

## Learning from Earthquakes

# Performance of Engineered Structures in the $M_w$ 9.0 Tohoku, Japan, Earthquake of March 11, 2011

The EERI reconnaissance team deployed to evaluate damage to engineered structures, with John Wallace of UCLA as team leader, was split into three groups: reinforced concrete buildings, structural steel buildings, and buildings with protective systems. This report discusses the building types in that order. Reinforced Concrete Team members included Mark Aschheim, Santa Clara University (group leader); John Wallace, UCLA; Toshimi Kabeyasawa, University of Tokyo; Masaki Maeda, Tohoku University; and Hitoshi Shiohara, University of Tokyo. The Steel Structures Group was comprised James M. Ricles (group leader), Lehigh University; Dimitrios G. Lignos, McGill University; R. Jay Love, Degenkolb Engineers; Mitsumasa Midorikawa, Hokkaido University, and Taichiro Okazaki, Hokkaido University. The Protective Systems Team included Geoff Bomba, Forell/Elsesser Engineers (group leader); Andrew Taylor, KPFF Consulting Engineers; Kazuhiko Kasai, Tokyo Institute of Technology; Kazuhiro Matsuda, Tokyo Institute of Technology; Troy Morgan, Tokyo Institute of Technology; and Wuchuan Pu, Tokyo Institute of Technology.

Unless otherwise indicated, photos in this report were taken by team members.

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## Introduction

Damage from the  $M_w$  9.0 earthquake resulted from both strong ground shaking and the huge tsunami that followed. The earthquake struck at 2:46 pm local time and ruptured a 300 km-long off-shore fault extending from near the southern end of Ibaraki Prefecture to central Iwate Prefecture. It was the largest-magnitude earthquake recorded in Japan in historic times, and the combined impacts of the earthquake and tsunami left at least 15,703 people dead, 4,647 missing, 5,314 injured, 130,927 displaced, and at least 332,395 buildings, 2,126 roads, 56 bridges, and 26 railways destroyed or damaged from Chiba to Aomori. Casualties and damages were focused in Iwate, Miyagi and Fukushima prefectures, and the total economic loss in Japan is estimated at US\$ 309 billion (USGS, 2012).

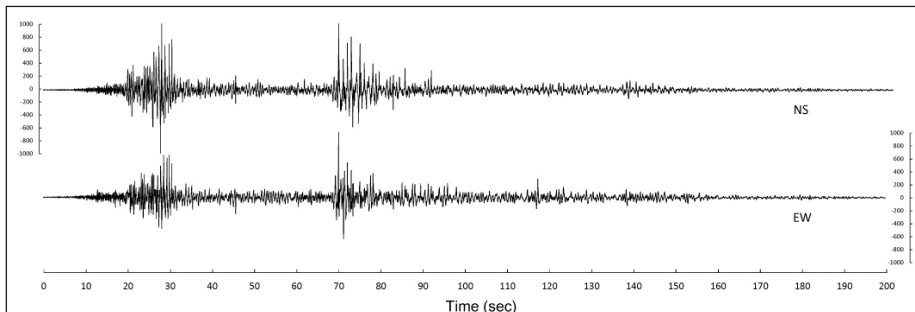
Figure 1 is the accelerogram from the March 11th earthquake recorded at station K-NET MYG013 in Sendai,

129 km from the epicenter. The peak ground accelerations in the NS and EW directions were 1.55g and 1.01g, respectively, and strong ground shaking lasted for about two minutes. The response spectra for this record are given in Figure 2, where the predominant period at which the accelerations are largest is near 0.8 seconds.

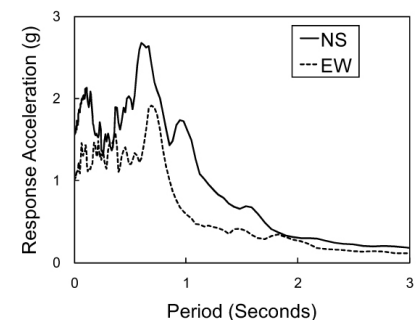
## Part 1. Reinforced Concrete Buildings

This section focuses on the Fukushima and Sendai areas, primarily school buildings and residential/office buildings, with additional information on buildings at the Aobayama Campus of Tohoku University in Sendai.

Damage to RC buildings was relatively light in the most affected prefectures, with only a small fraction of them heavily damaged or collapsed. In general, damage was concentrated in pre-1970 buildings, buildings of less than ten stories, and in unusual buildings (long-span, flexible diaphragm). In some cases, damage was focused in pockets, presumably because of soft soil conditions and higher shaking intensities. Damage to columns was similar to that observed in previous earthquakes, often associated with



**Figure 1.** Ground motions recorded at station K-Net MYG013 in Sendai (data: Okazaki)



**Figure 2.** Response spectra (data: Okazaki)



**Figure 3.** Elementary school in Sukagaya. Column damage at classrooms depended on presence or absence of cast-in-place infill at the base. Infill creating captive column with integrated wing walls caused shear failures.

short or captive columns developing shear failure. The presence of reinforced concrete wing walls on one or both sides of a column often induced diagonal cracking that penetrated through the column. Other column failures were associated with discontinued shear walls above. Significant damage, typically diagonal cracking, was observed in reinforced concrete partition walls that were used as nonstructural walls. Mid-rise multistory buildings did not show evidence of damage due to higher modes of vibration, although damage was concentrated at vertical structural discontinuities. Retrofits with infilled steel bracing or shear walls proved to be effective, while those with viscous dampers presumably reduced deformation demand but not sufficiently to preclude damage to stiff concrete elements.

**Japanese Code Provisions for RC Buildings.** From 1950-1971, column cross-section dimensions  $D$  were required to exceed  $H/15$ , where  $H$  is the column clear height, and column transverse reinforcement consisted of single perimeter



**Figure 4.** Junior high school built in 1966, located 10 km N. of Kuriyama. Wing walls along the exterior, nominally reinforced and cast eccentrically with columns, induced shear failures that penetrated through the columns.

hoops with spacing not to exceed 300 mm or 15db (Otani, 2000). Following the 1968 Tokachi-oki earthquake, new provisions were introduced in 1971 to prevent column shear failures and improve ductility. A capacity design approach was taken for column shear, and more stringent requirements were laid down for transverse reinforcement. At the top and bottom of columns, over a height of  $2D$ , spacing of hoops was limited to 100 mm, whereas within the middle portion of the column, hoop spacing was limited to 150 mm. Additional changes included requiring  $135^\circ$  (or more) bends on hoops and the consideration of reinforced concrete nonstructural walls in the design process.

