Learning from Earthquakes

Geotechnical Effects of the $M_w$ 9.0 Tohoku, Japan, Earthquake of March 11, 2011

Teams from the Geotechnical Extreme Events Reconnaissance (GEER) Association have contributed to reconnaissance on the geotechnical effects of the Tohoku, Japan, earthquake, in collaboration and coordination with colleagues in Japan and teams from the Earthquake Engineering Research Institute (EERI), the Pacific Earthquake Engineering Research (PEER) Center, the American Society of Civil Engineers (ASCE), and the Federal Highways Administration (FHWA). A list of team members and contributors is provided at the end of this report. All photos are credited to GEER 2011 unless otherwise indicated.

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Introduction

The March 11, 2011, earthquake—one of the largest in recorded history worldwide—struck along the subduction zone interface plate boundary between the Pacific and North American plates. The earthquake was immediately followed by a large tsunami that inflicted widespread damage to modern urban infrastructure and led to a crisis at the Fukushima Daiichi nuclear power plant. The world’s attention was captured by the scale of the devastation and by the nuclear crisis.

The geotechnical effects of this earthquake provide important opportunities for learning about the seismic performance of a wide range of geotechnical systems and constructed facilities. Facilitated by the extensive network of strong ground motion recording stations in the area, the unique characteristics of this large earthquake, the wide geographical area affected, and the modern infrastructure throughout the affected areas, scientists will be able to gain insights into the correlation between ground motion characteristics and the type and extent of damage—in ways not previously possible.

This report provides a brief summary of preliminary geotechnical observations from NSF-sponsored GEER reconnaissance teams working with other organizations and individuals, as noted above. Additional details regarding some of these observations are included in a series of GEER Quick Reports available at www.geerassociation.org. The field teams could not cover all the affected areas, and avoided some locales due to ongoing humanitarian aid efforts and radiation concerns. Fuller coverage and greater detail on the geotechnical effects will emerge from the studies currently underway in Japan.

Seismicity

The March 11 earthquake resulted from thrust faulting along the subduction zone interface plate boundary between the Pacific and North American plates off the eastern shore of Honshu. At that latitude, the Pacific plate moves approximately eastward at about 83 mm/yr relative to the North American plate. The rupture surface extended approximately 600 km parallel to the coast of Japan, and the maximum slip on the rupture surface has been estimated at about 32 m (USGS, 2011).

Finite source imaging studies have shown that the rupture process involved a series of distinct phases. Ide et al. (2011) concluded that the rupture consisted of a small initial phase, deep rupture for as much as

Figure 1. Finite source imaging of rupture process (source: Ide et al., 2011).
40 sec, extensive shallow rupture at 60-70 sec, and continuing deep rupture lasting over 100 sec (Figure 1). They further concluded that shallower parts of the rupture radiated weakly at high frequencies, whereas deeper parts radiated strongly at high frequencies.

The mainshock was the most well-recorded earthquake to date worldwide. It produced 693 recordings at K-Net sites (surface instruments), 496 recordings at Kik-Net vertical arrays, and 50 recordings at buildings instrumented by the Japan Building Research Institute. Initial comparisons of strong ground motion recordings to ground motion prediction equations (GMPE) for subduction zone events have provided valuable insights on the capabilities and limitations of some GMPEs. Preliminary observations by Boore (2011) suggest that recorded motions appear to decay more rapidly than predicted by many GMPEs. For distances less than about 150 km, the short-period motions were better predicted by GMPEs than the longer-period motions. The GMPEs by Zhao et al. (2006) and Kanno et al. (2006), both of which were developed using data from Japan, appear to compare best with the recordings.

The March 11th earthquake was preceded by a series of large foreshocks during the previous two days, beginning on March 9th with an $M_w$ 7.2 quake approximately 40 km from the March 11th earthquake, and continuing with three earthquakes greater than $M_w$ 6 on the same day. Aftershocks included the $M_w$ 6.6 Hamadori earthquake of April 11th in the shallow crust above the subduction zone and produced significant surface rupture.

