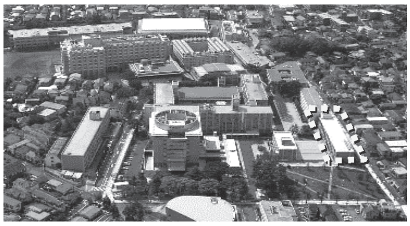

(b) In 2000

Photo 1. Building 4 at Kanagawa University

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[1] Prof., Dept. of Architecture, Kanagawa University, Yokohama, JAPAN, shimazaki@kanagawa-u.ac.jp

SUMMARY OF SEISMIC STRENGTHENING

The remaining three-story building was seismically strengthened with K-type steel-framed braces, reinforced concrete shear walls, with slits placed to improve the brittle column, as shown in Fig. 1. The seismic capacity evaluation (JBDPA, 2001) after strengthening indicated that the structure retained sufficient earthquake resistance.

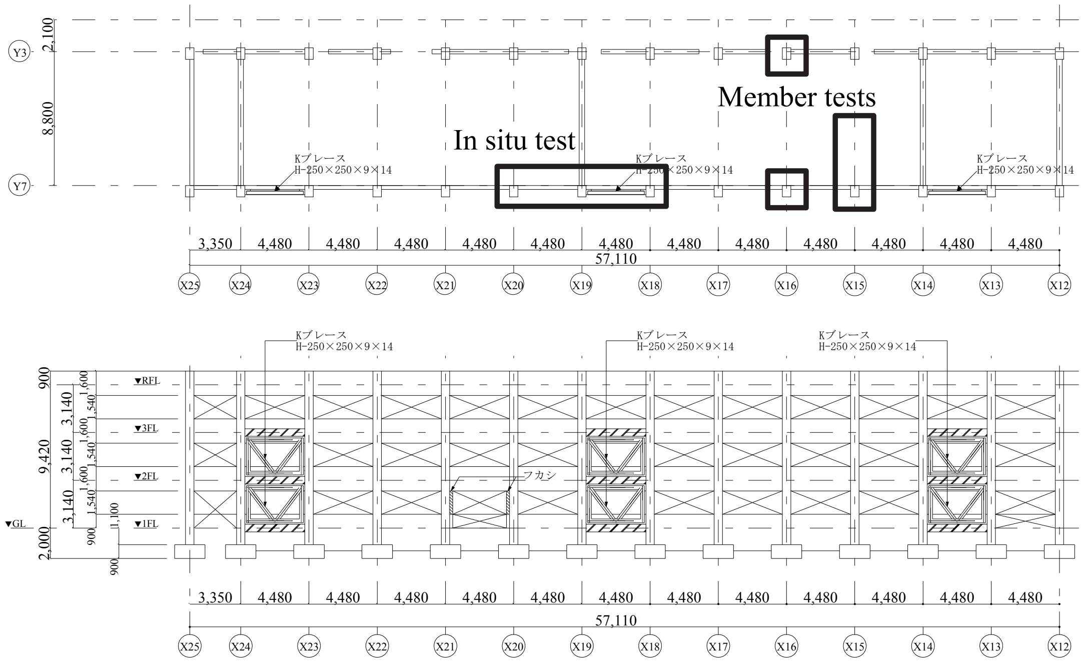


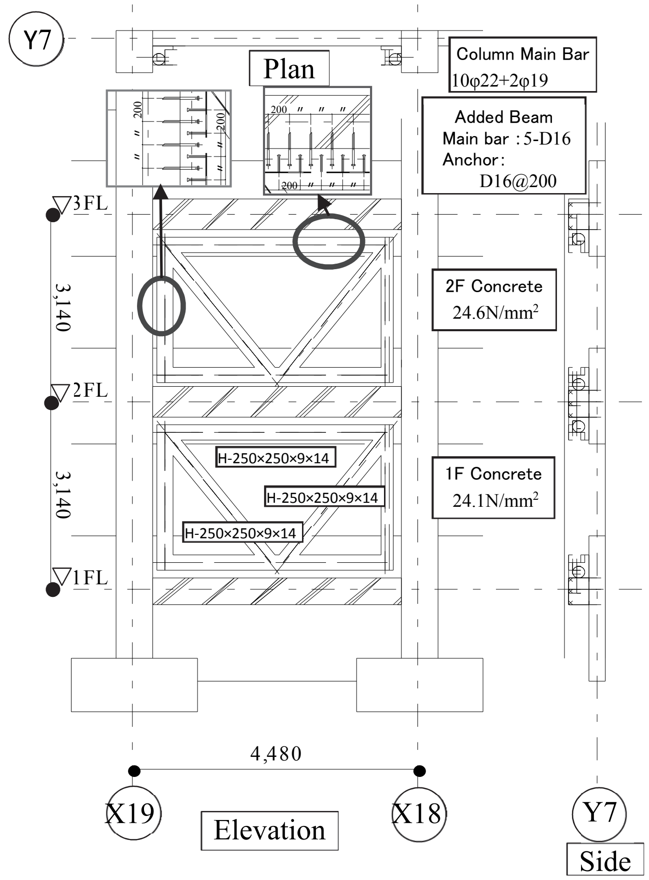
Figure 1. Outline of seismic strengthening (X Direction)

Figure 2 presents a brace mounting diagram. A K-type steel-framed brace was installed within the frame just outside the wall girder, with an additional beam attached to the outer face. The upright frame member of the brace frame extended to the rigid section of the existing RC columns and wall girders. A cross-section of the main member is represented in Fig. 2. The main reinforcement bars for the columns and girders were SR24 (SR235) round bars of 22 or 19φ, the shear reinforcements were SR24 (SR235) round bars of 9φ, and the second story concrete strength averaged 24.6 N/mm^2. The brace and frame members used had dimensions of H-250 × 250 × 9 × 14 (SS400).

IN SITU TEST