The 1978 Miyagi-ken oki earthquake was a major factor in the code changes introduced in 1981. These changes included a minimum shear reinforcement ratio, consideration of torsional eccentricity in design, and limits on story drift.

**Unretrofitted Buildings.** The team visited a large number of older school buildings, typically two to three stories tall, with classrooms located on one side of a hallway. Beam and column framing is sometimes supplemented with shear walls, often cast integrally with the columns. Windows are supported on cast-in-place infill, and are usually taller on the classroom side, which results in the exterior classroom columns being more flexible than the exterior hallway columns.

The reinforced infill panels restrained column deformations, creating a “captive column” and typically inducing column shear failures. Columns with longer clear heights (free of infill restraint) fared better (Figure 3). Columns restrained by infill supporting windows developed shear cracks, while those extending to the floor were undamaged. The most severe damage was seen in columns with reinforced wing walls that were cast integrally with the columns (Figure 4). In addition to reducing column clear height, the infill wing walls increased the effective width of the column, thereby affecting the aspect ratio.

Columns located beneath discontinued shear walls were prone to severe damage. Figure 5 shows an interior column in the hallway on the lowest floor of a three-story school. The adjacent exterior hallway column, not located below a discontinued shear wall, had shear failure but was not crushed as severely. In contrast, columns that were integrated into shear walls were often protected from failure, at least in the low-rise construction we observed.





**Figure 5.** Junior high school located 10 km north of Koriyama. Interior column below end of discontinued shear wall above.

A typical apartment building type in the region consists of strong-column/weak-beam moment-frame construction with cast-in-place exterior “nonstructural” walls between floor levels at beam midspan at nearly all stories. Height-to-length ratios for these walls were typically between one and two, so they contributed significantly to lateral strength and stiffness. As relative-

ly stiff elements, vulnerable to shear failure, they functioned as interstory drift gauges. Damage to these panels typically involved heavy diagonal cracking; however, the damage in mid-rise buildings (11-15 stories) generally decreased with height, suggesting that first mode contributions to interstory drift were dominant. While most of the shear panels had diagonal cracking characteristic of shear failure, some exhibited sliding failures or bond/anchorage failure (Figure 6).



**Figure 7.** Elementary school approximately 1 km east of Koriyama. Stiff retrofits, such as diagonal steel bracing and infill shear walls, were effective at preventing structural damage.



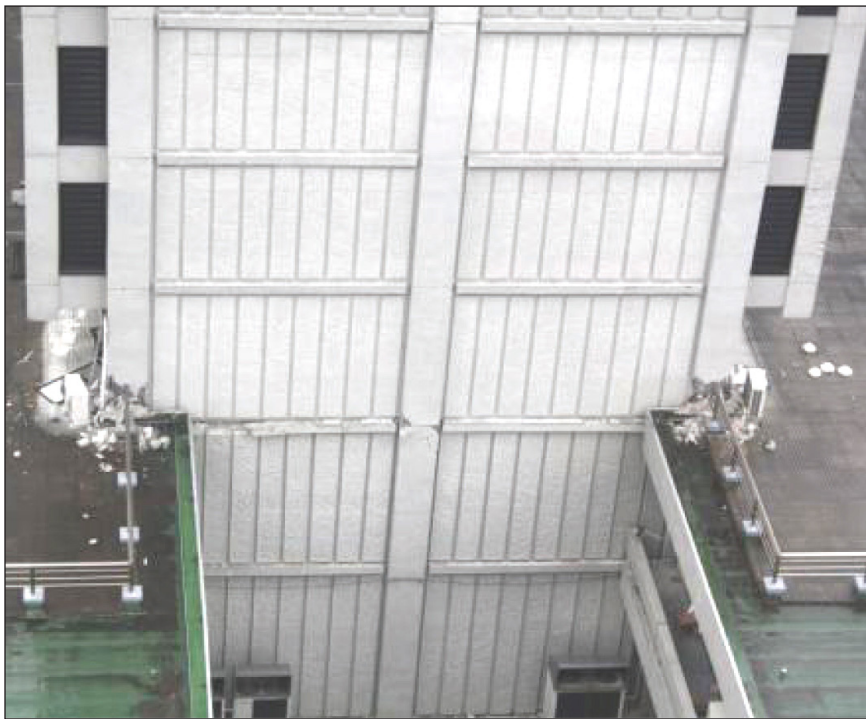
**Figure 6.** Failure modes for shear panel include diagonal cracking, bond/anchorage slip of vertical reinforcement, and sliding at the top of the panel.

new building had failures in reinforced concrete parapets, steel diagonal roof bracing, suspended ceilings, and mechanical equipment. Amplified response was attributed to soft soils associated with old stream-bed deposits.

Liquefaction-induced bearing failure was believed to be the cause of severe settlement and differential movement in adjacent school buildings located approximately 2 km southeast of Osaki.

**Repaired and Retrofitted Buildings.** Many school buildings and housing complexes had been evaluated and retrofitted prior to the earthquake, while others were repaired following previous quakes. In several retrofitted buildings, the “stiff” retrofits (diagonal bracing and/or infill shear walls) were successful in limiting damage to minor cracking at a few locations. In one case, supplemental dampers used to retrofit a ten-story housing complex near Sendai City likely limited interstory drift, but there was still enough to cause diagonal cracks in nonstructural cast-in-place walls. Figure 7 shows a three-story elementary school building near





**Figure 8.** Civil Engineering & Architectural building damage to east transverse end wall (photo: M. Motosaka).

Kuriyama that had been retrofitted with infill shear walls and diagonal bracing.

#### Tohoku University Campus.

At the Science and Engineering Aobayama Campus of Tohoku University, some buildings were severely damaged, possibly due to topographic amplification of ground motion, as the campus sits atop a hill 2 km west of Sendai City. Four buildings on the campus were so damaged that they were deemed irreparable (Motosaka et al. 2011), and there was serious damage to other buildings. The total damage estimate of \$400 million includes the replacement of the four buildings. The campus has had multiple damaging earthquakes over the past 30 years (Ms7.7 in 1978, M7.0 in 2003, Mw7.2 in 2005, Mw6.9 in 2008).

The Civil Engineering & Architecture Building, constructed in 1969, was badly damaged. This building, damaged during the 1978 Miyagi-ken-oki earthquake, had been seismically strengthened in two phases

prior to the 2011 quake (Motosaka et al., 2002; Motosaka, Sato and Yamamoto, 2004). The building is a steel-reinforced concrete building (in which the damaged columns were constructed of lattice steel sections encased in reinforced concrete) with three podium levels and seven tower stories above; this created a vertical irregularity in the mass, stiffness, and strength of the building at the transition level between the podium and tower.

