GEOTECHNICAL ENGINEERING RECONNAISSANCE OF THE 19 SEPTEMBER 2017 Mw 7.1 PUEBLA-MEXICO CITY EARTHQUAKE

Version 2.0

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Geotechnical Engineering Reconnaissance of the 19 September 2017 Mw 7.1 Puebla-Mexico City Earthquake: Version 2.0

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Executive Summary

An intraslab subduction zone earthquake of moment magnitude 7.1 occurred on September 19, 2017 approximately 60 km southwest of Puebla, Mexico, and 120 km southeast of Mexico City, Mexico. The earthquake occurred at a depth of 57 km as a normal faulting mechanism near the point of maximum curvature of the Cocos plate, which is being subducted beneath the North American plate. The event was recorded by over 80 strong ground motion instruments located in Mexico, and produced strong ground motions that exceeded an intensity level VII in Mexico City and Puebla according to the Modified Mercalli Index (MMI). More than 28 million people were within the zone of exposure of this event, including more than 8.6 million residential dwellings, 54,000 schools, and 5,700 hospitals. The event led to the loss of 369 lives and widespread damage to infrastructure, resulting in states of emergency in 320 municipalities across central Mexico.

Immediately following the September 19 event, a joint geotechnical engineering reconnaissance effort was organized between the Universidad Nacional Autónoma de México (UNAM) and the Geotechnical Extreme Events Reconnaissance Association (GEER), which is sponsored by the U.S. National Science Foundation (NSF). Two UNAM-GEER teams were sent to the region to investigate and document the effects from the earthquake: an advance team (September 24-30) and a main team (September 29-October 6). A Version 1 report was distributed immediately upon return of the Advance team; this report summarized preliminary observations from the UNAM-GEER advance team, and was used to inform and assist the UNAM-GEER main team in its investigation. The present Version 2 report is enriched with a wide array of detailed case histories documented by both the advance and main UNAM-GEER teams.

This report offers an introduction to the event and an overview of societal and infrastructure impacts. Importantly, the engineering seismology and recorded ground motions from the event, including comparison of micro-tremor data from the current and past events are also documented. Similar to previous events affecting Mexico City, site effects served an important role in the response of structures. Therefore, in the present report a comparison with past motion recordings (dating back to the 1985 event) is conducted. An important observation from analysis of the ground motions from this event is that soft rock motions contained a much higher frequency content than recorded previously during the 1985 Michoacan earthquake that ravaged Mexico City. As a result, rock ground motions appeared to resonate in transition zone soils (Zone II) and lake zone soils (Zone IIIb) in the western half of Mexico City, causing large horizontal spectral accelerations at periods between 0.8 seconds and 1.5 seconds and resulting in significant damage to many structures between five to eight stories in height. Damage maps across Mexico City and in various heavily impacted regions of the state of Morelos and Puebla reveal patterns of structural damage; which in some cases correlate with foundation damage and ground movement. As would be expected, unreinforced masonry and adobe structures did not perform well in this earthquake, particularly when approaching the epicentral region through the states of Morelos and Puebla. Notably, modern mid- and high-rise buildings in the State of Puebla sustained moderate damage. Observed foundation performance in areas of structural damage varied considerably. Despite the high plasticity lacustrine clays predominant in Mexico City, numerous cases of seismic-induced settlements were observed in the free-field soils around end-bearing pile-supported structures. In addition, several cases of tilted structures were observed. Finally, several instances of slope instability near the southern boundaries of Mexico City and in the state of Morelos, as well as groundwater subsidence cracks near Xochimilco and Colonia del Mar were observed and/or accelerated by the earthquake – in the later cases contributing to the poor performance of water distribution systems.
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1. Introduction

1.1 Overview of Event
An intraslab subduction zone earthquake of moment magnitude 7.1 occurred on September 19, 2017 approximately 60 km southwest of Puebla, Mexico, and 120 km southeast of Mexico City, Mexico. The earthquake occurred at a depth of 57 km as a normal faulting mechanism near the point of maximum curvature of the Cocos plate, which is being subducted beneath the North American plate. The event was recorded by over 80 strong ground motion instruments located in Mexico, and produced strong ground motions that exceeded an intensity level VII in Mexico City and Puebla according to the Modified Mercalli Index (MMI). Due to the nature of the event, only 39 aftershocks were recorded, of much lower intensity.

1.2 Societal Impacts
The Mexican Secretariat of the Interior (Secretaria de Gobernacion, SEGOB) estimates that more than 28 million people were within the zone of exposure of this event, including more than 8.6 million residential dwellings, 54 thousand schools, and 5.7 thousand hospitals (Figure 1.1). This event led to at least 369 confirmed deaths, approximately 60% of them in Mexico City and 20% in the state of Morelos, and severely damaged infrastructure such as collapsed buildings, roads, bridges, and lifelines. According to Moody’s Investors Services preliminary report, earthquake damage could potentially result in as much as 0.3% off Mexico’s gross domestic product for the last 2-quarters of the 2017 fiscal year (CNBC, 2017). AIR worldwide estimates insured losses between 13 billion – 36.7 billion pesos ($725 million - $2 billion US).

Figure 1.1 Level of exposure of the September 19, 2017 event (SEGOB, 2017).
1.3 Impact on Infrastructure

Buildings, bridges, and lifelines throughout Mexico City, Puebla and Morelos were damaged during this event. Because of the high population density and unique soil characteristics of Mexico City, the most pronounced damage to modern buildings were observed within the capitol city, though buildings in the states of Puebla and Morelos were also significantly impacted, particularly older dwellings. Although statistics are not complete, immediately after the event, government reporting to public media indicated that 9,722 buildings were affected by the earthquake, with 1,632 buildings suffering total collapse, 279 schools in need of repairs, and 17 hospitals in poor condition (CNNeSpanol, 2017). In Mexico City, the engineering societies were mobilized with support from the international community and under the auspices of the CICM (Colegio de Ingenieros Civiles de Mexico) to conduct immediate building inspections of buildings in CDMX between September 21 – October 10, 2017. Official reports from CICM (2017) released in late November 2017 indicated that a total of 38 building suffered total collapse, 340 buildings were identified as high risk buildings, and 273 buildings were found to be security uncertain (these values were within the inspection zones of the inspection brigades). Though these values are expected to be higher as evaluations continue, the majority of the buildings documented as collapsed or with significant damage were within mapped transition or lake bed soil zones, thus suggesting that site response and ground motion amplification due to the very soft lacustrine soils in CDMX contributed to the observed patterns of building damage. It is also important to note that the majority of the buildings that suffered collapse or significant damage were in the height range of 4 – 8 stories, aligning with the range of periods of greatest recorded motion amplification at the ground surface.