The test portion as shown in Fig. 3 is the second story with columns cut between the ceiling and the third floor, and wall girders cut on the third floor between the X17-19 and X20-21 lines. The horizontal strength of the brace expected from the strengthening design manual (JBDPA, 2001) was 2,420 kN. A horizontal force was applied as one-way repetitive loading using two hydraulic jacks. The displacements of the vertical and horizontal direction from the centroid line of each floor were also measured. Since torsional deformation of the frame is assumed, out-of-plane displacements of the beam on the third floor were also measured. The strain of the braces and frame members was also measured using strain gauges. The horizontal force was applied with a load control. The loading cycle is one-way loading of 500 kN once, 1,000 kN twice, 1,500 kN and 2,000 kN once each, 2,500 kN three times, 3,750 kN and 4,000 kN once each, until finally reaching maximum strength.

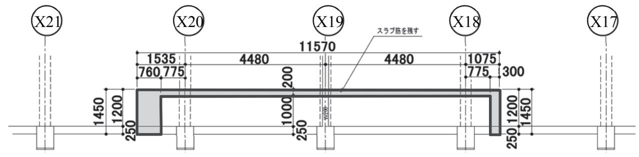
Figure 4(a) shows the relationship of the second story drift and the horizontal force, and Fig. 7(b) presents the out-of-plane deformation. Photo 2 shows the final destruction situation. The progression of damage was as follows. Bending cracks occurred in the column capital and base of the X20 column at 1,000 kN. At 2,500 kN, bending cracks occurred in the X18 column and the X19 ~ X18 beam. There was no loss of stiffness until the loading cycle reached 3,750 kN of horizontal force, and the story drift was less than 1/400 at that cycle. Stiffness did decline above 4,500 kN of horizontal force, with much greater deformation. Ultimately, the brace yielded and the RC columns suffered shear failure as shown in photos 2(a)-(c). The grout-filled section of the junction with the brace was damaged, but did not result in destruction of the anchor or stud as shown in photos 2(d)-(e).



