SEISMIC RETROFIT OF DUCTILE CONCRETE MOMENT-RESISTING FRAMES WITH INNOVATIVE PIN-SUPPORTED WALL SYSTEM

Hiroyasu Sakata, Zhe Qu, Saburoh Midorikawa, Akira Wada
Tokyo Institute of Technology
Yokohama, Japan

Abstract

A thirty-two years old eleven-story steel reinforced concrete frame building was seismically strengthened in 2010 by attaching six pin-supported post-tensioned concrete walls to the existing structure along the perimeter. Dozens of shear-type steel dampers were installed along both sides of each wall. The basic design concept is (1) to create uniform story drift distribution along the height of the building by the walls so that soft-story failure may not take place and (2) to dissipate energy by the dampers so that the global response of the building and the damage to the concrete frame is reduced. After one year of being retrofitted, the building experienced moderate ground motion during the M9.0 Tohoku earthquake in 2011. As was evident from the recorded motions and observed damage of the building, the pin-supported walls worked as expected and sustained no damage.

Introduction

Weak story failure of moment-resisting frames has long been a problem. Many historic major earthquakes, e.g., the 1985 Mexico City earthquake and the 1995 Hanshin-Awaji earthquake, have seen the failure of many reinforced concrete frames in this kind of local mechanism (Villaverde, 1991; AIJ, 1997). This has continued to be a problem. Weak story collapses of ductile reinforced concrete frames were observed in the recent M9.0 Tohoku earthquake (AIJ, 2011). It is not easy to avoid such unfavorable failure mechanism mainly because of lack of adequate global component along the height of the structure. This could also be considered as lack of integrity in the sense that individual stories or substructures in a system do not collaborate well with each other.

Walls, usually with much deeper cross sections, are much more effective than individual columns for providing flexural stiffness to control story drift. Paulay and Priestley (1992), pointed out that development of a weak story mechanism can easily be avoided as a result of the considerable stiffness of walls. In examining the effect of foundation flexibility on the seismic performance of a shear wall-frame dual system, they demonstrated the drift control effect and force distribution characteristics of dual systems with pin-supported walls. The results of their nonlinear dynamic analysis indicate that loss of wall base restraint would not significantly impair the seismic performance of wall-frame dual systems. In a wall-frame dual system, they work as coordinators that transfer redundant strength somewhere in the system to where the strength is inadequate.

Figure 1 demonstrates a simplified dual system with a pin-supported wall and two single-story subassemblies, which are connected to the wall by horizontal links at different levels. In this model, the wall provides no additional lateral resistance. Assume that the strength of the i^{th} story alone is not adequate to resist the earthquake action $F_{EQ,i}$ on it and is expected to go deep into the plastic regime and sustain large deformation. With the help of the pin-supported wall, the lateral drift of this story will be forced to be similar to those of other stories by an additional resisting force of F_w provided by the wall. At the same time, the j^{th} story, where the strength is redundant, must resist an additional force F_w, which is imposed by the wall. Thus, the deformation of this stronger story is increased. In other words, weaker stories use strength from stronger ones so that they will not fail prematurely. At the same time, the pin-supported wall must carry the resultant shear force and moment so that this mechanism can be retained even during strong earthquakes.

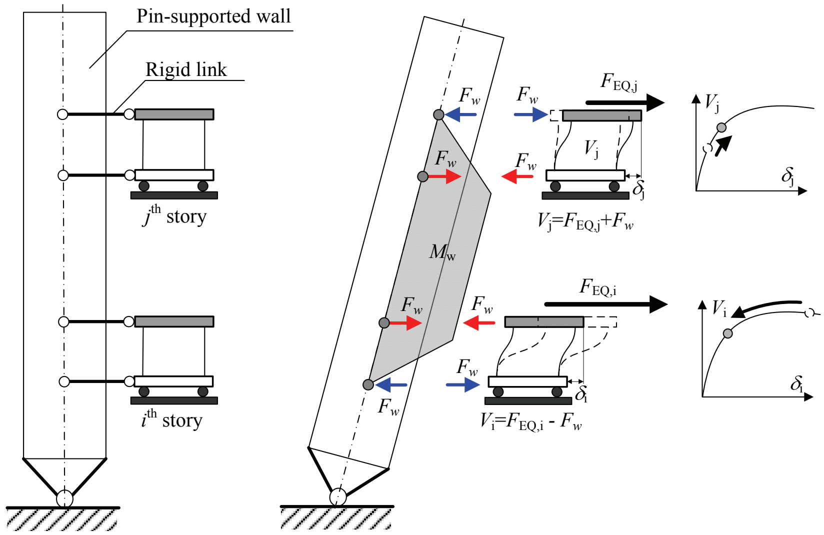


Figure 1. Mechanism of deformation pattern controlled by pin-supported walls.