Several recording stations produced particularly interesting records for the March 11th quake. A peak horizontal ground acceleration (PGA) of 2.7 g was recorded at the K-Net Tsukidate (MYG004) station. The soil profile consists of 1 m of fill ($V_s \approx 100$ m/s) overlying 3 m of lower-velocity bedrock ($V_s \approx 240$ m/s) overlying stiffer bedrock ($V_s \approx 550$ m/s). The recorded motion was rich in high frequencies, which may partly explain why the nearby residential and commercial buildings appeared to suffer only minor damages (Midorikawa and Miura, 2011).

Evidence of liquefaction was observed at seven strong ground motion recording stations in the Tokyo and Kanto Plain areas (Figure 2). Another seven recording stations with soil profiles that would be considered susceptible to liquefaction did not exhibit signs of it. These sites will be the focus of ongoing site characterization efforts by GEER, in coordination with broader studies by Japanese researchers.
appeared to be primarily fill materials or young alluvium.

Liquefaction caused extensive damage to light residential and light commercial structures in many of the areas visited, with the magnitudes of the settlements and tilts larger than often observed for such light structures. Tilts of up to 2 or 3 degrees were observed in many cases (Figure 3). Many of these structures were founded on mat-type foundations with deep grade beams (Figure 4) that limited damage to the superstructures despite the large settlements and tilts. Figures 5 and 6 illustrate a case in Urayasu where the sidewalk and street settled approximately 30 cm relative to a building on piles, while an adjacent three-story building on a mat settled 40 cm more than the adjacent ground surface (that is, 70 cm relative to the pile-supported building), and tilted noticeably without observable damage to the superstructure.

Liquefaction-induced ground surface and building settlements were often observed to vary significantly over short distances. Differences in settlements at some of these locations (Urayasu) may be related to the differences in the dates of fill placement and ground improvement (Tokimatsu et al., 2011). The boil materials did not appear to differ substantially in characteristics across some of these locations. These data may allow examination of whether the effects of age and differences in fill source materials are adequately reflected in the results of in-situ tests and accounted for in existing engineering procedures. Moreover, the large dataset on settlement patterns and rotations for buildings on shallow foundations (Figures 3, 5, and 6) at these sites provides a similarly unique opportunity for testing engineering procedures for predicting foundation performance in the presence of liquefaction.

Liquefaction-induced damage to utilities caused widespread disruptions for homeowners and businesses, described later in the section on lifelines.

Areas known to have been improved by sand compaction piles and other techniques were observed to have performed well, in that ground surface displacements were not observed.

The detailed studies of liquefaction effects underway in areas such as Urayasu can be expected to produce significant findings in the near future.

Future GEER efforts will include Spectral Analysis of Surface Waves (SASW) and Cone Penetration Test (CPT) testing at select sites in coordination with Japanese researchers.

Levees

Hundreds of kilometers of levees border several rivers in the Tohoku and Kanto regions in northeastern Japan. Data on levee damage resulting from the earthquake and tsunami, documented immediately following by the Ministry of Land, Infrastructure, Transport, and Tourism (MLIT), were graciously shared with the GEER team. Our field efforts were largely limited to the eastern parts of Miyagi and Ibaraki prefectures.

Most levee reaches performed well, with little or no damage or distress. This generally good performance may be partly attributed to the fact that river levels were relatively low at the time of the earthquake, and thus the majority of levee embankments were not saturated. In many areas, settlements of nondistressed levees appeared to be about 7-15 cm relative to structures such as bridge piers or buried water conveyance structures.
Nonetheless, numerous levee reaches sustained moderate to major damage (Figures 7 and 8). Settlements of 20-40 cm were observed in many areas, with greater settlements in reaches of relatively limited lengths (commonly only 100-300 m in length). Major damage can be ascribed mostly to foundation liquefaction. Little or no flooding resulted because the water levels were low and within channels, even with levees that were almost totally destroyed by foundation liquefaction.