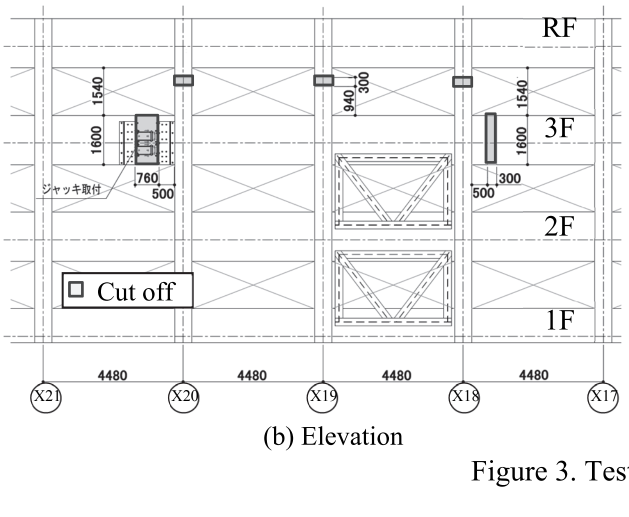
Member	Section	Rebar
Column (X18,X20)	X:550 Y:650	10-22φ+ 2-19φ 2-9φ@250
Column (X19)	X:550 Y:650	8-22φ+ 2-19φ 2-9φ@250
Brace	H-250×250×9×14	weak axis