In the state of Puebla, the Secretaría de la Gobernación declared a state of extraordinary emergency in 112 municipalities in the state, which translates to 51% of the state. Puebla's governor stated a day after the event that 1,700 houses in Puebla were completely destroyed by the earthquake and had to be destroyed (Animal Politico, 2017). The majority of the destroyed residences are located in the cities of Atlixco, Izúcar de Matamoros and Mixteca. The Secretaría de Educación reported that approximately 213 schools (4%) experienced some level of damage. However, at the time of this assessment, only 46% of the schools had been inspected. More than 350 prisoners from the Penitentiaries of Atlixco and Izúcar de Matamoros in the state of Puebla had to be relocated due to the damage sustained to the prisons.

In the state of Morelos, the most devastated municipalities where Jojutla, Cuernavaca, Tecamac, Miacatlan, Yecapixtla, Yautepec, Cuatla, Xochitepec, Axochiapan, Tlayacapan, Oculta, and Zacatepec, as reported by the general Secretary of the Government of Morelos (Animal Político, 2017; Emporis, 2017). In the municipality of Jojutla, at colonia Emiliano Zapata, at least four city blocks of the downtown area were completely destroyed. Approximately 20,000 houses were damaged to some extent in 22 out of 33 municipalities in Morelos, which represents 60% of the state. At least 185 schools experienced damage, and only 50% had been inspected.

Lifelines and critical infrastructure were also damaged during this event and caused substantial disruption within the exposure area. In terms of water utilities, over 6 million users lost potable water services after the earthquake (Excelsior, 2017a). A total of 64 municipalities were directly
affected by this, with 30 municipalities in the state of Puebla and 20 municipalities in the state of Morelos being affected. In CDMX, five regions (predominantly Iztapalapa and Tlalpan) were severely impacted by the loss of water, with a total of more than 2,300 identified leaks in the distribution network. The electrical power grid was also affected; however, the Federal Commission of Power (CFE) reestablished almost the entire electrical infrastructure to all affected customers within the first two weeks following the earthquake. CFE reported that their service to 33% of its customers (representing 5 million people) were affected by the earthquake in the state of Mexico City, Morelos, Puebla, Oaxaca, Guerrero, and Tlaxcala. Within CDMX, a total of 1.8 million customers lost power following the event (Noticieros Televisa, 2017).

According to Secretaría de Comunicaciones y Transportes (SCT), severe highway infrastructure damage was reported on bridges on Oaxtepec-Cuautla highway, Amecameca-La Alborada highway, and Mexico-Acapulco highway (La Razón, 2017). Minor embankment failures in the Puebla-Oaxaca highway near Leon-Huajuapan and Huajuapan-Nochixtlán and Santa Barbara highway in Izúcar de Matamoros were also immediately reported after the event (La Razon, 2017). Six out of the 195 metro stations (Tezonco, Olivos, Nopalera, Zapotitlán, Tlaltenco, and Tláhuac), suffered structural damage and were out of service immediately after the earthquake. They were estimated to be out of service for at least four weeks after the event (Excelsior, 2017b).

1.4 Scope of UNAM-GEER Reconnaissance
Immediately following the September 19 event, a joint geotechnical engineering reconnaissance effort was organized between the Universidad Nacional Autónoma de México (UNAM) and the Geotechnical Extreme Events Reconnaissance Association (GEER), which is sponsored by the U.S. National Science Foundation (NSF). Two UNAM-GEER teams of researchers were sent to the region to investigate and document the effects from the earthquake: an advance team (September 24 to September 30) and a main team (September 29 to October 6). A version 1 report was released on October 16, 2017. The Version 1 summary report provided immediate preliminary observations from the UNAM-GEER advance team, which were used to inform and assist the UNAM-GEER main team in its investigation. The present Version 2 report builds upon the Version 1 report, presenting additional details of specific sites where further reconnaissance was suggested by the advance team, while also presenting new and important observations made by the main UNAM-GEER team.

1.4.1 Survey Regions and Methods
The advance and main UNAM-GEER teams conducted ground level surveys across the zone of exposure as shown in Figure 1.2. More detailed inspections were also conducted by both teams throughout Mexico City (Figure 1.3). Both teams carried advance survey (LiDAR and UAV) and seismic testing (MASW, MAM, HVSR) equipment for use in more detailed characterization of select sites.
Figure 1.2 GPS tracks of the UNAM-GEER Advance and Main teams including Ciudad de Mexico (CDMX) and the states of Puebla and Morelos; and dates of travel.
1.5 Report Organization
This report is organized into seven chapters as follows. The present, Chapter 1, offers an introduction to the event, overview of societal and infrastructure impacts, and layout of the UNAM-GEER teams reconnaissance strategy. Chapter 2 summarizes the engineering seismology and recorded ground motions from this event, including comparison of micro-tremor data from the current and past events. Subsequently, Chapter 3 evaluates site effects particularly in Mexico City considering present and past events (1985 – 2017). Chapter 4 presents specific sites in Mexico City, and the states of Puebla and Morelos where the teams visited and building damage was documented. In this chapter, emphasis is given to mapping damage patterns and specifically correlating foundation to structural damage through the presentation of damage maps at select sites. Slopes that suffered stability issues; as well as rock landslides, were observed during this event, and are documented in Chapter 5. Subsequently, Chapter 6 documents the effects of this event on lifelines and critical infrastructure, with discussion of performance of water, electric and transportation infrastructure in particular. Finally, Chapter 7 summarizes the advanced survey
and seismic testing methods utilized by the teams, including the data gathered and methods used in processing.

References


2. Engineering Seismology and Earthquake Ground Motions

At the intersection of three large tectonic plates, Mexico is one of the world's most seismically active regions experiencing frequent earthquakes and occasional volcanic eruptions. Most of the Mexican landmass is on the westward moving North American plate. The Pacific Ocean floor south of Mexico is being carried northeastward by the underlying Cocos plate. Because the oceanic crust is relatively dense, when the Pacific Ocean floor encounters the lighter continental crust of the Mexican landmass, the ocean floor bends and slides below the North American plate, creating the deep Middle American trench along Mexico's southern coast (Figure 2.1). In addition, as a result of this convergence, the westward moving Mexico landmass is compressed, creating the mountain ranges of southern Mexico and resulting in earthquakes near Mexico's southern coast. The oceanic crust that is pulled downward melts, and the molten material is then forced upward through weaknesses in the overlying continental crust. This process has created a region of volcanoes across south-central Mexico known as the Cordillera Neovolcánica (source: USGS, Seismicity of the Earth, 1900-2010).

Lastly, the area west of the Gulf of California, including Mexico's Baja California Peninsula, is moving northwestward relative to the Pacific plate at about 95 mm per year. In this region, the Pacific and North American plates slide past each other creating strike-slip faulting, the southern extension of California's San Andreas fault. In the past, this relative plate motion pulled Baja California away from the coast forming the Gulf of California and is the cause of earthquakes in the Gulf of California region today (source: USGS-Earthquake Hazards Program)¹.

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¹ https://earthquake.usgs.gov/earthquakes/eventpage/us2000ar20#region-info
2.1 Summary of the September 19, 2017 Mw 7.1 Mainshock
At 1:14:40 p.m. local time (at 18:14:40 GMT), a moment magnitude Mw 7.1 earthquake struck just south of Puebla, Mexico, and 120 km from Mexico City, where almost 9 million people reside. According to the USGS, severe shaking was felt close to the epicenter; Mexico City experienced moderate to strong shaking, enough to cause significant structural damage. Ironically, this earthquake comes on the 32nd anniversary of a deadly M=8.1 Michoacan earthquake that struck 350 km southwest of Mexico City, which resulted in 10,000 lives lost and caused billions of dollars in damage.