Near river mouths, many levees were overwhelmed by tsunami waves that caused serious damage to the levees and floodwalls. In some tsunami-affected areas of the river systems, liquefaction-induced damage and levee settlement may have resulted in concentrations of overtopping flows by the subsequent tsunami waves and, subsequently, more overtopping damage to the levee and adjacent areas.

We observed that interim repairs had been completed within six weeks for most of the levees seriously damaged by liquefaction outside the tsunami areas. The repairs generally consisted of the following components:

- Removal of broken revetments and paving,
- Placement of new earth fill into cracks and return of levee section up to original grade,
- Placement of new buttresses or berms where needed,
- Placement of straw mats on landside slope to provide slope protection,
- Placement of articulated concrete blocks (cabled together) on waterside slope and portions of restored levee crown to armor these areas,
- Placement of gravel road base on center of levee crown.

In some areas, a double row of sheetpiles filled with soil was used as an interim retaining structure instead of, or in addition to, the repair of the levee itself. Figure 9

**Figure 7.** Major slumping of landside slope of Naruse River right levee and approach road at river km 30.0 (N38.5307, E 141.0064, April 20, 2011).

**Figure 8.** Levee damage along the Naruse River left levee at river km 11.3 (N38.4538, E141.1087, April 21, 2011).

**Figure 9.** Construction of large waterside berm as part of an interim repair of the Naruse River left levee at river km 30.1 (N38.5333, E 141.0050, April 20, 2011).

**Figure 10.** Later construction of a double-row sheet-pile barrier through the waterside berm shown in Figure 9 (N38.5333, E 141.0050, June 3, 2011).
shows the construction of a large waterside berm as part of the initial phase of an interim repair for a levee on the Naruse River. This repair is on the left levee at River Kilometer 30.1, directly across the river from the slumping on the right bank shown in Figure 7. As shown in Figure 10, a double-row of sheet-piles was later placed through this new berm.

Four levee reaches along the Naruse River that had major liquefaction damage during the 2003 Miyagi North Continuation earthquake (Mw 6.2), and that had been repaired with a mix-in-place soil cement foundation ground improvement technique, were inspected during our reconnaissance. All four sites appeared to have performed well, with no observable damage from the apparently stronger 2011 earthquake sequence. Notably, two of these sites had moderate liquefaction-related levee damage directly beyond the limits of the ground treatment. In some cases, this resulted in significant transverse cracks at the contact transition from the treated to nontreated sections of the levees. While the cost for the 2003 treatment was probably significant, it was successful in preventing major liquefaction-related damage. However, the cracks between some of the treated and nontreated levee reaches may be an issue that needs to be considered and addressed in future ground improvement projects.

The Hinuma River flows from Lake Hinuma to the ocean near the town of Mito and is bordered on the west by levees that had relatively extensive damage from foundation liquefaction (Figure 11). Apparently founded on dredged fill/reclaimed land, the levee cracked and slumped continuously for over 2.5 km along the western margin of the shallow, near-coastal lake. In some areas, after the earthquake new fill had been placed into some reaches where it appeared that the river/lake might breach the slumped levee. The difference in the length of damage along this levee, as opposed to the more limited (but multiple) damaged reaches in other locations, appears to reflect the presence of man-made foundation materials and/or of lake water that was high enough to saturate both the foundation and the lower portions of the embankment. In some lengths along the western levee, the water level was about a meter higher than the landside ground surface. Exposed embankment material in the numerous large cracks indicated that at least the upper fill was composed of clayey sands and gravels. However, the numerous sand boils found in the cracks were generally clean, fine to medium-grained sands—the latter material consistent with dredged or hydraulic fill. The length of liquefaction damage has significant implications for saturated levees constructed of or on dredged material, such as those in the Sacramento-San Joaquin River Delta in California’s Central Valley.

Dams

Japanese authorities inspected over 400 dams following the earthquake, almost all of which performed relatively well. Many dams withstood severe ground shaking, with generally minor to moderate damage (Matsumoto, 2011). As examples, the 83.5-m-high Shitoki rockfill dam, completed in 1984, had minor settlement and cracking, and the Nishigo Dam, a 32.5-m-high earthfill dam constructed in 1955, developed about 20 cm of crest settlement with longitudinal cracking and up to 45 cm of displacement on the upstream face. The Shitoki Dam was 60 km from the main shock (PGA of 0.10 g in foundation), but only 5 km from the Mw 6.6 aftershock of April 11th. A PGA of 1.08 g was recorded about 3.2 km south-east of Nishigo Dam.