Energy dissipation is another important issue for mitigating seismic responses and protecting buildings from seismic damage. Innovative solutions for introducing energy dissipation capacity for walls have been explored in the past few years. Readers are referred to Ajrab et al. (2004), Restrepo and Rahman (2007), Marriot et al. (2008) among others for more information.

In the present paper, post-tensioned pin-supported walls are used to strengthen an eleven-story steel reinforced concrete (SRC) frame. Taking advantage of the plan layout of the existing structure, a distributed energy dissipating solution is proposed. Basic design concerns and the final details of the pin-supported wall-frame systems are introduced below.

Tokyo Tech G3 Building

The G3 Building on the Suzukakedai campus of Tokyo Institute of Technology in Japan is an eleven-story SRC frame building that was completed in 1979, during which, in 1978, the M7.4 Miyagiken-oki earthquake hit Japan and led to a major revision of the seismic provisions of Japan in 1981(Aoyama, 1981). As concluded by a recent seismic inspection, there was an urgent need to strengthen the G3 Building (Wada et al, 2009).

The structural layout of the original frame is feathered with six slots distributed along the perimeter (Figure 2). This makes the attachment of pin-supported walls much easier. Six post-tensioned concrete walls with bottom hinges are installed in those slots and connected to the existing frame at each floor level by horizontal trusses behind them. Shear-type steel dampers are installed in the gaps between the pin-supported walls and adjacent SRC columns as well as between the walls and added transverse shear walls at both ends. The main components of the retrofitting plan, as shown in Figure 2, are visible from outside the building so passersby can appreciate the engineering solution.