The earthquake occurred in the complex region of normal and reverse faults with a regional tectonic mechanism associated with the subduction of the Cocos plate under the North American plate. The focal mechanism was normal faulting. The strike of the rupture plane was approximately 112 degrees and dipped to the north or south at about 42 degrees. The epicenter was located 12 km southeast of the city of Axochiapan in the state of Morelos. The epicenter of the September 19, 2017 intraplate mainshock at a depth of of 57 km (Figure 2.2).

Figure 2.2 Locations of the magnitude 7.1 earthquake of September 19, 2017 (red) and some others of the same type in the region. The “beach balls” illustrate the orientation of the faults (Source: Temblor.net)
2.2 Seismotectonic Environment and Historical Earthquakes in Mexico

Mexico has a long history of intermediate-depth, normal-faulting earthquakes within the Cocos plate. Examples include the 1931 M7.8 earthquake that had catastrophic consequences in Oaxaca (Singh et al. 1985), the 1957 M7.7 earthquake that caused damage in Michoacán and Mexico City (Singh et al. 1996), the 1980 M7.0 earthquake which devastated Huajuapan de León in Oaxaca (Yamamoto et al. 1984), and the M7.0 earthquake of 1999 which resulted in many damaged adobe dwellings in the region close to the epicenter and damaged colonial structures in the city of Puebla (Singh et al. 1999; Yamamoto et al. 2002). Following the devastating M8.1 1985 Michoacán earthquake, Mexico City’s building code was revised, and explicitly incorporated intermediate-depth normal-faulting events when estimating the seismic hazard in Mexico City (Rosenblueth et al. 1989). More recently, several studies have proposed ground motion prediction models specifically aimed at estimating ground motion intensities caused by intermediate-depth normal-faulting events within the Cocos plate in Mexico (García et al. 2004, 2005; Ordaz and Singh 1992; Pacheco and Singh 1995).

Figure 2.2 also shows the epicenters and depths of normal-faulting earthquakes that have occurred in the last 40-50 years. Ruptures of these events occur at depths greater than typical subduction earthquakes (such as the M8.1 1985 Michoacán mainshock), which take place under the coasts of the Mexican Pacific, at the interface between the Cocos and North America tectonic plates (red line, Figure 2.2). Intraplate earthquakes of intermediate depth, by contrast, are produced by extension stresses along the Cocos plate. Studies on intraplate earthquakes in Mexico show that the probability of ground shaking in Mexico City caused by intraplate earthquakes is similar to that caused by subduction earthquakes, such as that of 1985, among others. This implies that the seismic hazard in Mexico’s capital is equally controlled by intraplate earthquakes (such as the 7 and 19 September 2017 events) and by subduction earthquakes that occur under the coast of the Mexican Pacific.

The Mw 7.1 2017 September 19th event was similar in nature to several large-magnitude (Mw>6.5) intermediate-depth (60-100 km) events that have occurred in the central region of Mexico (e.g., Singh et al. 1999, Alcantara et al. 1999). Examples of recent earthquakes with a similar moment magnitude (Mw ~7.0) that occurred near the epicentral area were the 1999 events of June 15 and June 21, with depths ranging from 60 to 90 km (Pestana et al., 1999) which also affected Central Mexico. According to Servicio Sismológico Nacional (SSN) the focus of the June 15, 1999 event was located at 18.40 north latitude and 97.45 west longitude and at a depth of 71 km; and the focus point of the June 21 event was located at 18.34 north latitude and 101.49 west longitude and at depth 50 km.

It should also be noted that on September 7, 2017, just twelve days before the event, an Mw 8.2 earthquake occurred in the Tehuantepec Gulf at 133 km southeast of Pijijiapan in the Chiapas state. The epicenter was at 14.85 north latitude and -94.11 west longitude at a depth of 58 km, according to SNN. The September 7th earthquake caused major damage to houses in the states of Oaxaca and Chiapas. These states have a population of approximately four million people. Several geotechnical issues such as landslides, topographic effects, and site effects were also observed in these areas. Specifically, in Oaxaca 325 historical buildings suffered
severe damage according to the National Institute of Anthropology and History (INAH). Early hypotheses that the September 7th event could have triggered the September 19th event have been disproven by mapping the changes in Coulomb stress triggered by the September 7th event (source: Temblor.net). The location of the epicenters of the two events relative to the most important subduction zone events since the 1950’s is depicted in Figure 2.3.

To demonstrate salient differences between the effects that source mechanisms and seismic energy attenuation of subduction and intraplate events have in the basin response of Mexico City, we next compare ground motions recorded on a hill and a lake zone stations during the 1985 Mw 8.1 Michoacán earthquake and the 2017 Mw 7.1 intraplate event. We specifically compare the strong motion records at stations CUP and SCT (locations depicted in Figure 2.4), stations that were widely used to demonstrate site effects following the 1985 Michoacán earthquake. On the same figure, we also compare the strong motion stations at TACY, which we used as reference site in subsequent site response calculations. We should note here that originally, the code of the stations was CUMV and SCT1 respectively; the upgraded network instruments were renamed to CUP5 and SCT2. Since the instruments are practically co-located, and to avoid unnecessary confusion, we shall heretofore refer to records at these locations as CUP and SCT. The recorded seismograms show that the amplitude of the seismic waves with periods of oscillation less than 2 seconds was much bigger in 2017 than in 1985 (on average about 5 times). The opposite was observed for periods greater than 2 seconds, which at the reference station CUP were shown to be up to one order of magnitude higher in 1985 compared to the corresponding record of 2017.

![Figure 2.3 Modified from Franco et al. (2005) by Temblor.net, this figure shows the location of the two intraplate events that occurred within the subducting Cocos Plate in September, 2017. Additionally, it shows the rupture areas of other large historic earthquakes in the country.](image-url)
Figure 2.4. Comparison of the acceleration response spectra at 5% damping at reference stations CUP and TACY; and a station in the lake zone (SCT) that recorded both the 1985 Mw8.0 and the 2017 Mw 7.1 mainshocks; (left) Thickness of the sedimentary basin underlying Mexico City. The 2017 Mw 7.1 19 September epicenter is shown on the top left insert. The region between the blue and red contours represent the transition zone between the firm ground and the soft soil.