The most notable exception to the good performance of most dams was Fujinuma Dam, an embankment located in southern Fukushima Prefecture. The failure of the dam shortly after the earthquake resulted in the uncontrolled release of the entire reservoir, which flowed downstream through a small village and killed eight people. The PGA at the site is estimated to have been about 0.3 g based on two nearby recordings, and the dam is reported to have begun breaching within 20 minutes of the earthquake. The photograph, taken about 25 min after the main shock, in Figure 12

Figure 11. View of slumped Hinuma River left levee induced by liquefaction (N36.2861, E140.5238., April 24, 2011).
shows the dam being overtopped. A photograph of the breached section is shown in Figure 13.

Fujinuma Dam had a maximum height of about 18.5 m, a crest length of 133 m, and held a maximum reservoir volume of approximately 1.5 million m³ (~1,200 acre-feet) (Matsumoto, 2011). Construction of the dam began in 1937, was halted during World War II, and was then completed in 1949 (Masumoto, 2011; Towhata et al., 2011; Wikipedia/DAJ, 2011). The dam had a crest width of 6 m and an upstream slope between 2.5H:1V and 2.8H:1V, together with small benches and a relatively steep 1.5H:1V upper slope. The downstream slope was 2.5H:1V, with a small bench at mid-height. It had a steepened downstream toe, perhaps indicative of a rock or gravel toe. Soils exposed in the breach section included a central/upstream embankment section comprised of dark reddish brown clayey gravelly sand, with overlying embankment fills consisting of light brown clayey gravelly sand, and a foundation consisting of an approximately 2-m-thick horizon of black organic silt/clay overlying alluvium, colluvium, and tuff bedrock.

The reservoir for Fujinuma Dam was also retained by a 15 m-high, 60 m-long auxiliary dam, which developed a large slide in its upstream slope (Figure 14). The slide removed almost the entire upstream half of the embankment and had the appearance of a flow slide. The geometry of the slide was documented using LiDAR imagery (Figure 15). There was no obvious distress to the downstream slope of the auxiliary dam, likely composed of materials similar to those of the main dam, but with steeper side slopes at about 1.8H:1V.

There were signs of minor slope distress and failure within the soils.

Figure 12. Overtopping of Fujinuma main dam (N37.3021°, E140.1952°, 3:11 p.m. on March 11, 2011, photo: M.Yoshizawa).

Figure 13. View of breach in Fujinuma main dam from left abutment looking upstream (N37.3021°, E140.1952°, March 22, 2011, photo: M.Yoshizawa).

Figure 14. View of upstream slide in Fujinuma auxiliary dam from left abutment (N37.2995°, E140.1956°, photo: M.Yoshizawa).

Figure 15. Two model surfaces were constructed for the landslide debris and the headscarp of the auxiliary dam failure. Both volumes balanced to approximately 11,000 m of debris.
around the reservoir rim. Some of these areas were earth fills covered with concrete revetments, and other areas appeared to be sliver fills atop native ground.

Our reconnaissance efforts were too limited to allow any definitive conclusions regarding the failure of the Fujinuma main dam or the upstream slide in the auxiliary dam. Nevertheless, several potential failure modes warrant close examination. The main dam’s breach appeared to develop due to overtopping of the lowered crest; the crest was likely lowered due either to an upstream slope failure resulting from strength loss in the fill materials, or a downstream slope failure resulting from sliding on weak foundation soils (which included a thick organic paleo-soil) or sliding through poorly compacted fill. It is also possible that flow through cracks along the crest or internal erosion and seepage were responsible. The auxiliary dam’s upstream slope sliding could have been caused directly by strength loss of material and earthquake shaking, or by rapid drawdown loading resulting from the rapid release of the reservoir water through the main dam breach. Clarification of the most likely causes of each are expected to come from additional detailed studies.