Members

Figure 2. Outline of seismic strengthening with K-type steel braces

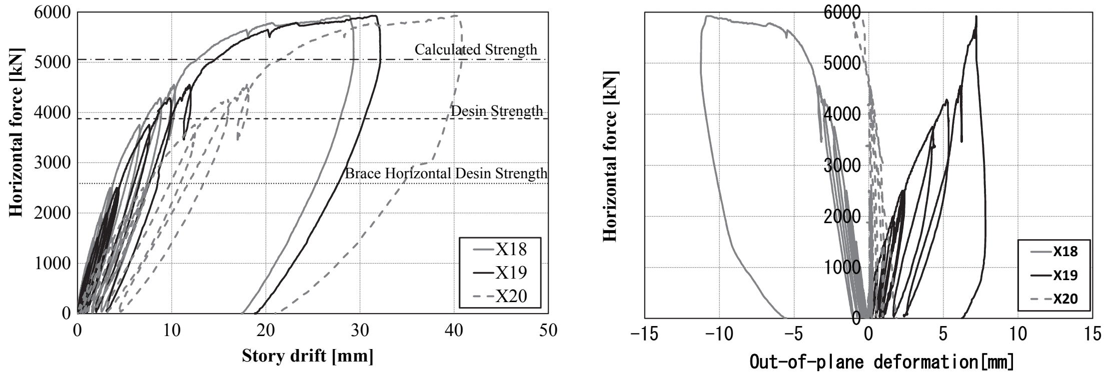


(a) 3FPlan



(b) Elevation

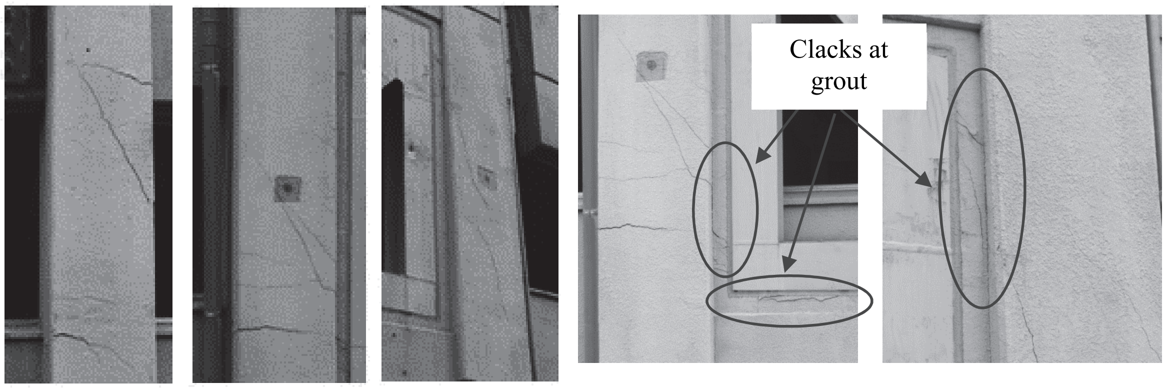
Figure 3. Test portion



(a) Relationship between the second story drift and the horizontal force

(b) Relationship between the out-of-plane deformation and the horizontal force

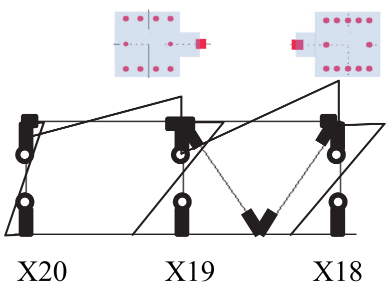
Figure 4. Test results



(a) X20 (b) X19 (c) X18 (d) X19 Brace side (e) X18 Brace side

Photo 2. View of the final damage

The maximum horizontal force applied was 5,920 kN. The strength calculated using the strengthening design manual (JBDPA, 2001) was 3,799 kN, and the strength based on the actual strength of the material and the solid cross-section of the member was 5,056 kN. This value is the sum of 3,377 kN for the horizontal component of the yield strength of the braces and the horizontal strength of the RC columns with the vertical steel frame shown in Fig. 5. Flexural strength M_u of the RC columns was calculated using the ACI stress block method (ACI 2011). Shear strength Q_u was calculated using Equation (1) (Arakawa 1960). The experimental value proved to be 1.9 times the design strength and 1.2 times the computed value.



Position		Axial Force kN	M_u kNm	Q_{Mu} kN	Q_u kN
X18	up	3211	1723	2010	783
	Dn		1373		
X19	up	-2835	1308	1090	498
	Dn		370		
X20	up	-202	350	398	475
	Dn		263		

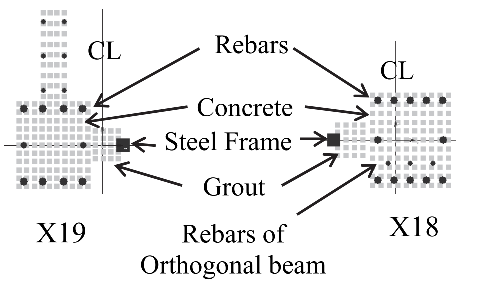
Figure 5. Analytical model and computed results

$$Q_{SU} = \left\{ \frac{0.068 p_t^{0.23}(18+Fc)}{M/(Qd)+0.12} + 0.85\sqrt{p_w \sigma_{wy}} + 0.1\sigma_0 \right\} bj \tag{1}$$

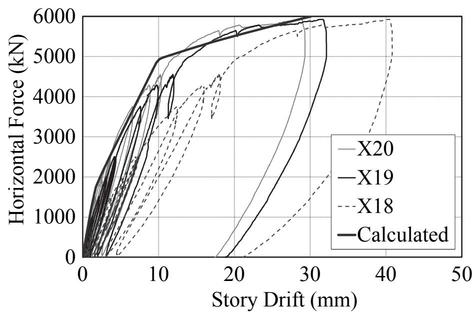
Where, p_t is the tensile reinforcement ratio (%), F_c is the compressive strength of concrete (N/mm^2), M/Q *is the* ratio of the bending moment to the shear force, d is the effective depth of the beam (mm), p_w is the shear reinforcement ratio (decimal number), σ_{wy} is the yield strength of the shear reinforcing bars (N/mm^2), b is the beam width (mm), and j is the distance between the centroids of the tension and compression portions.

Out-of-plane deformation on the X18 line is in the direction of the building exterior, and the opposite side on the X19 line. It is about 80% of the horizontal deformation at 2,500 kN, about 50% at 3,750 kN, and about 30% at maximum strength. It is considered that this torsional deformation is impeded because it is constrained by the orthogonal beam and slabs in the actual building; however, careful consideration of the confined stress is necessary.