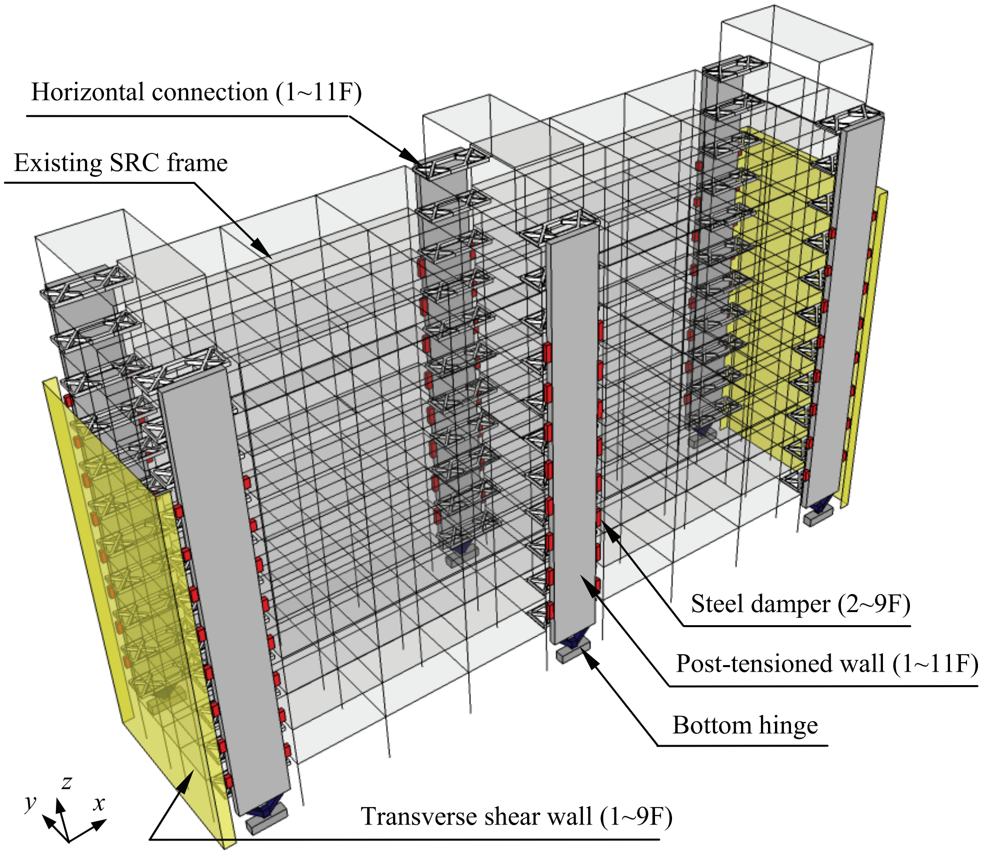


Figure 2. Retrofit plan of G3 Building.

Performance Evaluation. Nonlinear dynamic analysis with one code-stipulated and two site-specific ground motions was carried out to assist the determination of major parameters of the retrofitting parts. More extensive analysis was then carried out to better evaluate the seismic performance of the G3 Building before and after the retrofit. Detailed nonlinear model was built in ABAQUS 6.8 and 22 ground motions (FEMA P605, 2009) which were scaled to have PGV = 50 cm/s were used for the evaluation. The readers are referred to Qu et al. (2012) for more details of the analysis model. The displacement responses of the building before and after the retrofit are compared in Figure 3 in terms of maximum inter-story drift ratio (IDR). The median and 84th percentile responses are also plotted. The original structure experiences extremely large IDRs in certain stories under many of the ground motions, indicating collapse at these stories (Figure 3(a)). In contrast, the distributions of IDRs over the height of the retrofitted building under various ground motion records are much more uniform (Figure 3(b)).

Post-tensioned Concrete Walls. The numerical analysis results were used in turn to determine the required capacity of the pin-supported walls. The moment and shear demand for the pin-supported walls are shown in Figure 4. There is large moment demand in the middle part of the wall while the shear demand is large at the base as well as in some middle stories. The final design of the wall section is shown in Figure 5. All six pin-supported walls have identical cross sections of 4300 mm in width and 600 mm in depth. The total cross-sectional area of the pin-supported walls at each story is about 50% to 61% of that of the original SRC columns from the lower to upper stories. Concrete with 36 MPa nominal compressive strength is used. Each wall is prestressed by six units of post-tensioning tendons. Each unit consists of 30 strands each 12.7 mm in diameter. Initial pre-stressing force of no less than 18000 kN per each wall was introduced primarily to prevent cracking and consequent stiffness degradation of the walls under strong earthquakes. By the pre-tensioning, the cracking moment of each wall is increased by about 70% from

$M_{cr,RC} \approx 13000$ kNm to $M_{cr,PC} \approx 22000$ kNm. As can be seen in Figure 4(a), the increased cracking moment, $M_{cr,PC}$, is even greater than the largest moment demand obtained in the analysis.

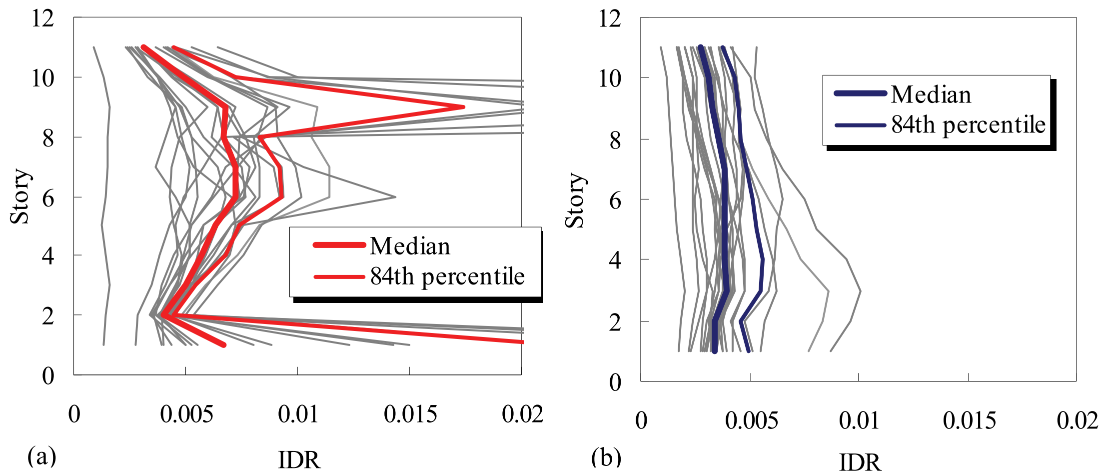


Figure 3. Inter-story drift ratios (IDR) of G3 Building before and after being retrofitted: (a) existing (original) structure and (b) retrofitted structure.