In their preliminary report documenting their 2017 post-event structural damage reconnaissance, Galvis et al (2017) compared the distributions of the number of stories of collapsed structures in the 1985 and 2017 earthquakes. They clearly shown that the 1985 earthquake had a greater impact on taller structures than the 2017 event. This observation is in agreement with the difference in frequency content from events of different magnitudes and with considerably different source-to-site distances. A closer event with a considerably shorter rise time compared to the event of 1985, the 2017 Mw 7.1 mainshock had a richer high-frequency content, which transpired into greater site amplification of ground motions on the surface of the transition zone sediments that were characterized by similar resonant characteristics. The combination of high frequency incident motion, amplified through the Mexico City clay sediments, had a greater impact on shorter period structures located on shallower sediments compared to the affected structures during the long period 1985 earthquake. The frequency content of the incident energy alone (reflected in ground motion records on the hill zone) and the associated resonances triggered within the Mexico City sedimentary basin had strong implications on the structural vulnerability exposed during these earthquakes, as will be shown in greater detail in Chapters 4-6.
2.3 Ground Motion Characteristics of the Mw 7.1 Earthquake Mainshock

Various organizations were operating and maintaining ground motion recording instruments at the time of the September 19th event. Among others, Centro de Instrumentación y Registro Sísmico (CIRES) was operating 53 strong motion stations at the time of the event, Red Acelerográfica de Movimientos Fuertes del Instituto de Ingeniería (IINGEN) at UNAM was operating 18 stations, and the Servicio Sismológico Nacional (SSN) del Instituto de Geofísica (IGEOF) of UNAM was operating 10 stations (Figure 2.5). However, not all of the CIRES, IINGEN, and IGEOF stations were working properly during the time of the earthquake.

![Figure 2.5 Twenty-five IINGEN and IGEOF strong motion records and 49 CIRES records were made available to the UNAM-GEER team. Figure 2.5 shows the seismological stations locations along with the corresponding seismic zonation (adopted from Arroyo et al. 2013), and Figures 2.6 and 2.7 present the time series and response spectra of the records obtained at the said stations.](image)

2.3.1 Mexico City Seismic Geozonation and Soil Conditions

In addition to the location of the stations where ground motions have been made available to the GEER team in the Mexico City, Figure 2.5 depicts the areas where sedimentary deposits govern the seismic hazard of the basin – the seismic geo-zonation of Mexico City: Zone II indicated by yellow, and Zone III by the orange, brown and red shaded areas. Zone III been subdivided into Zones IIIa, IIIb, IIIc and IIId (orange, brown, red and deep red accordingly) to account for the increasing depth of the clay deposits when moving from the hill zones to the center of the old lakes.
Site conditions in Zone II, referred to as the Transition Zone, are characterized by soft clay deposits interbedded by series of thin silty sand and sandy silt layers and lenses, which range in thickness from 0-20 m. These layers are underlain by stiffer sandy silt and silty sand deposits, with interbedded clay layers of varying thickness ranging from a few tens of centimeters to meters.

The typical soil profile for Zone III (Lake Zone), includes a 1-2m thick desiccated clay crust underlain by a soft to very soft clay layer approximately 25 to 35 m thick, with thin interbedded lenses of sandy silts and silty sands. Below the upper soft clay lies a layer of very dense sandy silt (4 to 7 m thick), which rests on a stiff clay deposit 50 to 60 m thick, itself interlaced by very dense sandy silt and silty sands lenses. At larger depths lies a competent layer of very stiff to hard sandy silt and silty clay. For more information, interested readers are referred to Auvinet et al. (2011) and Mayoral et al. (2016).

Figure 2.6 Horizontal components of ground surface recordings in and around Mexico City from the 2017 Mw 7.1 mainshock (x-axis is time in units of seconds; PGA are reported in cm/s²).

(continued)
Figure 2.6 Horizontal components of ground surface recordings in and around Mexico City from the 2017 Mw 7.1 mainshock (x-axis is time in units of seconds; PGA are reported in cm/s²). (continued)...
Figure 2.6 Horizontal components of ground surface recordings in and around Mexico City from the 2017 Mw 7.1 mainshock (x-axis is time in units of seconds; PGA are reported in cm/s²).

(continued)
Figure 2.6 Horizontal components of ground surface recordings in and around Mexico City from the 2017 Mw 7.1 mainshock (x-axis is time in units of seconds; PGA are reported in cm/s²).
Figure 2.7 Elastic 5% damped acceleration response spectra of the 2017 Mw 7.1 mainshock in and around Mexico City (continued)...
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Figure 2.7 Elastic 5% damped acceleration response spectra of the 2017 Mw 7.1 mainshock in and around Mexico City (continued).
Figure 2.7 Elastic 5% damped acceleration response spectra of the 2017 Mw 7.1 mainshock in and around Mexico City.
Figure 2.8 Spatial variability of ground shaking in the vicinity of La Condesa, Mexico City, where extensive damage was caused to 5-10 story buildings by the 2017 Mw 7.1 event (see also Chapter 4). Ground shaking characteristics represented by means of 5% damped elastic response spectra.

Historical macroseismic and recorded evidence suggest that the ground surface response will reveal the dominant role of site effects in determining seismic hazard, particularly pertaining to the shallow slope of the dipping layer (transition zone between the hill and the lake zones) and to the strong impedance contrast between the shallow unconsolidated clay and the deeper sediments. At the same time, one observes considerable spatial variability in the strong motion distribution over distances of tens of meters. Figure 2.8 shows the response spectra of ground motions in the vicinity of the severely stricken neighborhood of La Condesa. In this example,
across a distance of less than 3 km, one can observe a shift of the fundamental period from station ES57: 0.8 s to CI05: 1.0 s to CJ03: 2.0 s to BA49: 2.5 s. The complex geometry of the basin edge, with spatially varying basement slope across short distances, is likely responsible for the spatially varying ground motion characteristics in the outskirts of the lake zone.

Preliminary evaluation of the ground motions records available to date indicate that response spectral ordinates did not exceed those of Appendix A of the 2004 code (Ordaz et al 2003) or those of the new code that was about to be published. Similarly, preliminary evaluation of some of the post-1985 structures that collapsed suggest that these structures did not comply with one or more of the requirements of the building code. Lastly, the large majority of the buildings that collapsed had one or more of the following characteristics: (1) being older pre-1985 non-ductile reinforced concrete structures; (2) having a structural lateral resisting system consisting of flat-slabs supported by reinforced concrete columns; and (3) having a soft story (Galvis et al., 2017). Also very common (in 41% of the collapsed buildings) were buildings located in block corners where effects of torsion are typically more severe (Galvis et al, 2017). All of these characteristics were also commonly observed during the 1985 earthquake. In that sense, the difference in motion characteristics of the 2017 and 1985 events exposed similar poor construction practices, albeit at different locations across the basin, where the incoming ground shaking was amplified; and for different building heights that further resonated with the amplified ground shaking.