After the Miyagi-ken-oki earthquake, the deflection angle of the building was on the order of 10-2 rad. The transverse end concrete walls of the tower were strengthened by additional concrete thickness during 2000 (Motosaka, Sato and Yamamoto, 2004). The concrete floor diaphragms at each end of the tower were also strengthened with additional concrete thickness. After an earthquake in 2003, additional steel braced frames were added to the south exterior to provide additional torsional stiffness to that side of the building, counteracting the stiffness of the original concrete core walls located on the



**Figure 9.** Damage to columns at tower-podium transition (photo: M. Motosaka).

north half of the floors. The university installed continuous monitoring equipment after the 2008 earthquake, which provided a detailed history of the building's response in micro-tremors and larger earthquakes (Motosaka et al., 2011).

The most severe damage in the 2011 quake was at the vertical boundary elements of the transverse concrete shear walls at the transition floor from the tower to the podium level, as shown in Figure 8. The concrete columns crushed with buckled vertical reinforcement are shown in Figure 9. Additional concrete damage was evident at the concrete floor lines, especially at the tower-podium transition floor.

By early June 2011, the damage to the wall boundary elements had been temporarily repaired with addition of concrete encasement over the height of the damaged column section and of high-strength tension rods to connect the concrete column above the damaged area to the concrete column below the damaged area. Additionally, new longitudinal concrete walls





**Figure 10.** Temporary column repairs with adjacent new concrete walls.



**Figure 11.** Electrical and Communication Engineering building, south elevation.

were added at the corners of the tower at the first floor above the podium transition (Figure 10).

Also seriously damaged was the eight-story steel reinforced concrete Electrical and Communication Engineering Building (Figure 11) built in 1966, which was still closed to access in early July. At the east end of the building at the transverse walls, there was significant damage to the concrete girders that span between two concrete shear walls, particularly at the openings through the girders for mechanical ducting (Figure 12). Reportedly, the damage to these elements has been continuing (with increased cracking and spalling) in the aftershocks. Residual deformation of the external water pipes at the upper stories of this building indicates some higher mode effects during the earthquake. The coupling beams had circular mechanical penetrations and exhibited shear failures and bond-splitting failures at others. The shaking in this building was strong enough to topple anchored pressurized gas cylinders on the 7th floor, while those on the

5th floor remained anchored. Also damaged were the walls of a two-story reinforced concrete mechanical penthouse above the roof (Figure 13).

The Applied Chemistry and Chemical Engineering Building (Figure 14), also of reinforced concrete construction, had been previously strengthened with an external steel braced frame system. Connection details for the retrofit are shown in Figure 15. This building appeared to perform well, with no visible evidence of distress in the bracing members or connections.



**Figure 12.** East elevation: Shear failures and bond-splitting failures of reinforced concrete coupling beams. Circular mechanical penetrations appear to have been installed after construction.



**Figure 13.** Severe damage to two-story reinforced concrete mechanical penthouse.





**Figure 14.** Steel braced frames in Applied Chemistry & Chemical Engineering building.



**Figure 15.** Steel strengthening of concrete building.

Steel jacket repairs to columns of a two-story wing of the Department of Civil Engineering and Architecture appeared to be unscathed, although delamination of fiber wrap jackets was observed at several stories of another two-story building nearby. Shear walls and diagonal steel braces added to the building also performed well, although there was heavy structural damage at the 3rd floor transition to the steel reinforced concrete tower.

## Part 2. Steel Buildings

*This section summarizes the main observations of the Steel Structures Group on structural damage in the Sendai region caused by the earthquake and its aftershocks, and includes observations on tsunami damage to industrial buildings and essential facilities in the Ishinomaki and Onagawa fishing ports. The*

*areas visited by the group included Sendai, Oroshohimachi, Ishinomaki, and Onagawa. This report is divided into three parts: 1) behavior of steel parking garages and other steel buildings; 2) behavior of cladding and external facades for steel structures; and 3) performance of steel buildings in tsunami-affected areas.*

Structural damage was concentrated in steel structures designed according to older Japanese seismic provisions. Typical failure modes included 1) net section fracture of braces; 2) gusset plate fractures due to low cycle fatigue, a consequence of the long-duration ground motions; and 3) foundation uplift in braced frames as well as panel zone yielding in steel moment-resisting frames. Typical failure modes in industrial buildings affected by the tsunami included 1) column twisting due to impact; 2) fracture and crack-

ing of steel rods and exposed base plates; and 3) toppling due to pile foundation failures.

**Parking Garages.** In the Oroshimachi area, largely flat coastal plain, two-story parking garages (Figure 16) had moderate to extensive structural damage after the  $M_w = 9$  earthquake and the main  $M_w = 7.4$  aftershock. The early 1990s buildings had shear beam-to-column connections with steel braced frames in both loading directions, a typical story height of 3m, and a typical bay width of about 5m. Poor detailing of steel braces and gusset plates were identified as the main weaknesses.

In both parking garages, the steel braces were designed not to buckle. However, plastic deformation was concentrated in the gusset plates in the area between the end of the brace and the overlap from the



**Figure 16.** Typical parking garages in the Oroshimachi area with moderate (left) and severe (right) structural damage.





**Figure 17.** Typical gusset plate design details and failure modes in garage 16a: (a) foundation uplift, (b) welded stiffener, (c) gusset plate yielding.

column (see Figure 17c). In the parking garage shown in Figure 16a, gusset plates buckled in both loading directions. The gusset plates in this parking garage included a welded stiffener in the center (see Figure 17b), generally to prevent the out-of-plane movement of the gusset plate. The unbalanced vertical load, transferred to the columns from the damaged steel braces, caused column base foundation uplift (see Figure 17a). These gusset plates were not designed according to standard guidelines followed in the United States (AISC, 2005a) or to more recent recommendations by Lehman et al. (2008). Japanese practice for gusset plate design does not include providing a zone for flexure, out-of-plane deformation. AISC guidelines (2005b [6.1-179]) expect weak-axis bending in the gusset plates and require a suffi-