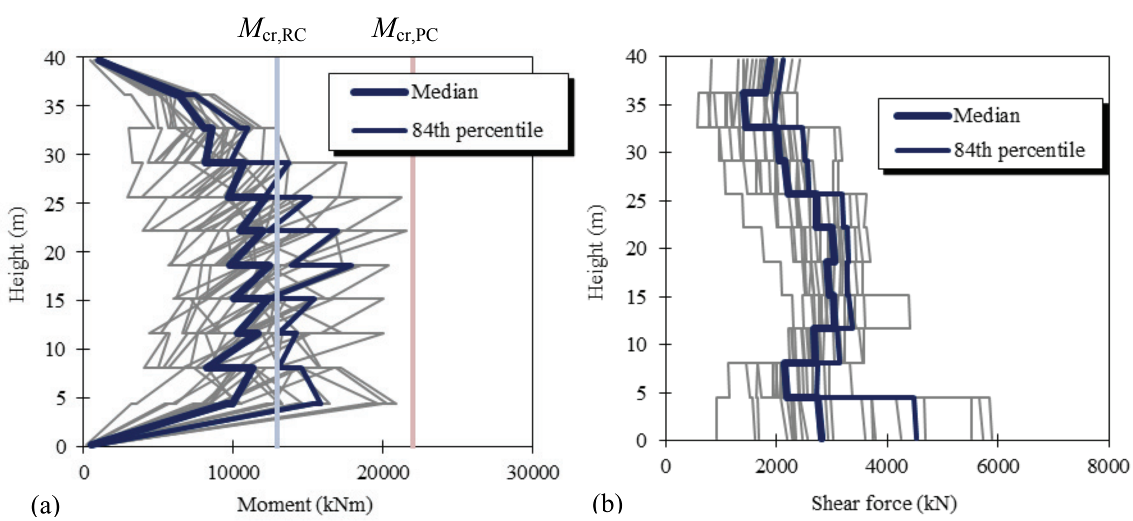


Figure 4. Strength demand for pin-supported walls: (a) Moment and (b) Shear force.

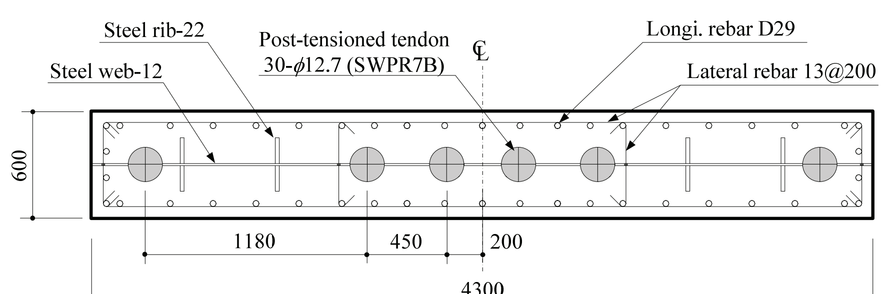


Figure 5. Cross section of post-tensioned wall.

Connections for Pin-supported Walls. Pin-supported walls are connected to the foundation and the existing frame. The bottom hinge is made of two pairs of steel braces contacting each other through a tooth-like bearing. Details and a photo of one of the completed bottom hinges are shown in Figure 6.

The tooth-like bearing is made of cast iron NCH490 with nominal yield strength of no less than 325 MPa. It consists of two separated tooth-shaped pieces (the lower and upper pieces), which interlock with several teeth and a separated stopper in the middle to prevent slip in the out-of-plane direction (Figure 7). The teeth in the lower piece are 20 mm longer than are those in the upper one to create a small gap, and their tips are chamfered to allow rotation of the upper piece. The shear resistance of a single pair of teeth (i.e., at the critical shear surface in Figure 7) is confirmed to be greater than maximum shear demand and is about 2.7 times the average shear demand as shown in Figure 4(b).