**Ports**

Ports in the disaster area were heavily damaged by both earthquake shaking and the tsunami. The northern ports of Soma, Hachinohe, Sendai, Ishinomaki, and Onahama were among the most heavily damaged. The port at Sendai required more than a week of emergency work before it could handle its first relief ship on March 19th, and another four weeks of work before its first commercial shipment on April 17th. Ports in areas south of the disaster area did not have the same level of tsunami-related damage, but were still significantly affected. The Japan Times reported that approximately 90% of the 29,000 fishing boats in the affected areas were rendered unusable after the earthquake. GEER members inspected the major ports of Ibaraki prefecture, in the southern portion of the affected region, including the ports of Kashima, Oarai, Hitachinaka, and Hitachi, as well as ports north of Sendai, including Kesennuma, Onagawa, and Rikuzentakata, about one month after the earthquake.

Kashima is a medium-size port in the southern part of Ibaraki prefecture, less than 80 km east of Tokyo. The Japanese government began to develop this port in 1963 and completed it in 1973. The port is comprised of a central passage that connects to two public wharves (the North and South Public Wharves) through the North and South Passages. Two breakwaters protect the entrance to the port. In 2006 a container terminal went into operation, upgrading the functions of the port as a distribution terminal. We observed a variety of damage associated with liquefaction around the port area: caisson displacements up to about 15 cm in the central area; wharf backfill settlements up to about 30 cm; negligible wharf displacements; disruption of cranes due to slight misalignment of their rails in the North Wharf; and backfill settlements up to 2 m adjacent to a heavily damaged pile-supported wharf (Figure 16) alongside a caisson wharf with much smaller deformations in the South Wharf area. Cranes at a steel mill were damaged by shaking, and some collapsed. Cargo containers were displaced up to 500 m around the facility by the tsunami, with water marks suggesting inundation heights of at least 60 cm.

Oarai is a medium-size port 120 km northeast of Tokyo comprised of two ferry wharves, an “event” wharf, and a series of fishery wharves. It is home to the biggest inshore fishery operation in Ibaraki prefecture. Damage associated with liquefaction included lateral spreading-induced deformations with a localized collapse along the breakwater, caisson wharf displacements of up to 20 cm, and backfill settlements of up to 50 cm. Tsunami inundation stranded boats across the wharf area.

The small port of Hitachinaka, approximately 110 km northeast of Tokyo, opened in 1998. The outer facilities include the approximately 4.4 km-long East Breakwater, the approximately 0.5 km-long North Breakwater, and the approximately 0.3 km-long Central Breakwater. The North Wharf Public Container Terminal has a 0.350 km-long quay with alongside depth of 15 m to accommodate large vessels. Liquefaction-induced damages included lateral pier movements of up to 30 cm.
cm, with associated disruption and misalignment of crane rails, and backfill settlements of up to 1 m, with associated differential settlements affecting pavements and structures. Water marks indicated tsunami inundation heights of about 1.2 m.

Hitachi is a medium-size port with cargo piers and an oil terminal; opened in 1959, it became an active port in 1967 and has 17 quays with water depth up to 12 m. Liquefaction-induced damage included backfill settlements of up to 1 m, much smaller lateral deformations of the wharves in most areas, and the collapse of the northwest corner of Wharf #4 (Figure 17).

Bridges

Earthquake impacts to bridges were most significant in tsunami-affected areas and ranged from surface scour around bridge piers (Figure 18), to underscour of a box-culvert type structure (Figure 19), to complete removal of large volumes of fill at approach embankments of both highway and railroad structures (Figure 20).

In general, we observed relatively successful performance of bridge foundations despite their being subjected to very large tsunami forces and scour. At a railway bridge at Koizumi O-hash (Figure 18), scour was limited to surficial removal of soil. The beveled caps above the piled foundations appeared to have worked well in limiting scour depth. An adjacent section of the elevated railway structure failed where tsunami waves transported the superstructure several hundred meters upstream and caused failure of the bridge piers. At the same time, it appeared that local structural failure and not scour was the cause of pier failures (Figure 21).