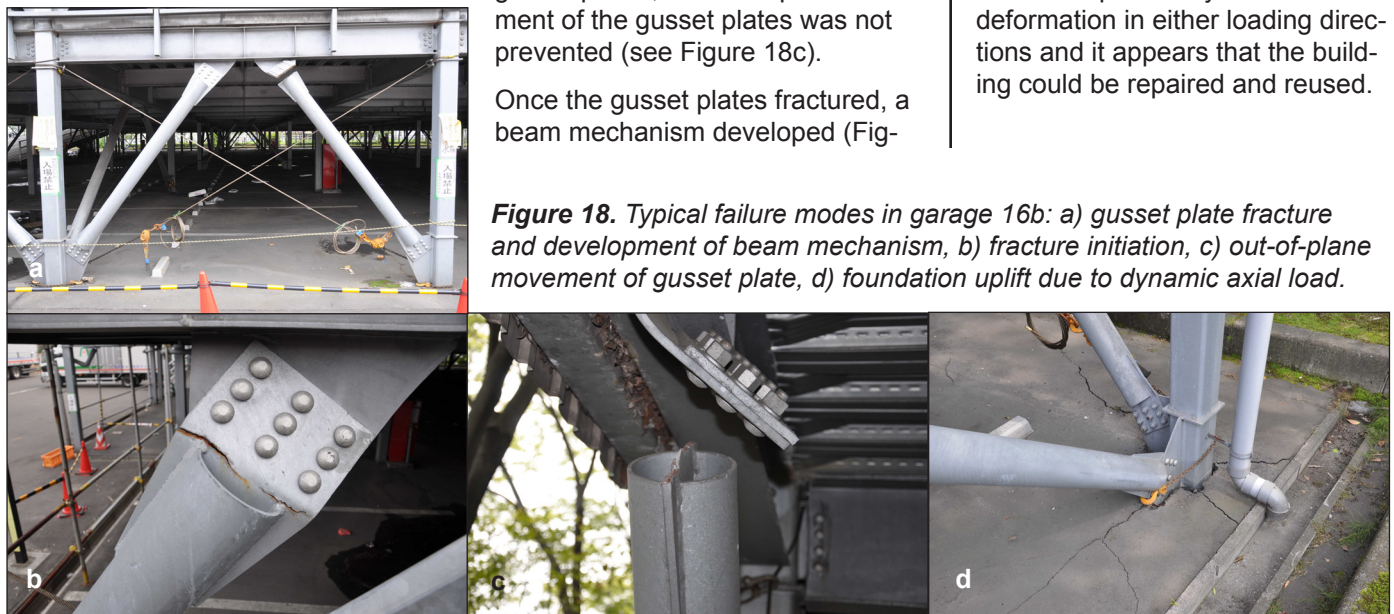
cient free length of plate to reduce the localized strains over the plastic hinge length at the gusset plate.

In the parking garage shown in Figure 16b, designed in 1991, all the top gusset plates of the steel braces in the east-west (EW) loading direction fractured due to low cycle fatigue (Figure 18a). This raises the issue of cumulative damage effect (cyclic deterioration and fracture due to low cycle fatigue) due to long-duration ground motions on steel components including beam-to-column connections (Chung et al. 2010, 2011; Lignos et al. 2011). In all the fractured gusset plates, fracture initiation started at the toe of the weld of the brace in the heat-affected zone and propagated along the length of the gusset plate after the many inelastic cycles of the ground motion (see Figure 18b). There was no stiffener welded in any of these gusset plates, so out-of-plane movement of the gusset plates was not prevented (see Figure 18c).

Once the gusset plates fractured, a beam mechanism developed (Fig-

ure 18a). The separation between the concrete slab and the top of the beam indicate unbalanced loading due to one brace losing stiffness, but it appears that the unbalanced load was successfully delivered through the beam in flexure to the column. This highlights the importance of beam mechanism design in steel braced frames.

In the north-south (NS) loading direction, all the top gusset plates of the steel braces buckled, but almost none of them fractured, implying that the strong component of the ground motion was applied in the EW direction. Extensive cracks were observed around the exterior columns of the garage due to the dynamic axial load that was transferred to these columns from the steel braces in both loading directions. Notwithstanding the damage, there was practically no residual deformation in either loading directions and it appears that the building could be repaired and reused.



**Figure 18.** Typical failure modes in garage 16b: a) gusset plate fracture and development of beam mechanism, b) fracture initiation, c) out-of-plane movement of gusset plate, d) foundation uplift due to dynamic axial load.





**Figure 19.** Typical failure modes in old Japanese construction design: (a) lateral torsional buckling, (b) net section fracture, (c) net section fracture, (d) panel zone yielding.

**Industrial/Residential Buildings.**

There were varying levels of damage in two- to four-story residential steel buildings with a steel moment frame in one loading direction and a braced frame in the other. The most common failure modes observed in braced frames involved (1) lateral torsional buckling of double angle steel braces with X-brace configurations due to thin gusset at the crossing of the double angle braces (Figure 19a), (2) net section fracture (Figures 19b and 19c), and (3) panel zone yielding (Figure 19d).

**External Facades.** Widespread damage to external cladding was observed in office buildings, warehouses, factories, and residential buildings. In Japan, steel industrial buildings constructed before the 1970s used timber lath or corrugated metal coated with mortar, or other exposed corrugated metal, for their external finishes; currently, how-

ever, Autoclaved Lightweight Concrete (ALC) panels (known as Autoclaved Aerated Concrete or AAC in the U.S.) are the first choice for non-load bearing external cladding for steel structures. Prefabricated, standardized ALC panels are used for residences constructed of light-gauge metal, and for steel office buildings, warehouses, and industrial facilities. ALC panels have a minimum specified compressive strength of 3.0MPa (Okazaki et al., 2007). When used for external wall cladding, the ALC panels have a minimum thickness of 100mm and are typically used in 600mm wide modules.

Widespread damage to all types of external finishes was observed in the Oroshohimachi area. Figure 20 shows an older residence where large blocks of mortar that covered timber lath fell. Figure 21 shows a three-story office building where ALC panels fell off a steel moment frame over

one quarter of the perimeter, and where the remaining ALC panels show extensive damage along the panel boundaries. Figure 22 shows a warehouse building where ALC panels fell from the second story. These buildings were two- to three-stories high, and believed to be at least ten years old.