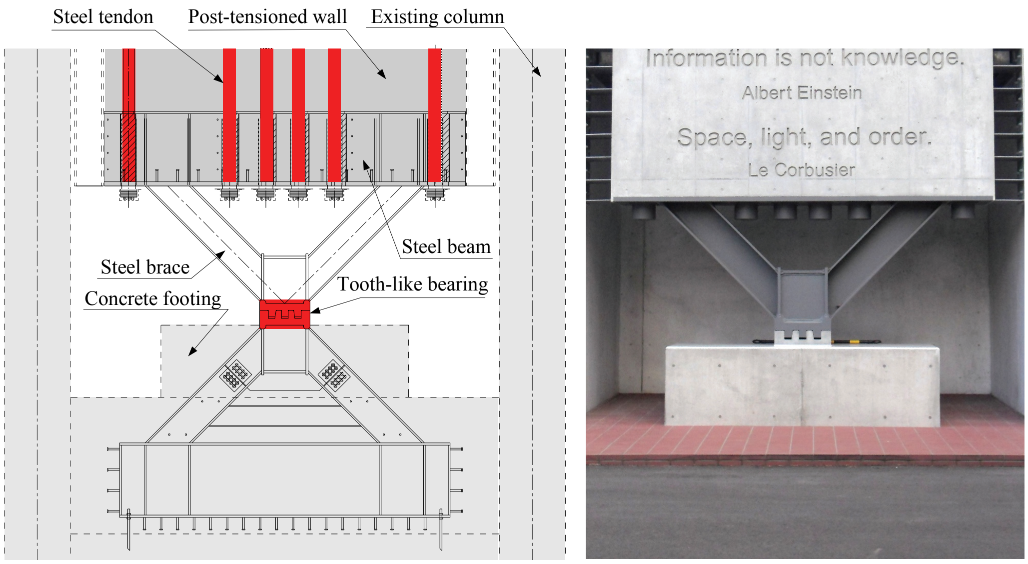


Figure 6. Bottom hinge.

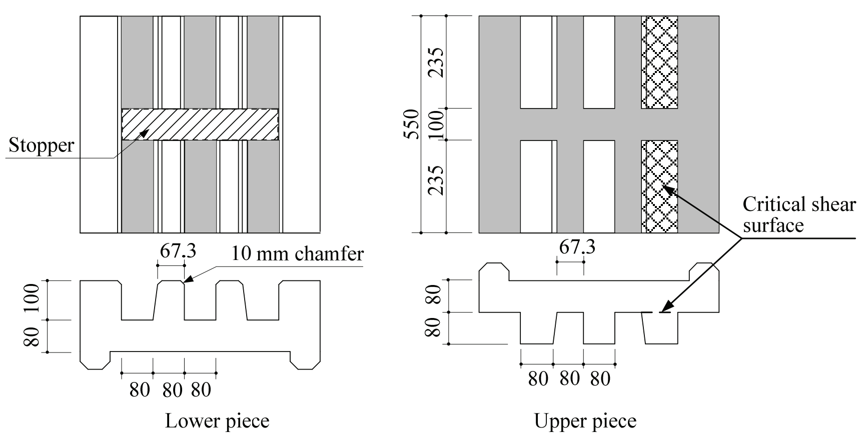


Figure 7. Details of pin bearing.

Pin-supported walls are connected to the existing frame mainly by horizontal connectors at each floor level behind them. Each connector consists of a horizontal truss and a steel shear key (Figure 8). The horizontal truss is firmly anchored to the original SRC beams. Steel shear keys are used to connect the horizontal trusses and pin-supported walls, which are expected to impose very small rotational constraints for pin-supported walls. The principal functions of the horizontal connectors are to transmit lateral forces between the pin-supported walls and the SRC frame and to provide out-of-plane support for the walls.