2.4 Intensity Maps of the 2017 Mw 7.1 Earthquake Mainshock
Figures 2.9 – 2.11 present Modified Mercalli Intensity (MMI), peak ground acceleration (PGA) and spectral acceleration at 1 s spectral acceleration (SA) Shakemaps, as distributed by the United States Geological Survey (USGS) based on strong motion records from the 2017 Mw 7.1 mainshock. The pattern of strong motion amplification by the basin sediments is clear, although as mentioned above, the high frequency content of this intraslab event resulted in ground motion amplification at the outskirts of the basin as opposed to the center of the lake bed.
Figure 2.9 Modified Mercalli Intensity (MMI) contours of the Mw 7.1 mainshock (source: USGS; overlaid with Google Earth)

Figure 2.10 PGA (%g) contours of the Mw 7.1 mainshock (image overlay—MMI intensity, see Figure 2.9 for intensity contours). (source: USGS; overlaid with Google Earth)
Finally, Table 2.1 lists the ensemble of stations used in this Chapter, along with their epicentral distance, location, and recorded PGA, SA (0.2 sec) and SA (1 sec). On the basis of this preliminary analysis, the correlation of ground motion amplitude in the basin with epicentral distance of intraslab events is shown in Figure 2.12. We here avoid comparison of the empirical data with existing GMPEs until consensus is built upon the correlation that most closely represents the physics of the event.
Figure 2.12 Empirical PGA (upper left), SA(0.2 sec) (upper right) and SA(T = 1 sec) (lower) attenuation as a function of distance from strong motion records in and around Mexico City during the Mw 7.1 19 September 2017 mainshock
Table	  2.1	  Strong	  motion	  stations	  and	  ground	  motion	  characteristics	  of	  the	  2017	  Mw	  7.1	  
mainshock.	  Spectral	  values	  have	  been	  computed	  using	  elastic	  response	  spectra	  at	  5%	  damping	  

	  
	  

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References


3. Site Effects in the Mexico City Basin: 1985 to 2017

Similar to San Francisco, Caracas, Kathmandu and Seattle to name a few, seismic risk in Mexico City is strongly correlated with site effects. The M8.1 09/19/1985 and M7.5 09/21/1985 subduction zone earthquakes, which caused significant damage and the loss of thousands of lives, despite having an epicentral distance of 350km from Mexico City, have become quintessential examples of the role that site effects can play in modifying the amplitude, frequency and duration of ground shaking; and in aggravating the catastrophic consequences of earthquakes. During the September 19, 1985 event for example, peak acceleration in the lake zone was approximately equal to that of the epicentral region (more than 300 km away), casualties were at least 10,000, more than 50,000 people were left homeless, and economic losses were estimated on the order of 4 billion US dollars (Lermo and Chavez-Garcia, 1994).

3.1 1985 Mw 8.1 Michoacan Earthquake: Evidence of 3D site effects in Mexico City

The 1985 consequences on the performance of urban infrastructure prompted a wealth of research on seismology, earthquake engineering, soil mechanics and structural dynamics. The severe damage was predominantly attributed to local site effects, namely to the amplification and duration elongation of seismic waves trapped in the extremely soft shallow lake sediments. Various authors studied the phenomenon (e.g., Seed et al., 1988; Bard et al., 1988) and attributed the extensive structural damage to a double resonance of building and lake sediments in the period range 2.0 to 3.5 s. Results from 65 research projects financed by the Mexican and United States governments were published in three special editions of the Earthquake Spectra journal in 1988 and 1989, while additional results of other, more isolated, efforts continue to be published in scientific journals. Selective results from these projects were incorporated into the new version of the building code for Mexico City, issued in 1987 (NTCS-87, 1987).

The amplitude and duration of ground motion could not have been explained, however, without accounting: (i) for regional amplification by a deeper basin with resonance period also around 3 sec (Bard et al., 1988); or (ii) for the effect of small scale irregularities of the superficial soft layer (Campillo et al., 1989). More recently, Cruz et al. (2016) revisited the effects of the deeper basin and concluded that the duration and ground motion amplification observed in 1985 within the transition zone was predominantly the result of 3D regional site effects from the deeper geological structures, and less the result of local site effects. Cruz et al. (2016) used large scale ground motion simulations to draw the above conclusions but their results have been contested due to the oversimplified shallow clay layers included in their simulations that may have led to erroneous results. The subject of the relative role of the shallow vs. deeper basin effects in the extraordinary ground motion amplification factors observed in Mexico City remains open. What became clear, nonetheless, was that the complete description of the causes of the 1985 disaster, required simultaneous consideration of source and path effects in addition to the local site effects (Campillo et al., 1989).

More specifically, taking into account the characteristics of the teleseismic records, Singh et al. (1989) concluded that the earthquake source made an anomalous additional contribution to the ground motion around the period of 3 sec. The reverberating pulses of period between 2 and 4
sec discussed by Singh and his colleagues can be seen on the displacement records obtained in the region of Mexico City: while the horizontal components of the displacement were strongly dependent on the characteristics of the soil beneath the stations, the vertical displacement records showed nearly identical waveforms and amplitudes at all stations, regardless of their strongly variable local site conditions. In Figure 3.1 for example, stations TACY and CUIP\(^1\) are located on the hill region, where the soil consists of compact sedimentary layers and basaltic lava flows, respectively. Stations SCT and CDAF on the other hand are in the lake region covered by the soft lacustrine clay deposits. Station SXVI is located in the transition zone between the above two regions. Clearly the 3 sec reverberations superimposed with a long-period pulse that was observed at all these sites was not due to local site conditions but represent the characteristics of waves incident from the depth.

![Figure 3.1](image_url)  
*Figure 3.1 (left) Examples of vertical displacement records obtained at different sites in Mexico City (after Mena et al., 1986); station locations and geotechnical zones are shown on the right: Green denotes the Hill Zone (or Zone I); Yellow the Transition Zone (or Zone II); and orange, brown, red and maroon the Lake Zone, which is further subdivided into four Zones, Illa - IllId)*

Using 2D simulations, Campillo et al. (1989) showed that most likely, a 3 sec period perturbation was emitted during the first stage of the subduction zone rupture and propagated as guided S waves (also known as Lg waves) in the crust. These Lg waves are the most efficient mode of propagation of seismic energy for the period range (3 sec) and distance range (350 km) relevant to the destruction in Mexico City, and results in a significant increase of the duration of the signal, which explained the fact that the duration in the hill zone, outside the basin, was longer than the duration observed on the teleseismic or near source records.

This Lg wave train, however, had an amplitude that was not sufficient to represent a real danger for the city. In an earlier study, Campillio et al. (1988) had showed that the 2D response of a sedimentary basin to Lg waves is very similar to its response to vertically incident plane waves. That same year, Bard et al. (1988) showed that the deep Mexico City basin, comprised of the so-called pre-Chichinautzin sediments, is responsible for an amplification factor due to 2D basin effects between 3 and 7 also in the period range around 3 sec. Lastly, the heavily damaged area
was located on the lake zone, where the soil consists of lacustrine deposits with very low S-wave velocity, and fundamental period also around 3 sec. Using realistic shear wave velocity and damping in a 1D wave propagation model, Bard et al. (1988) estimated the amplification in the vicinity of the first mode for the lacustrine sediments on the order of 20. Combining deep and shallow peak amplification factors, the Fourier amplitude spectrum of the ground surface motion in the 3 sec period range was approximately two orders of magnitude larger than the amplitude of the crustal incident motion in that same range, which already contained a strong 3 sec radiated pulse.