Figure 23 illustrates the typical method of ALC panel installation that was used until ten years ago. All ALC panel damage observed by the investigation team, including that shown in Figures 21 and 22, used this installation method. The observed damage to ALC panels was likely caused by the inability of the panels to accommodate story



**Figure 20.** Cladding damage to older residential building.

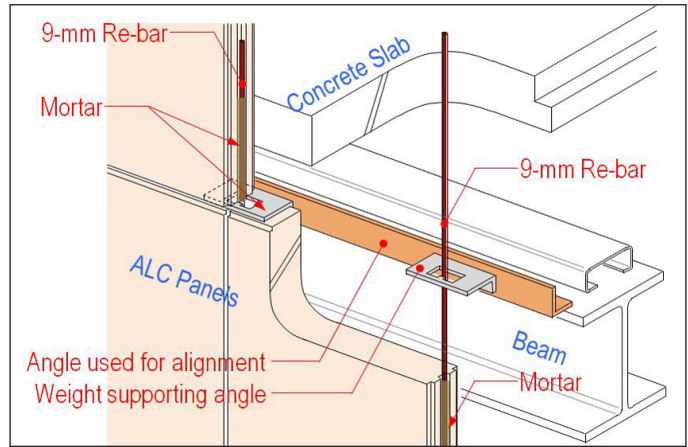


**Figure 21.** ALC panels fallen from a three-story office building.





**Figure 22.** ALC panels fallen from a two-story office-and-warehouse building.



**Figure 23.** Older method of ALC panel installation used up to ten years ago.

drift of the structural frame, even though the story drift was rather limited. In most buildings with severe damage to ALC panels, and hence with their structural framing exposed, no evidence was found of structural damage; an exception was the three-story braced frame building discussed above (Figures 19a and b).

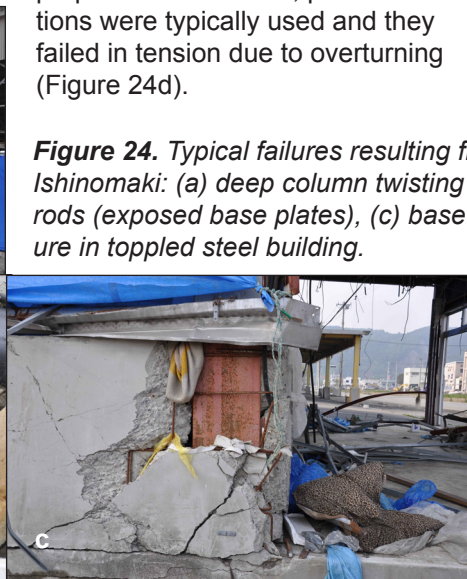
Little damage to ALC panels was observed in newer single-family residences. Most of these residences were believed to have been constructed of light-gauge metal by mass-production companies.

**Tsunami Damage.** Well-designed steel industrial buildings subjected to the earthquake did not collapse,

but a number of industrial buildings built close to the fishing ports were damaged by tsunami-carried debris. The most common failure mode associated with tsunami was steel column global buckling in addition to local and/or lateral torsional buckling (Figure 24a). This problem was acute when deep W sections were used in the design because these columns are susceptible to lateral torsional buckling. Other failure modes — due to water pressure from the tsunami waves — involved steel rod fractures of exposed base plate connections (Figure 24b), and cracking of base plate connections (Figure 24c). In the Onagawa area there were at least five buildings that toppled over, two of which were steel structures of three and four stories. Due to poor soil properties in this area, pile foundations were typically used and they failed in tension due to overturning (Figure 24d).

The team observed two hospitals that had been damaged by the tsunami. The town hospital in Onagawa is located on a hill above the harbor, at approximately 12-13m in elevation (Figure 25). Onagawa lies at the end of a narrow harbor and had tsunami heights reaching 15m that completely destroyed all buildings within the inundation zone.

The hospital campus includes a four-story acute care wing and a three-story clinic building, interconnected by an elevated pedestrian bridge at the second level. The ground level of the hospital showed evidence on the corridor walls of about 4 feet of water (Figure 26). This level was not operational in early June, nearly three months after the tsunami, with clean-up operations in progress, but the hospital's upper floors were providing some medical services. One of



**Figure 24.** Typical failures resulting from tsunami action in Onagawa and Ishinomaki: (a) deep column twisting (ramming), (b) tensile fracture of steel rods (exposed base plates), (c) base plate cracking, (d) pile foundation failure in toppled steel building.





**Figure 25.** Onagawa hospital on hill above the town.



**Figure 26.** Water signs, first story of Onagawa hospital.

two elevators that serve the hospital building was not operable and boarded up at the lobby.

The Ishinomaki City Hospital, a modern four-story building, is located adjacent to the mouth of a river that empties into the bay and appeared to have been badly damaged by the tsunami (Figures 27 and 28). In early June, the hospital was not in service because of significant damage to the ground floor, although the upper floors did not appear to have significant damage.

The Onagawa Hospital was able to maintain limited services due to its location on higher ground, while the Ishinomaki hospital — barely above sea level — was closed to patients and staff. Considering the known tsunami hazard on the Japanese coastline, site selection for essential

facilities should be a significant part of planning for new or rebuilt facilities.

### Part 3. Protective Systems

*Between June 1 and June 6, 2011, the team visited 17 buildings in or near the city of Sendai in Miyagi Prefecture, and in the Tokyo area. Of these buildings, six incorporated supplemental damping systems, and 11 incorporated seismic isolation systems, with supplemental dampers acting in parallel with the isolation system in some cases.*

Among the thousands of buildings shaken by the earthquake in eastern Japan were many buildings that incorporate special protective systems; for example, seismic isolation or supplemental damping systems. It is probably not an exaggeration to say that more buildings with protec-

tive systems were affected by the Tohoku earthquake than had been in all previous global earthquakes combined. Since many of these buildings were instrumented, the earthquake created an unprecedented body of performance data for the structures.

In Japan, protective systems for buildings (seismic isolation, supplemental damping, and hybrid systems) are much more widely used than in the United States. There are fewer than 100 seismically isolated buildings in the United States, but in Japan there are more than 2,600 commercial buildings and more than 3,800 single-family homes with seismic isolation systems, according to the Japan Society for Seismic Isolation (JSSI). The magnitude and duration of the Tohoku quake produced



**Figure 27.** Ishinomaki City hospital.



**Figure 28.** Main lobby of Ishinomaki City hospital.





**Figure 29.** Sendai MT Building.

movements in some systems that far exceed responses measured in any previous quake. In the United States, we have based design of earthquake protection systems on the results of advanced analysis and shake table testing; however, the Tohoku earthquake has allowed us to observe the performance of full-scale structures subject to long-duration, low-frequency, high-amplitude ground shaking.