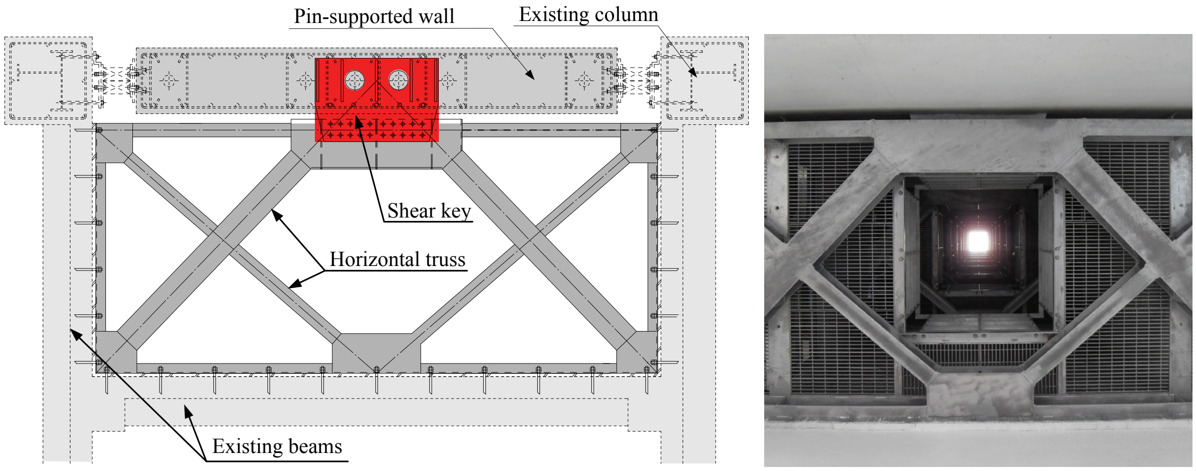
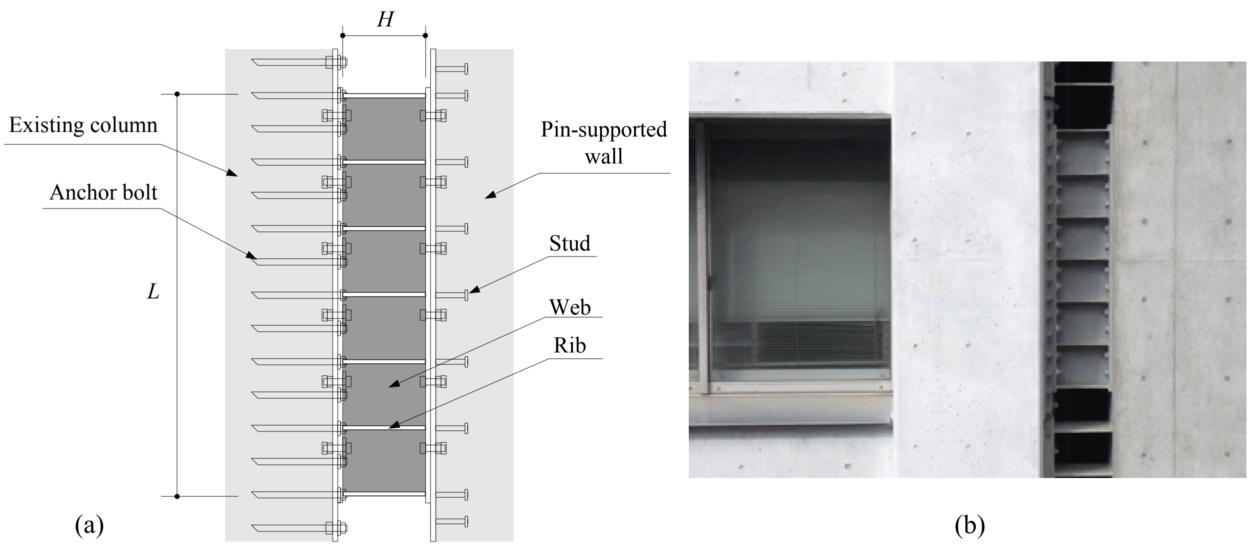


Figure 8. Horizontal connector.