We observed an interesting comparison of the performance of shallow versus deep foundation-based bridge structures in tsunami scour-prone areas at Sodeo-gawa hashi. Pedestrian bridge structures were added on either side of a curved
highway bridge using two different approaches: one was a shallow foundation box-culvert type structure, and the other was a pile foundation-supported pier type structure. A two-opening module of the box-culvert rotated about its long axis and settled about 0.6 m at one end as a result of under-scour (Figure 19). In contrast, while the superstructure of the pedestrian bridge on the other side of the highway bridge was moved upstream by tsunami wave forces, the 0.75 m-diameter single pile foundation elements and pile caps remained perfectly intact, despite scour of up to 0.6 m around the piles.

In some cases, bridge structures were also extensively damaged, with superstructures being moved several hundred meters upstream by tsunami wave forces (Koizumi O-hashii). However, often the entire bridge structure remained intact but inaccessible due to scour effects on the approach structures; this was the situation at both the Nijyu-ichihama hashi railway bridge (Figure 20) and the nearby highway bridge.

In general, limited damage was noted to inland bridges unaffected by the tsunami. We noted minor settlement of abutments fills and bridge piers due to strong ground motion in several locations. At the Naruse River Bridge, there was damage due to liquefaction (river km 30.1), with differential settlements at the abutments and differential settlements of the piers along the main span, warping the bridge deck. Evidence of liquefaction was observed at a few other bridge and viaduct locations, with the most common manifestations being differential settlements at abutments and sinkholes around the piers.

**Lifelines and Transportation**

Water supply was suspended to approximately 2.23 million households in the damaged area; it required over two months to restore water to all habitable service areas. Although much damage was caused by the tsunami, there was widespread disruption from earthquake-induced ground deformation. Liquefaction-induced soil movements damaged water treatment plants in Kashima City (Figure 22) and Wanigawa, as well as the Hebita and Abuta Water Treatment Plants in Ishinomaki, and a wastewater treatment plant in Itako City.

Eighteen wastewater treatment plants were damaged, mainly by the tsunami but also by shaking and liquefaction-induced ground deformations. In tsunami-ravaged areas, it is often difficult to determine whether ground deformations resulted from ground movements or erosion. About 922 km of sewer pipes in the Tohoku region were damaged by tsunami and shaking, and aftershocks increased the damage. Liquefaction-induced differential movements around buried tanks at a water treatment plant in Kashima City (N35.932, E140.628).
induced damage was common in the lower plains (IWA, 2011), and shaking-induced soil settlement and tectonic subsidence of up to several meters has resulted in flood inundation problems.

A system of 2400-mm-diameter transmission pipelines in Miyagi Prefecture had extensive damage, with 52 locations affected. Figure 23 shows pullout and leakage at a slip joint in a pipe with axial slip capacity of 500 mm. The damage is believed to have been caused by seismic-induced slope deformations and other sources of permanent ground movement, with 37 locations of damage at slip joints similar to that in Figure 23.

There were many instances of pipeline damage at bridges, where liquefaction-induced movement of the abutments ruptured pipes. At other locations, pipelines supported on bridges were battered and ruptured by tsunami debris. Electric power losses shut down pump stations. Where diesel engines provided back-up pumping capacity, delays in fuel delivery led to additional shutdowns when fuel ran out. All 47 potable water supply and 128 wastewater pump stations in Sendai were affected in varying degrees by the loss of electricity and fuel.

Figure 23. Pullout and leakage of 2400 mm-diameter water transmission pipeline at slip joint (courtesy of Miyagi Prefectural Government).

Extensive damage to potable water, wastewater, and natural gas pipelines in Urayasu was caused by liquefaction-induced lateral and vertical ground deformation. After the earthquake, 5,100, 33,000, and 7,300 houses lost natural gas, potable water, and wastewater service, respectively. Restoration of the gas, water, and wastewater pipelines was accomplished 18, 25, and 34 days, respectively, after the main shock. Despite the strong shaking and extensive liquefaction, there were no fires following the earthquake in Urayasu.