#### Instrumentation and Sensors.

Real-time monitoring of structural parameters is prevalent in structures with protective systems, and the instruments provide engineers and owners with valuable information about the structural health of buildings after major earthquakes. Universities have typically been forthcoming with data on the performance of structures they own or have instrumented; in contrast, private corporations and construction companies have been reticent about releasing the data gathered in their structures.

Electronic instrumentation data is typically reported in the form of acceleration and displacement records. For seismically isolated structures, scratch plates are often installed, and serve as a simple



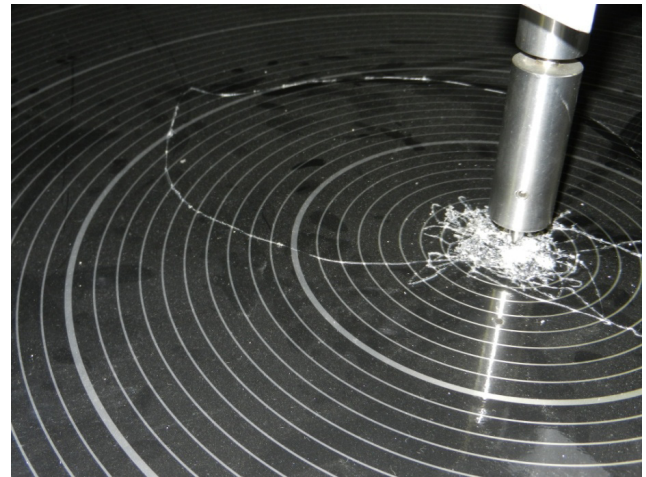
**Figure 30.** (left) Rubber isolation bearing, (right) sliding isolation bearing.

method for obtaining information about maximum displacement and its direction. The scratch plate data also assist with calibration of displacement records. Raw acceleration records are processed through double integration to create uncorrected displacement records. In a parallel effort, the structure's modal properties — vibration periods, damping ratios, and participation vectors — are extracted from the processed acceleration records to create an analytical model of the structure (using modal superposition). This model is used to check the displacement records obtained through double integration of acceleration records, and maximum displacements are also compared with scratch plate data. Free vibration data from micro tremors or aftershock shaking also allow researchers to extract the structure's damping properties.

#### Buildings with Seismic Isolation Systems.

The 18-story Sendai MT office building in Sendai City (Figure 29) was constructed in 1999 and is supported on a combination of 26 rubber bearings and 10 flat sliding PTFE bearings (Figure 30).

During the Tohoku earthquake, the maximum displacement of the isolation system was approximately 23cm, as recorded by a scratch plate at the



**Figure 31.** Scratch plate indicating maximum isolation system displacement of 23cm.

isolation level (Figure 31). The peak ground acceleration recorded below the isolation level was 311cm/sec<sup>2</sup>. Accelerometers in the isolated superstructure showed that peak floor accelerations were in the range of one-half to two-thirds of the peak ground acceleration over all 18 stories.

There was no structural damage in the building and it was fully usable after the earthquake. Minor non-structural damage was observed in some joints covering the isolation gaps at the base of the building, but this damage was repairable and did not affect the functionality of the building.

The Yozemi Tower (Figure 32) in central Tokyo incorporates a seismic isolation system combined with a semi-active damping system at the base isolation level. The building has 26 stories above grade, and three stories below grade. The



**Figure 32.** The Yozemi Tower in the Shinjuku area of Tokyo.

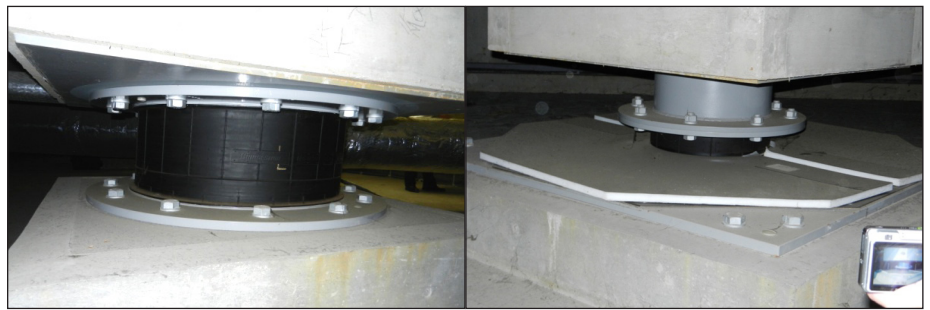
isolation level is between the 1st and 2nd stories of the basement so that the total height of the isolated structure is 27 stories.

The isolation system consists of 25 rubber bearings (Figure 33a) and 24 flat sliding PTFE bearings (Figure 33b). The isolation damping system includes 12 semi-actively controlled oil (hydraulic) dampers (Figure 34) and 12 passive oil dampers. The semi-active dampers may be switched between two damping coefficients, depending on the structural control requirements. Accelerometers and displacement meters within the building provide

feedback to the control system (Figure 35), which switch-



**Figure 36.** Tokyo Institute of Technology Building J2.



**Figure 33.** (left) Rubber bearing, (right) sliding PTFE bearing.



**Figure 34.** Semi-active damper.

es the damping coefficient of the semi-active dampers to optimize structural response during an earthquake.

In addition, the lateral force-resisting system in the upper floors of the building incorporates viscoelastic (VE) material dampers. In these dampers, the VE material is located in the annular space between two telescoping brace pipes. The VE dampers improve not only the seismic response of the structure, but also control wind-induced accelerations, to improve occupant comfort.

The maximum observed isolation system displacement in the earthquake was approximately 10 cm. There was no reported structural or nonstructural damage, and the building remained fully functional after the earthquake.



**Figure 35.** Control system accelerometer mounted on superstructure.

The 20-story J2 building of the Tokyo Institute of Technology (Figure 36) is a base-isolated building located on the Suzukakedai campus in Yokohama. The isolation system includes 16 rubber bearings acting in parallel with 14 yielding metal dampers and two oil (hydraulic) dampers (Figure 37a).

The plan dimensions of the building are roughly 46m by 18m, and the height is approximately 91 m. Thus, the aspect ratio (height/width) of the building in one direction is approximately 5, which creates the potential for uplift forces acting on rubber bearings during strong ground shaking. To counteract this effect, certain bearings are outfitted with conical

Belleville spring washers on the mounting bolts (Figure 37b); if uplift occurs, the Belleville springs will limit it without inducing large tensile strains within the rubber. The effects of potential uplift were considered in the dynamic analysis and design of the structure.



**Figure 37.** (left) Oil damper, (right) rubber bearing with steel dampers.





◀ **Figure 38.**  
Main building at the Shimizu Institute of Technology.