Shear-type Steel Dampers. Shear-type steel dampers are aligned on both sides of each pin-supported wall. Low yield steel with 130 MPa nominal shear strength is used. Steel webs 6 mm thick, which are stiffened by ribs at 250 mm spacing, function as energy dissipaters. The web height H is 312 mm for all the dampers while the length L varies from 750 mm to 1500 mm (Figure 9(a)). Table 1 lists the web length L of steel dampers in different locations. Figure 9(b) is a photo of an installed steel damper with L = 1500 mm. Cyclic loading test of the steel dampers reveals that the nominal shear capacity of the damper can be satisfactorily retained at up to 9% shear strain, which is about 58 times the yield shear strain of the damper. Most earthquake input energy is expected to be dissipated by these steel dampers.

To avoid uplift of the pin-supported walls due to damper forces, which might lead to dislocation of these walls from the pin bearings, the steel dampers are always arranged on both sides of the walls, even for walls at both ends of the building, so that the damper forces on both sides of a wall form a force pair and the resultant vertical force on the wall becomes insignificant. Besides, vertical ground motion-induced forces on a pin-supported wall can be resisted by the dampers, the total yield strength of which is more than five times the self-weight of the pin-supported walls at the center and about three times that of the walls at both sides.



(a) (b)

Figure 9. Completed steel damper with L = 1500 mm.

Table 1. Web length *L* of steel dampers (unit: mm)

Story	Central wall	Side wall
8 and 9	1500	750
2~7	1500	1000

Seismic Performance during 2011 Tohoku Earthquake

Five accelerographs, each with three channels for the three translational degrees of freedom, were installed in the basement (B1F), on the 2nd, 5th, 8th and 11th floor of the G3 Building when the retrofit was completed. Taking the records at the basement level as the ground motions, the peak ground acceleration (PGA) and the peak ground velocity (PGV) is 67.3 cm/s^2 and 14.4 cm/s in the longitudinal (x) direction and is 52.8 cm/s^2 and 12.7 cm/s in the transverse (y) direction, respectively (Miura et al., 2011). Figure 10 compares the acceleration response spectra of the recorded ground motions (B1F records) with the Level I design spectrum prescribed in Building Standard Law of Japan. In both directions, the spectra of the records were much lower than the design spectrum in period range below 1.0 s. In addition, the structure seems to have experienced quite similar shaking in both directions.

The accelerograms are processed by 4th order Butterworth high-pass filter before they are integrated to get the displacement response. Filtered accelerograms are integrated twice to obtain the displacement response. Numerical integration with trapezoidal rule is used and time increment Δt is set to be 0.01 s. The maximum lateral displacement in both x- and y-direction is depicted in Figure 11. The building exhibited different story drift distribution along the height in orthogonal directions. This was mainly a consequence of story stiffness distribution because the building remained essentially elastic during the earthquake.

The building was inspected for seismic damage immediately after the Tohoku earthquake. Many minor cracks of width less than 0.2 mm were observed near the bottom of the conventional fixed-base walls at both ends of the building in the transverse direction, while on the other hand, no visible damage was observed on the pin-supported walls in the longitudinal (x) direction except that the painting of the bottom bearing was slightly peeled off, indicating that the bearing may have rotated as expected during the earthquake (Figure 12). In addition, diagonal rust was observed on the steel web plates of several steel dampers at lower stories (Figure 13), which indicated that these web plates may have yielded during the earthquake and the protective painting along the directions of principle strain was peeled off.

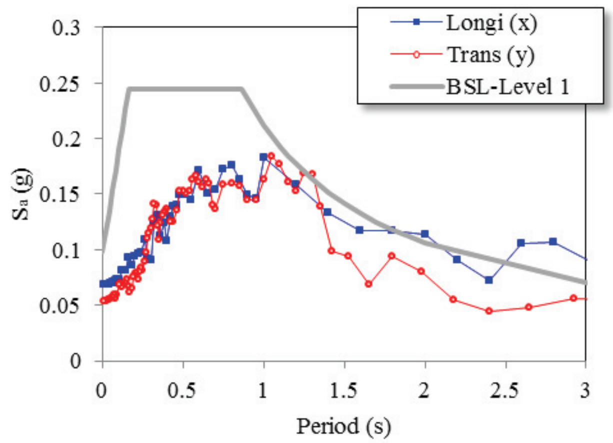


Figure 10. Acceleration spectra of recorded ground motion in longitudinal (x) and transverse (y) directions (5% damping)