Surface Fault Rupture from the April 11th Earthquake

A series of moderate to large aftershocks hit the Iwaki region of Fukushima Prefecture one month after the March 11th earthquake (JMA, 2011). The largest of these, the Mw 6.6 Hamadori earthquake, struck on April 11th and was followed a few hours later on April 12th by an Mw 6.0 aftershock; both centered beneath onshore Fukushima Prefecture at shallow crustal depths (about 10 km). There were five other aftershocks from M 5.4-M 5.8 and originating in the shallow crust beneath the Iwaki area between March 19th and April 13th (GCMT, 2011); these generated locally strong ground motions and appear to be associated with displacement on normal faults in the shallow subsurface (JMA, 2011).

Field observations immediately following the April 11th Hamadori quake demonstrated that it generated surface fault rupture over a distance of at least 11 km in southeastern Fukushima Prefecture (Ishiyama et al., 2011a).

Our observations confirm that the April 11th aftershock produced surface fault rupture along the south-central part of the previously mapped Shionohira fault. The fault rupture provides evidence of west-down normal faulting on a fault that strikes approximately N10°W and has a steep dip. The amount of displacement ranged from about 0.8 to 2.3 m, with most of the scar showing about 1.2-1.5 m of vertical deformation. There was about 0.3 m of dextral offset as well. Where present in shallow bedrock, the fault rupture was relatively distinct and linear (Figure 24); however, in unconsolidated alluvium, the rupture is characterized by a fold scarp and hanging-wall cracking. In a few places, buildings and other engineered structures were present in the zone of deformation (Figures 25 and 26), although none collapsed. Limited measurements suggest that some structures were tilted as much as about 3° westward but did not collapse. Most of the structures overlying the surface fault features performed well, but some were severely damaged (Figure 26).

Figure 24. Distinct single scarp in bedrock with net vertical displacement of 0.8-1.5 m typical (2.3 m max) (N36.9735°, E140.6978°; April 23, 2011).
The pattern of surface deformation documented from field reconnaissance is consistent with satellite-based definition of regional crustal block readjustments following the main Mw 9.0 subduction-zone earthquake. The pattern of deformation seen in satellite data and field observations of surface rupture provide evidence of post-mainshock, distributed readjustments of the shallow crust in the hanging wall of the subduction zone, and suggest that these adjustments took place along or near previously mapped upper crustal faults.

Concluding Remarks

Effects from the main shock, foreshocks, and aftershocks of the earthquake provide numerous important opportunities for learning about the seismic performance of a wide range of geotechnical systems and constructed facilities. While this brief report concentrates on observations of damage, it is important to note that many engineered systems performed remarkably well and that documenting their good performance will also provide important lessons for future designs. The earthquake has also illustrated that the resiliency of entire communities and regions subjected to large events is affected by the design of and management practices for geotechnical systems and constructed facilities.

Contributors

The GEER co-team leaders were Ross Boulanger, University of California (UC) Davis, and Nicholas Sitar, UC Berkeley. Members contributing to this report include Pedro Arduino, University of Washington; Scott Ashford, Oregon State University; Jonathan Bray, UC Berkeley; Shideh Dashti, University of Colorado, Boulder; Craig Davis, Los Angeles Dept. of Water and Power; Jennifer Donahue, Geosyntec; David Frost, Georgia Tech; Les Harder, Jr., HDR Engineering, Inc.; Youssef Hashash, University of Illinois at Urbana-Champaign; Robert Kayen, USGS; Keith Kelso, Fugro; Stephen Kramer, University of Washington; Jorge Meneses, Kleinfelder; Thomas O’Rourke, Cornell University; Ellen Rathje, University of Texas, Austin; Kyle Rollins, Brigham Young University; Isabelle Ryder, University of Liverpool; Jonathan Stewart, UCLA; Ashley Streig, Oregon State University; Joe Wartman, University of Washington; and Josh Zupan, UC Berkeley.

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