In order to evaluate the stiffness and strength, an analysis using a three-dimensional frame elastic-plastic analysis program was carried out. The analytical model was a one-story plane frame model as shown in Fig. 5. The multi-spring model shown in Fig. 6(a) was used as the member model for the columns. The steel vertical frame member of the brace frame was considered to be part of the section of the X18 and 19 columns. The X19 column also has an orthogonal wall. The structural centroid position of the section was considered as the center of gravity for the steel, reinforcing bars and concrete in consideration of the ratio of Young's modulus for the X18 column. For the X19 column, which is a tensile column, it is the center of gravity of the steel and reinforcing bars ignoring the concrete. These structural centroid positions are shown in Fig. 5 as "CL". The top ends of X18 and 20 columns had the anchorage rebars of the orthogonal beams (3-25φ). Elastic stiffness was assumed to be an equivalent cross section. In order to assess the expected shear strength of the steel vertical frame, the X18 and 19 columns were assumed to have the shear strength with 1% of the elastic stiffness after any shear failure of the RC column. The thick solid line in Fig. 6(b) shows the analysis results. The results demonstrate good correspondence with the experimental results, except for the stiffness degradation at 4,000 kN.



(a) Analytical model



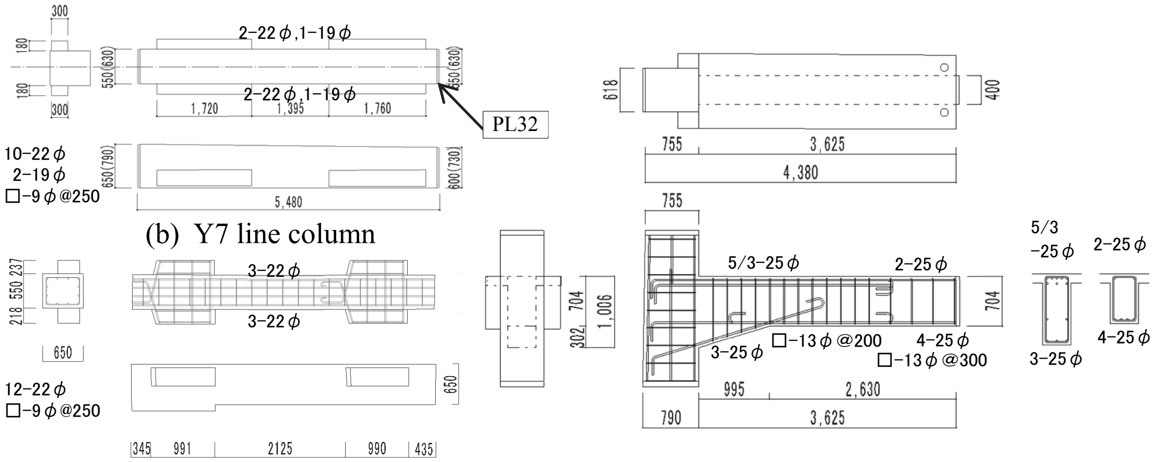
(b) Calculated result

Figure 6. Push-over analysis

MEMBER TESTS

Member experiments involve two column test specimens and one column-beam joint specimen. The column specimens are tree form members cut with the beam position of the lower end of the lower floor beam and the upper end of the upper floor from the X16 line shown in Fig. 1. The column-beam joint specimen is a ⊥ form member from the X15 line. The Y7 line column is a short column with an eccentrically clinging wall girder. The Y3 line column also has eccentrically clinging beams. Reinforcements of the cut surface of the column are welded to PL32 to ensure their fixing and filled with grout. Figure 7 shows the specimen's cross-sectional shape, and Fig. 8 shows the test setup. The column specimens were mounted horizontally in the frame as shown in Fig. 8(a). A constant axial load of 0.15 BDF_c (950 kN) was applied by a 1-MN hydraulic jack taking the reaction force by the PC steel bars. The shear force was loaded with a 750-kN actuator using the "Ohno loading system" to apply an anti-symmetric moment. Because of the beam's eccentricity, the loading point was the center of the beam. For column-beam joint specimens, the column was fixed to the frame as shown in Fig. 8(b), and the 750-kN actuator applied shear force to the beam like a cantilever beam.

The loading cycles are determined on the basis of the calculated shear capacity using the design strength. The cycles were ±1/3 and ±2/3 of the design strength applied twice, and then ± the full design strength three times, so as to force failure. Figure 9 shows the displacement measurement position. For the column specimens, story drift was measured at three points. The loading was controlled to keep the same value at these three displacement meters. For the column-beam joint specimen, deformation of the beam was measured as the relative deformation from the column.