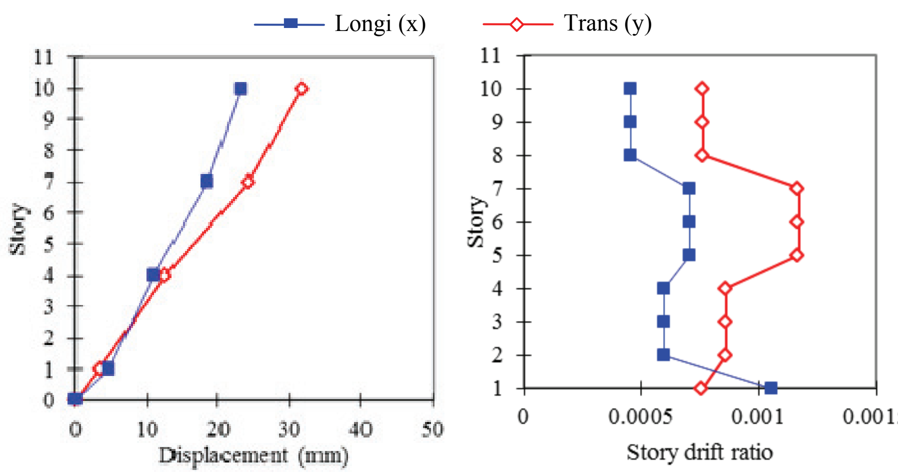


Figure 11. Recorded peak motions of G3 Building: (a) lateral displacement and (b) story drift ratio.

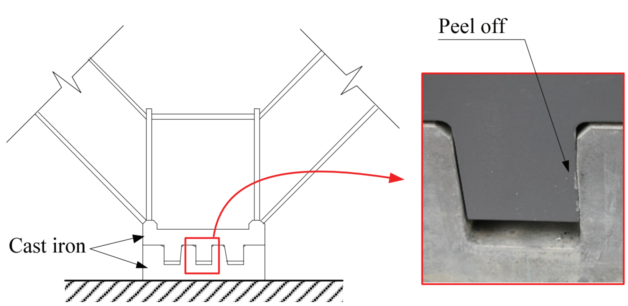


Figure 12. Peel off of painting of bottom tooth-like hinge (north, center).

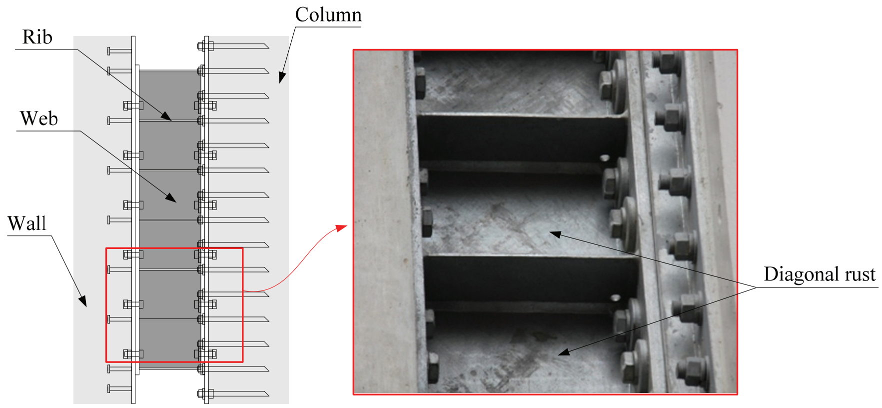


Figure 13. Diagonal rust marks on web plates (north, center at 2F).

Summary

Innovative pin-supported walls were applied to the retrofit of an eleven-story ductile moment-resisting frame on the campus of Tokyo Institute of Technology. The carefully design cast iron hinge at the bottom allows the wall to exhibit finite rotation around its base without damage so that the wall can better accommodate the lateral drift of the building and prevent soft story mechanism to take place. Accompanied with the steel dampers along both sides, the walls can also increase the lateral-force resisting capacity, and more importantly, the energy dissipation capacity of the building.

The retrofitted building suffered moderate shaking during the 2011 Tohoku earthquake. In contrast to the many visible cracks on the conventional shear walls in the transvers direction, the pin-supported walls in the longitudinal direction sustained no visible damage. The peeling of protective paint on the cast iron hinges and on some steel dampers suggests that the system worked as expected under shaking.

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