**Figure 39.** ▶  
One of six lead rubber isolators supporting the main building.



**Figure 40.** ▲  
Frame from video at <http://www.shimz.co.jp/english/theme/earthquake/effect.html>.

During the earthquake, the maximum measured radial displacement of the isolation system was approximately 12.5 cm (9.5 cm in the NE-SW direction). It is not known at this time whether there was uplift at any of the rubber bearings. The peak ground acceleration measured below the isolation plane was 69.0 cm/sec<sup>2</sup> in the NE-SW direction. At the first level above the isolation plane, the maximum acceleration in the NE-SW direction was 69.6 cm/sec<sup>2</sup>. The maximum acceleration at the 20th floor in the NE-SW direction was 116.2 cm/sec<sup>2</sup>. Comparing the peak ground acceleration and the peak acceleration at the 20th floor, it can be seen that the isolation system was highly effective at minimizing dynamic amplification of ground motions over the height of the building.

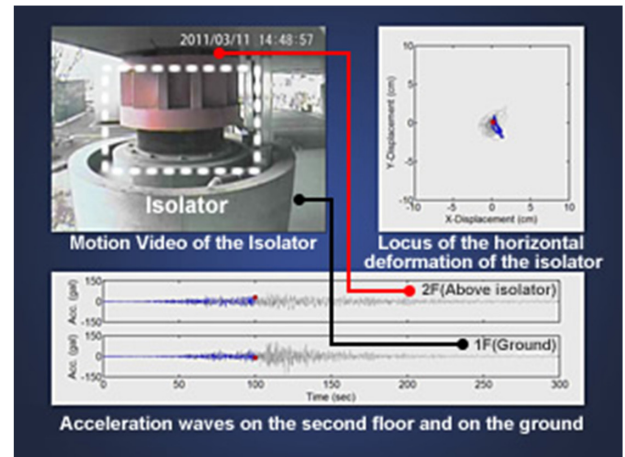
It is interesting to note that a second base-isolated tower is currently under construction approximately 10 m southeast of this building (visible on the right in Figure 36). The second tower will also be 20 stories tall, and it will be base-isolated. The towers will share a common base level, so that they will act together as a single base isolated structure.

There are three base-isolated buildings at the Shimizu Institute of Technology campus in Tokyo, a research campus for the Shimizu Corporation, one of the largest construction companies in Japan. The Safety & Security Center building

incorporates a unique core-suspended isolation system; the Wind Tunnel Testing Facility is constructed on a seismic isolation system that is partially submerged in water in order to take advantage of buoyancy to improve seismic performance; and the six-story main building is isolated on six 1.1-m-diameter lead-core rubber bearings (Figures 38 and 39). The main building is of particular interest because, during the earthquake, a monitoring system captured live video of the seismic isolation system at work (Figure 40).

The measured maximum displacement of the isolation system in the main building was approximately 9cm. The peak ground acceleration below the isolation system was 132cm/sec<sup>2</sup> and, at the second floor just above the isolation system, the peak acceleration was 69cm/sec<sup>2</sup>. There was virtually no dynamic amplification over the height of the building, as at the 6th (top) floor the peak acceleration was 72cm/sec<sup>2</sup>. There was no reported structural or nonstructural damage in the main building.

**Seismic Isolation Joint Covers and Connections.** The base-isolated buildings inspected by the EERI team all performed well in the earthquake from the standpoint of structural response, but in some cases we observed minor damage to nonstructural elements associated with seismic iso-



lation “moat” covers (see Figure 41) or cover plates over seismic isolation joints between base-isolated and fixed-base buildings. This damage could generally be classified as cosmetic since it did not inhibit the function or recovery of the building; however, it appeared that the damage could have been avoided by designing the moat and joint



**Figure 41.** Damage to moat cover of base isolated building.





**Figure 42.** Tohoku Institute of Technology Building B10 supplemental damping.

covers for larger ranges of motion. In Japan, as in the United States, seismic moat covers and joint covers are often not designed by the structural engineer, but by the architect. The architect may not have the same detailed understanding as the engineer about the expected ranges of motion in the base-isolated building. Therefore, it is important that both the architect and engineer participate in the design of seismic moat and joint covers, to assure that the full range of potential motion is provided for.

**Buildings with Supplemental Damping Systems.** The Tohoku Institute of Technology campus outside of Sendai has two structures with protective systems; both were damaged during the earthquake. One of these structures, Building 10, was constructed in 2003 and demonstrates how excessive demands on dampers can lead to partial failure of the system. Building 10 is a seven-story structure with moment-resisting frames and four dampers in each direction at each floor. The first level of Building 10 is 8m high, and all upper stories are 3.8m high.

Presumably, the damping system was intended to control drifts and shaking duration during small events. Each damper frame bay holds a chevron brace with a horizontal damper connected between the tip of the chevron brace and a snubber assembly mounted at the



**Figure 43.** Tohoku Institute of Technology Building 5 with exterior bracing and interior supplemental damping.

floor level below (see Figure 42). The damper device contains oil that is forced through a narrow orifice, in addition to a viscoelastic material that is engaged at the end of the damper stroke.

The available stroke of each damper is approximately 12mm, but the limit was exceeded during the March 11 quake at the first floor dampers (the first floor is taller than other floors). This led to yielding of the snubber assembly and, ultimately, to complete failure of the damper in one E-W (longitudinal) frame at the first story.

The building was instrumented with accelerometers and, in the 2005 Miyagi earthquake, a peak ground acceleration of 101 cm/sec<sup>2</sup> was recorded at Building 10, as were roof accelerations of 159 cm/sec<sup>2</sup>. When compared with similar surrounding structures without damping systems, where recorded roof accelerations were 290 cm/sec<sup>2</sup> and 459 cm/sec<sup>2</sup>, this demonstrated successful control of amplification of ground motions over the building height. However, though measurements were taken during earthquakes before and after the main March 11 quake, data were not recorded during the big one because the data acquisition computer was damaged by ground shaking. By

analyzing the data that were recorded before and after the big quake, researchers were able to determine that the fundamental period of the structure elongated from 1.2 seconds to 1.3 seconds due to the loss in stiffness of the damped chevron brace that failed at the first floor. The moment frame and remaining damped bays of the superstructure were undamaged. After professors at Tohoku Tech determined that the loss of a single damped chevron brace at the first story would not compromise the safety of the building, Building 10 was re-opened at the same time as the rest of the institution, one month after the Tohoku earthquake.