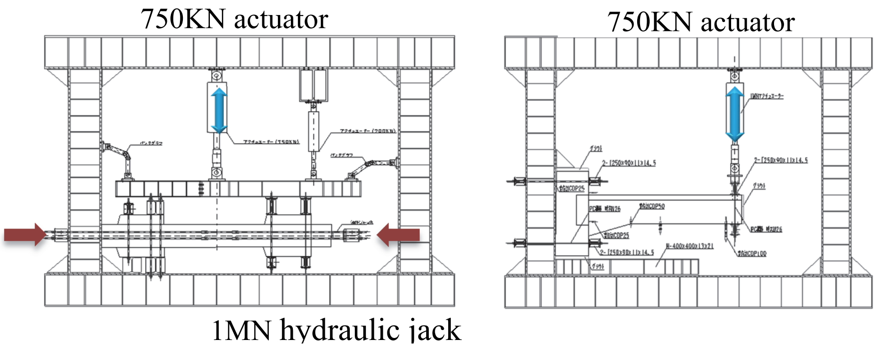


(b) Y7 line column

(a) Y3 line column

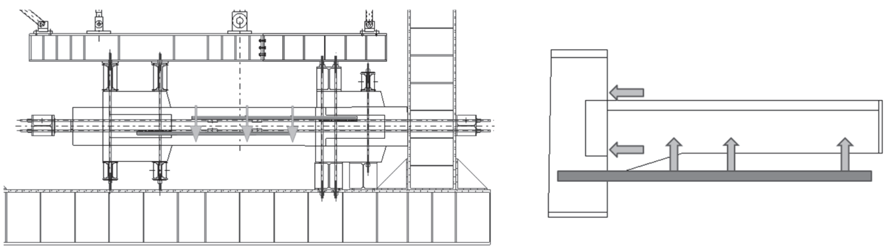
(c) Column-beam joint

Figure 7. Test specimens



(a) Column specimens (b) Column-beam joint

Figure 8. Test setup



(a) Column specimens (b) Column-beam joint

Figure 9. Measurement setup

Photo 3 shows the situation upon final destruction. Figure 10 shows the relationship between the shear force and deformation. For the Y7 line column, a bending crack was observed at 143 kN, and a shear crack (crack width: 0.05 mm) was induced at 366 kN, but without any reduction in stiffness. Finally, a shear fracture occurred at 698 kN. The shear crack entered to form a diagonal line along the column on the side of the eccentric beam, and it extended to the beam-to-column joint on the other side. The calculated shear strength according to Equation (1) using the actual strength of the material is 552 kN, which is shown in Fig. 10(a) as a one-dot dashed line. The experimental value was 1.26 times the calculated value. In the relationship between the shear force and deformation shown in Fig. 10(a), this column has three lines because the test specimen was not fixed to the frame completely, and the load control was not accurate. The anti-symmetric moment seemed to fluctuate, so the value of M/Qd was not constant during the test.

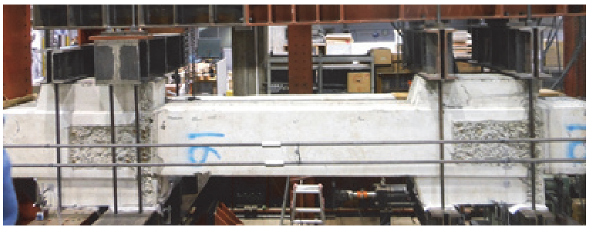
The other column of the Y3 line experienced bending failure of the connecting beam. At the cycle at one third of the design strength, flexural cracks are observed. At 244 kN, there were signs of concrete crushing in the compression position of the beam. Then, the load control was changed to deformation control. At R=1/100 (339 kN), the compression zone of the beam was crushed. The loading jig was inclined by the collapse of the beam because the beams are eccentric to the column as shown in Fig. 7(b), so loading was discontinued. The maximum load applied was 357 kN. This specimen was the weak-beam strong-column type. The calculated shear force value at the ultimate bending strength of the connecting beams is 174 kN. The dashed line in Fig. 10(a) is the shear force value with the crushing strength in the compression region of the connecting beams. The actual shear strength of the column is larger than these values.