These large-scale effects were sufficient to explain the heavy damage in the lake zone (Sanchez-Sesma et al., 1988), but could not reproduce the extreme spatial variability of the damage (Hall and Beck, 1986) and of the records. In the presence of very soft surficial layer as in Mexico City, the effect of small scale (hundreds of meters) lateral heterogeneity can be important. This type of effect is not usually studied because the details of the structure are not sufficiently known to follow a deterministic approach. Using idealized stochastic models of spatially varying properties, Campillo et al. (1989) showed that small, smooth variations of the surficial structure can produce striking effects, such as the spectral amplitude increase of one order of magnitude with respect to a homogeneous flat clay layer even for periods of several seconds, over distances as small as a few hundred meters apart (see also Seed et al., 1988 and Romo et al., 1988). This strong spatial variability attributed to lateral variations in the depth of boundary between alluvium and deeper deposits that cannot be captured by simplified 1D clay layer models, could explain in part the irregular distribution of damage that was induced by the destructive sequence of strong motion events in Mexico City.

### 3.2 Damage Distribution in Mexico City during the 2017 Mw 7.1 September 19 Event

As mentioned in Chapter 2, the Mw 7.1 September 19, 2017 event was triggered by an intraplate rupture at approximately half the epicentral distance of the 1985 event relative to Mexico City. The shorter rise time of the crustal source and smaller epicentral distance resulted in ground shaking in the hill zone that was considerably richer in high-frequencies than the 1985 event. In turn, these ground motion characteristics transpired into more significant site amplification of ground motions on the surface of the shallower sandy silts and silty sands layers of the transition and outer lake zones; and consequently, resulted in more significant impact to shorter period structures than those affected during the 1985 earthquake.

A detailed and extensive description of the structural damage distribution in the valley can be found in Chapter 4. However, it is important to note here that site effects must have played an important role in determining the damaged zones: importantly, the majority of severely damaged structures in Mexico City were concentrated within a 7 x 20 km² area in the west and southwest Transition Zone (II, in yellow) and Lake zones IIIa and IIIb (orange and brown, accordingly). While Zones II, IIIa and IIIb contained 85% of the high risk and 95% of the collapsed buildings (source: CICM 2017), it is important to note however that the majority of these structures (~90%, Galvis et al., 2017) were constructed prior to 1985 but lacked ample inspection/retrofit. Thus, no clear conclusion could be drawn on the relationship between building height and site response, because parameters such as construction quality, design code, material degradation from
continuous seismic activity, and building to building interaction were factors we could not control for in aggregate. Detailed damage maps of severely damaged neighborhoods and their correlation with site response as recorded by strong motion stations or measured by non-invasive techniques by the GEER team; and well as more details on building damage statistics organized by (CICM, 2017) can be found in Chapter 4 of this report.

3.3 Spatial Variability of Strong Motion Records during the 2017 Mw 7.1 September 19 Event

Similar to the 1985 Michoácán Earthquake, strong motion records and damage distribution patterns reveal strong spatial variability of ground shaking during the 2017 Mw 7.1 mainshock. The effects of the spatial variability were, as expected, most pronounced in the Transition and outer Lake Zones (IIIa and IIIb), which are characterized by variations in the depth of boundary between alluvium and deeper deposits that cannot be captured by simplified 1D clay layer models. In the next figures, we demonstrate the spatial variability of ground motion in the vicinity of La Condesa neighborhood, which was perhaps the most severely hit area in central Mexico City (Figure 3.2).

![Figure 3.2 Map of Mexico City valley indicating the zone around La Condesa neighborhood, where the ground motion spatial variability of the Mw7.1 mainshock is explored](image)

Figures 3.3a-c plot the ground motion records in the vicinity of La Condesa, filtered each in a different frequency band: in Figure 3.3a, ground motions have been bandpass filtered between 1-2Hz (0.5 – 1 s period); in Figure 3.3b, motions have been bandpass filtered between 0.5-1Hz (1 – 2 s period); and in Figure 3.3c, motions have been lowpass filtered between 0-0.5 Hz (period larger than 2s). Filtering was performed using Butterworth zero phase IIR filter of fourth order.
The purpose of this procedure is to reveal how different frequency characteristics are amplified in intensity and duration as they travel through the various geotechnical zones of the basin.

Figure 3.3a Ground motion recordings in the vicinity of the La Condesa neighborhood, filtered in three frequency bands (a: 0 - 0.5 Hz; b: 0.5 - 1 Hz; c: 1 – 2 Hz) representative of the predominant periods in the Transition Zone II (yellow) and outer Lake Zones IIIa, IIIb and IIIc (orange, brown and red correspondingly). Vertical axis of filtered seismograms denotes acceleration amplitude, [-200, 200] cm/sec²; for each station, top time series corresponds to NS and bottom to EW direction. (T = 0.5-1.0s)
Figure 3.3b Ground motion recordings in the vicinity of the La Condesa neighborhood, filtered in three frequency bands (a: 0 - 0.5 Hz; b: 0.5 - 1 Hz; c: 1 – 2 Hz) representative of the predominant periods in the Transition Zone II (yellow) and outer Lake Zones IIIa, IIIb and IIIc (orange, brown and red correspondingly). Vertical axis of filtered seismograms denotes acceleration amplitude, [-200, 200] cm/sec²; for each station, top time series corresponds to NS and bottom to EW direction. \( T = 1.0 - 2.0 s \)
Figure 3.3d Ground motion recordings in the vicinity of the La Condesa neighborhood, filtered in three frequency bands (a: 0 - 0.5 Hz; b: 0.5 - 1 Hz; c: 1 – 2 Hz) representative of the predominant periods in the Transition Zone II (yellow) and outer Lake Zones IIIa, IIIb and IIIc (orange, brown and red correspondingly). Vertical axis of filtered seismograms denotes acceleration amplitude, [-200, 200] cm/sec²; for each station, top time series corresponds to NS and bottom to EW direction. (T > 2.0s)

Although subsequent sections will explore in greater detail the frequency characteristics of the Transition and outer Lake Zones, this simple exercise is intriguing. Figure 3.3a, which depicts the ground motions with the highest frequency content of the three subfigures, shows that the reference station (TACY in the Hill Zone) was particularly rich in high frequencies. This is expected, given the source mechanism (intrplate crustal earthquake) described in Chapter 2. As the incident ground motion in this frequency range [1 – 2 Hz] propagated through the sediments, one observes systematic patterns of the effects that these specific frequency components experience, namely, all records showed amplification compared to the record of TACY, with the maximum effects observed at the stations CI05, CJ03 and SCT2 in the Transition Zone. However, the record at station CO56, only a few hundred meters away from CI05 and in the same geoseismic zone, showed very different amplification pattern – in amplitude and duration. Perhaps this is an example supporting the hypothesis of Campillo et al. (1989) that lateral variations of material properties may affect the variability of ground motion due to the very low
shear wave velocity of the clay deposit, but this local irregularity in the pattern requires further study.