Near Building 10 is an older structure, Building 5, an eight-story structure constructed in 1971 with a force-resisting system consisting of a nonductile concrete frame combined with concrete walls in the transverse (N-S) direction. In 1978, the Miyagi-ken-oki earthquake damaged the building, which was subsequently repaired and strengthened. The retrofit scheme consisted of steel cross-brace frames located on the exterior of the building in the longitudinal (E-W) direction (see Figure 43). The cross braces were constructed of wide flange steel



shapes with reduced flange sections near the ends of the braces. The intention was to concentrate brace yielding at these reduced sections. The 2005 quake demonstrated the need for more earthquake protection so additional chevron braces with oil dampers were added at certain exterior concrete frame bays, and critical concrete columns were strengthened with fiber wrapping.

During the 2011 earthquake, the peak ground accelerations at the base of three buildings surrounding Building 5 were 336-354cm/sec<sup>2</sup>, and the peak acceleration measured at the upper floor of Building 5 was 820cm/sec<sup>2</sup>. Building 5 is devoted mainly to office spaces and classrooms, and the occupants there during the quake reported that the shaking was severe and lasted for a long time. The spaces had severe nonstructural damage: the ceilings were stripped of light fixtures and the walls of bookshelves. Some of the exterior braces yielded at the reduced flange sections as intended, but some braced bays also buckled outwards, resulting in distorted gusset plates at the intersections of cross braces. The damage visible three months after the earthquake included the distorted gusset plates in the exterior braces and cracked interior shear walls in the transverse direction.

In Japan, there are numerous high-rise buildings (over 60 meters tall) that employ either base isolation or supplemental damping systems to attenuate earthquake forces and reduce undesirable deformations. In the United States, we have not typically applied seismic isolation to high-rise buildings, and we have applied damping systems to only a limited number of high-rise buildings. In Tokyo, high-rises represent a significant portion of the building stock. Given recent advances in characterization of earthquake hazards and design ground motions, it is understood that some vintage

high-rises may be at risk, since they were designed without consideration of long-period ground motions. Since 2000, Japanese codes have required design for velocity response spectrum values of 80cm/s up to a period 10 seconds, but prior to 2000, three ground motions (El Centro, Taft, and Hachinohe) were used for design of high-rise structures. These ground motions have much smaller peak ground velocity values for periods above 3 seconds than current code requirements. Retrofitting high-rise buildings by installing dampers is an effective means of mitigating the effects of shaking, and has been employed in many structures, but long-duration events remain a concern for two reasons: the frequency of the shaking, and because the duration of shaking can induce cumulative damage.

The EERI team visited the Shinjuku Center, a 54-story building in the Shinjuku area of Tokyo constructed in 1979. The structure's height is 223m, and the first natural period of the structure is 5.2 and 6.2 seconds in two perpendicular directions. In 2009, the building was retrofitted from the 15th to 39th floor with 288 oil dampers that were configured to exhibit a form of deformation dependency, in addition to velocity dependency (see Figure 44). Taisei Corporation developed this damper type for reinforcing existing high-rise buildings against long-period motions. Because the deformation-dependent damper is designed to reduce the damping force at the instant the frame deformation comes close to its maximum value, engineers determined that no reinforcement was needed in the surrounding columns, girders, and foundations. In the March quake, the dampers were calculated to have reduced the maximum accelerations by 30% and roof displacement by 22%. Acceleration data are not available at this time, but we do know that shaking in the structure lasted nearly 10 minutes without any reported structural or nonstructural damage.



**Figure 44.** Shinjuku Center Building retrofitted with damper brace.

**Conclusions.** Seismically isolated buildings performed well during low-frequency, long-duration shaking with ground motion demands approaching or exceeding design requirements. In some cases, moat covers and joint covers around seismically isolated buildings were damaged because they were not designed for the full required range of motion. High-rise buildings with supplemental damping systems exhibited reductions in structural displacements and accelerations, as well as in duration of shaking. The earthquake also demonstrated that protection systems such as seismic isolation and supplemental damping are effective at reducing nonstructural earthquake damage.

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## References

- AISC, 2005a. *Seismic Design Manual*. Chicago: American Institute of Steel Construction, Inc.
- AISC, 2005b. *Seismic Provisions for Structural Steel Buildings, #341-05*. Chicago: American Institute of Steel Construction, Inc.
- Chung, Y-L., et al., 2010. "Seismic resistance capacity of high-rise buildings subjected to long-period ground motions: E-Defense shaking table test," *Journal of Structural Engineering*, ASCE, Vol. 136 (1), pp. 637-644.
- Chung, Y-L., et al., 2011. "Seismic resistance capacity of beam-to-column connections in high-rise buildings: E-Defense shaking table test," *Earthquake Engineering and Structural Dynamics*, EESD, Vol. 40, pp. 605-622.
- Lehman, D.E., et al., 2008. "Improved seismic performance of gusset plate connections," *Journal of Structural Engineering*, ASCE, Vol. 134 (6), pp. 890-901.
- Lignos, D.G., et al., 2011. "Numerical and experimental evaluation of seismic capacity of high rise buildings subjected to long duration earthquakes," *Computers and Structures*, Vol. 89 (11-12), pp. 959-967.
- Motosaka, M., et al., 2002. "Evaluation of seismic damage and reinforcement effects of an existing building based on forced vibration tests before and after repair work," *Proceedings, Structural Dynamics, EURO DYN*.
- Motosaka, M., T. Sato, and Y. Yamamoto, 2004. "The amplitude dependent dynamic characteristics of an existing building before and after seismic retrofit." Paper #1023 in *Proceedings, 13th World Conference on Earthquake Engineering*, Vancouver, Canada.
- Motosaka, M., et al., 2011. "Change of dynamic characteristics of a damaged building before, during, and after the 2011 off pacific coast Tohoku earthquake," *Proceedings AIJ Convention* (in Japanese).
- Otani, Shunsuke, 2000. "Japanese Development of Earthquake Resistant Building Design," Invited Paper, *Proceedings, International Symposium on Earthquake Engineering*, University of Montenegro, pp. 19-33.
- Okazaki, T., et al., 2007. "Interaction between cladding and structural frame observed in a full-scale steel building test," *Earthquake Engineering and Structural Dynamics*, EESD, Vol. 36, pp. 35-53.
- USGS, 2012. "Magnitude 9.0 Near the East coast of Japan March 11: Earthquake Summary." Last updated January 1. <http://earthquake.usgs.gov/earthquakes/eqinthenews/2011/usc0001xgp/#summary>