For the beam-column joint specimen, at 250-kN shear force (R = 1/100), the beam suffered shear failure on the loading side of the beam. The weld of the fixing plate for the main reinforcement at the cut surface gave way. The main reinforcement of the beam is largely at the column end, and it is cut off in the center part as shown in Fig. 7(c). The shear forces at the ultimate bending strength of the cut-off position and at the column end were 253 kN and 310 kN respectively. The value at the cut-off position is smaller than the value at the column end. Shear reinforcement of the central portion is small and the shear capacity based on Equation (1) is 236 kN, but is 497 kN at the column end portion. As shown in Fig. 10(b), stiffness declines with about 25-mm displacement (250 kN of shear force). This seems to be the ultimate bending strength at the cut-off position. The analysis results using a simple fiber model with variable cross-sectional beams show the main reinforcement yields at the cut-off position. The calculated results shown in Fig. 10(b) as a dark solid line afford good correspondence with the experimental results. For the actual building, the vertical loads are distributed, and the shear force is small in the central portion. This member test was planned to simulate the moment gradient during an earthquake considering a vertical load effect. As the load was applied with a concentrated load, the shear force was constant over the entire length. As a building member, the beam would not experience shear failure.

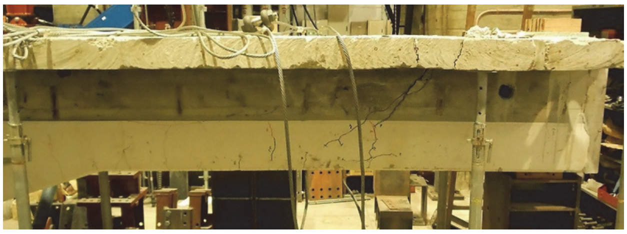
CONCLUSIONS

It was decided to demolish the three-story seismically strengthened “Building 4” at Kanagawa University. We planned two series of full-scale tests to determine the actual strength of this seismically strengthened building. One was an in-situ test of the frame which had been seismically strengthened with a new steel brace, and the other involved member tests of two columns and a beam-column joint. In the in-situ test, the experimental value reached 1.9 times the design strength, and 1.2 times the computed value based on the actual strength of the materials. In member tests, one column experienced shear failure at 698 kN, the others did not fail until 357 kN, but the connecting beams failed due to crushing in the compression region. The beam-column joint specimen suffered shear failure of the beam at a shear force of 250 kN. These observed strengths can be estimated on the safe side by common evaluation formulas.

(a) Y7 line column

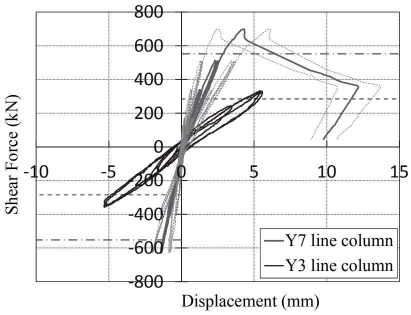


(b) Y3 line column

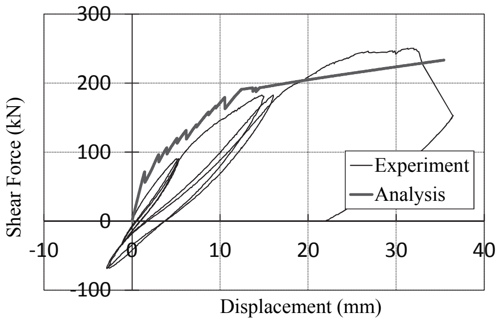


(c) Column-beam joint

Photo 3. Final destruction situation



(a) Column specimens



(b) Column-beam joint

Figure 10. Test results

ACKNOWLEDGMENTS

Kajima Corporation performed the actual design and construction for seismic strengthening of the building. They also drafted the safety plan for the in-situ-test and extraction of the specimens. This study was funded by Kanagawa University. The opinions and findings do not necessarily represent those of the sponsor.

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