Figure 3.3b shows ground motions filtered between 1-2 s period. Despite the very low content of rock outcrop motion in these frequencies, the frequency characteristics are strongly amplified in Zone IIIb, beyond which the amplification and duration decreases. Again, station CO56 recorded an amplification pattern very similar to station BA49 in Zone IIIc, an anomaly that may be the result of 3D effects that cannot be explained by 1D wave propagation.

Finally, Figure 3.3c shows ground motions filtered in the 0-0.5 Hz range, that is, ground motions with only long period (T > 2s) waveform components. In this case, the rock outcrop motion contains very little energy at long period waves (again, attributed to the source effects). However, when filtered through the basin layers, these weak long period components are dramatically aggravated, in amplitude and duration: station ES57 in the Transition zone shows the least amplification (most likely because the shallow sediments in that zone have fundamental periods shorter than 2s), whereas all other stations located in the Lake Zone (IIIa, b and c) show qualitatively similar amplification, although with large variability even for stations that are less than 1km apart (for example, CI05 and CO56).

The same trends are revealed when we look at the seismogram evolution across the basin, both the broadband and the filtered records (Figure 3.4). Despite the high frequency content of the strong motion records at TACY (that is, the high frequency content of the energy from the source and path), long periods are still the dominant components of the strong motion records in the lake zone. This observation appears to be in contradiction with the documented evidence that the most severely damaged buildings were located in the transition zone, and they had relatively short resonant periods (approximate estimate 0.5-1.5 s – see Chapter 4). An additional observation is that the amplification patterns are far from linear: long period components (T > 1 s) that are amplified at station BA49 in Zone IIIc are strongly deamplified in the adjacent station JA43 located approximately 2km away, in the same geoseismic zone. This is one additional strong piece of evidence of 3D basin effects.

To put it differently, one would expect that taller structures with resonant periods longer than 2 s should have been affected as well, due to amplification patterns that most likely are related to resonant modes of the deeper basin sediments. It appears therefore that the Mw 7.1 mainshock exposed the vulnerabilities of the pre-1985 building code in the transition zone by inducing double resonance – of the soil column, and of the building fundamental modes. Compared to the systematic failure of 15-20 story buildings in 1985, one could argue that the 2017 event served as a validation of the 1987 building code revision in terms of the performance of modern high-rise buildings.
Figure 3.4(a) Strong motion stations across the basin used to demonstrate the amplification patterns of short and long period components
Figure 3.4(b) Broadband and filtered records across the basin stations shown in part a. From left to right in period [T sec]: Broadband, $T > 2$ s, $1 < T < 2$ sec, and $0.5 < T < 1$ sec
3.4 Site Amplification in Mexico City Basin during the Mw 7.1 2017 Mainshock

Quantification of ground motion site amplification is a key component in mapping seismic hazard in urban areas (e.g., Frankel et al., 2000), and has been traditionally expressed by means of spectral ratios, namely ratios of the Fourier amplitude spectra of a soil-site record to that of a nearby rock-site record (Borcherdt, 1970). Despite the fact that: (i) direct S-wave recordings often consist of a limited data set when compared to coda-wave observations due to the saturation of micro-earthquake observation networks at the time of first ground motion arrivals (Phillips and Aki, 1986; Chin and Aki, 1991; Su and Aki, 1990); and (ii) many of the stations in seismogram networks consist of single-component high-gain vertical sensors, which are not optimally oriented to record direct S-waves, advancements in the instrumentation technology and acquisition of high-quality data from new events in the recent years have rendered S-wave spectral ratios widely employed for the estimation of site amplification spectra (Hartzell, 1992; Field and Jacob, 1995; Margheriti et al., 1994; Hartzell et al., 1996; Su et al., 1996; Field, 1996). The spectral ratio method, however, depends on the availability of an appropriate reference site, namely a site with negligible site response. The fact that the existence of such a site is not always available has led to the development of alternate techniques called non-reference site methods, which have been employed for site response studies in the absence of reference site recordings. One of these methods to estimate site response uses the spectral ratio between the horizontal and vertical (H/V) spectra of the S-wave window for each site (Lermo and Chávez-García, 1993).

In what follows, amplification observed in the basin using reference-site and single-station site amplification techniques, first for the Mw 7.1 2017 mainshock, and successively for three additional intraslab earthquakes at a similar epicentral distance is quantified. First a brief description of the alternative techniques employed is offered, namely horizontal-to-vertical site response estimates on ground surface, and surface-to-rock outcrop transfer functions. The selection of the appropriate reference station is also discussed.

To compute the strong motion site amplification, the horizontal motion is represented following Tumarkin and Archuleta (1994) by treating the acceleration time histories as two-dimensional signals of a complex time-series as follows:

$$a_v(t) = a_{NS}(t) + i a_{EW}(t)$$

where \(a_{NS}(t), a_{EW}(t)\) represent the North-South (NS) and East-West (EW) horizontal components of the accelerogram. The amplitude spectrum of the complex time series \(a_v(t)\) provides the total amplitude of horizontal motion at a given frequency, while preserving the phase between components. This involves a single fast-Fourier transform (FFT) and eliminates the need for rotation of the components (Shoja-Taheri and Bolt, 1977), and produces results similar to standard averaging methods (Steidl et al., 1995). Both strong motion H/V ratios and reference station amplification factors were smoothed using the Konno-Ohmachi (Konno and Omachi, 1998) logarithmic filter with smoothing parameter \(b = 55\), to preserve the information of high frequency amplification components.
3.4.1 Reference Station Site Amplification: Surface to Rock Outcrop Ratio (SROR)

The frequency content of a seismogram may be evaluated by convolving source, path and site effects, and the instrument response (Borcherdt, 1970; Borcherdt and Gibbs, 1976); therefore, the Fourier spectral ratio of the acceleration at a station \((j)\) to the reference station \((k)\), referred heretofore as site amplification factor, may be simplified as follows:

\[
\frac{A_j(f)}{A_k(f)} = \frac{S_i(f) \cdot P_{ij}(f) \cdot G_j(f) \cdot I_j(f)}{S_k(f) \cdot P_{ik}(f) \cdot G_k(f) \cdot I_k(f)} = G_j(f)
\]

(2)

where \(S_i(f)\) is the source term of the \(i^{th}\) event, \(P_{ij}(f)\) is the path term between the \(j^{th}\) station and \(i^{th}\) event, \(G_j(f)\) is the site term for the \(j^{th}\) station, and \(I_j(f)\) is the instrument response term for the \(j^{th}\) station. The formulation of the above equation is conditioned on the following assumptions: (i) the spectral acceleration content of a single event will correspond to the same source term \(S_i(f)\) for both the site and reference site stations, provided that they are located at the same azimuth with respect to the source; (ii) the instrument response is removed from the data prior to the estimation of site amplification; and (iii) the separation distance between the site and reference site stations is much smaller than the hypocentral distance from the source, resulting in practically common path terms. A number of variations of the reference-site approach are described by Field and Jacob (1995), and the approach has been widely used by Boatwright et al. (1991a & b); Borcherdt and Glassmoyer (1994); Harmsen (1997); Hartzell et al. (2000); Borcherdt (2002) and Asimaki et al. (2007) to name a few. One of the challenges associated with the method is the selection of an appropriate reference site (Steidl et al., 1997). For the analysis of all the strong motions in the remainder of the chapter, we used station TACY in the hill zone on the west side of the Mexico City valley. We briefly defend this selection in the next section.

3.4.2 Single Station Site Amplification: Horizontal-to-Vertical Spectral Ratio (HVSR)

This alternative method for evaluating site amplification effects does not require the presence of a reference site (Field and Jacob, 1995). The HVSR method was originally developed to interpret site amplifications from microtremor measurements (H/V method) by Nakamura (1989). The HVSR method was introduced by Lermo and Chávez-García (1993) to analyze the 1985 Mexico City earthquake strong motions, following the work done by Langston (1979), who studied the upper mantle and the crust from teleseismic records assuming that the vertical component is transparent to the site response. Because HVSR can be computed from a single station without the need of a nearby reference site, this method is quite inexpensive. Much of the work has been done trying to compare the H/V spectral ratio estimates with more traditional methods such as spectral ratios or generalized inversions of the S-wave spectra of the horizontal components only. These comparative studies (e.g., Lachet and Bard, 1994; Lachet et al., 1996; Field and Jacob, 1995; Field, 1996; Theodulidis et al., 1996; Bonilla et al., 1997; Dimitriu et al., 1998; Satoh et al., 2001b; Tsuboi et al., 2001; Asimaki et al., 2007) show that estimates of the frequency of the predominant peak are similar to those obtained with traditional spectral ratios; however, the absolute level of the site amplification does not correlate with the amplification obtained from traditional methods. An additional drawback of the method is associated with the fact that H/V ratios are strongly affected by scattered phases and surface waves propagating within the shallow deposits, which contribute to the spectral amplitudes of both the horizontal and vertical.
components (e.g., Riepl et al., 1998) contrary to the case of vertically propagating body waves. In cases where a shallow soft soil layer overlays a stiffer underlying deposit with a high impedance contrast, and the propagation of seismic waves through the shallow layer can be approximated by 1D conditions, HVSR provides remarkably accurate estimates of the first modal frequency of the shallow layer.

It should be noted here that both the advanced and the main UNAM-GEER teams performed measurements of microtremor H/V ratios and more details are provided in Chapter 7. These measurements are compared to the estimated fundamental frequency of the strong motion sites by the HVSRs in the next section.

3.4.3 Single vs. Reference Station Site Amplification Estimates

To compute the reference station amplification factors in the basin, the first step is to select a reference site. Figure 3.5 shows the horizontal-to-vertical spectral ratio from the Mw 7.1 mainshock at five candidate strong motion stations in the Hill Zone surrounding the basin. The best candidate will be a station with an amplification spectrum as flat as possible, with no distinct peaks corresponding to site response. The station that systematically satisfied the above criteria for the 2017 event as well as older crustal and subduction zone earthquakes that will be studied next, was TACY. This was thus selected as the de-facto reference station, and unless otherwise stated, all amplification factors presented in the rest of this Chapter have been computed with respect to TACY.

![Figure 3.5 Horizontal-to-vertical strong motion spectral ratios (HVSRs) of candidate reference stations in the hill zone of the Mexico City basin; ratios are shown for the 2017 mainshock](image)

Figures 3.6-3.8 compare the reference station to the single station spectral ratio estimates of site amplification, for the most significantly impacted neighborhoods of central Mexico City. The estimated fundamental mode of HVSR remarkably captures the fundamental frequency of the strong motion sites estimated from the reference station methodology, with the exception of the
strong motion station ES57. This is understandable since ES57 lies in the transition zone, where 3D effects likely dominate the response and are poorly captured by HVSR.

While the fundamental period of the sediments in the lake zones IIIa and IIIb estimated from the mainshock strong motion data is in very good agreement with the Arroyo et al. (2013) map of predominant periods in the Mexico City Basin (Figure 3.9), strong variability in the absolute value of amplification is observed over a distance less than 1km and in the same geotechnical zone (IIIb), peak amplification changes from 0.72Hz = 1.38 s at station CI05 to 0.45Hz = 2.22 s at station CO56. More generally, the site amplification at 7 stations in the 2 km x 3 km area shown in Figures 3.6-3.8 indicates that the site period with ranges from 0.47-0.74 Hz (40% difference) and peak amplification which ranges from 7 to 56 (almost one order of magnitude). These effects cannot be fully explained by the mechanics of 1D response of a soft layer overlaying a stiff substratum.

*Figure 3.6 Insert indicates area of concentrated structural damage induced by the Mw 7.1 earthquake, where amplification factors and fundamental frequency are compared in Figures 3.7 and 3.8*
Figure 3.7 Reference station spectral amplification ratio (SROS) vs. single station site amplification (HVSR) obtained in the broader vicinity of La Condesa neighborhood, using the 2017 Mw7.1 mainshock strong motion records
Figure 3.8 Spectral amplification ratios at several stations located in a 6km² area in the lake zones IIIa – IIIc: CO56 and PCJR are 180m apart, and although the first mode at the two sites is very similar (both amplification ratios peak at 0.47 Hz), the maximum amplification at PCJR was 4 times larger at approximately the same peak frequency.

Figure 3.9 Interpolated map of predominant periods in 2010 (contour period values in seconds) proposed by Arroyo et al. (2013) as a modification to the NTC (1987) building code zonation
Arroyo et al. (2013) developed a predominant period map of Mexico City using a total of 37 interface and intra-slab earthquakes ranging from Mw = 5-8 in the Mexican subduction zone recorded between 1985 and 2010 at approximately 100 stations in Mexico City (Figure 3.9). The predominant period was estimated using spectral acceleration amplification factors (SA) (the reference station used was not provided in the publication) and analyzing the dependence of the first site response mode over 25 years. A number of open questions that could be further investigated include: (a) using response spectra instead of Fourier spectra in computing amplification ratios may lead to erroneous estimates of the predominant period due to filtering effects; and (b) all of the observed changes are attributed to the evolution of the elastic clay properties due to subsidence, despite the fact that the analysis used strong motion records and thus observed changes could be also the result of nonlinear site response.

Despite the assumptions of the Arroyo et al. (2013) study, particularly as they pertain to the proposed evolution of dynamic soil properties due to subsidence (their 2010 zonation that was based on interpolated data, as opposed to extrapolated data); their results compare very well on average with the distribution of the empirical fundamental modal periods estimated using strong motion records, and the H/V and surface wave testing results obtained by the advance and main UNAM-GEER teams. The field measurements and empirical estimations of fundamental mode are compared to the Arroyo et al. (2013) map in Figures 3.10a-b.

### 3.5 Site Amplification in Mexico City Basin from Subduction and Intraslab Events (1985-2017)

In this section, the hypothesis by Arroyo et al. (2013) that the empirical amplification factors indicate systematic trend of resonant period reduction with time is evaluated. Arroyo et al. successively use this trend to extrapolate predominant period maps in the basin up to year 2050. A conjecture is that the trend may also reveal dependency of the dominant period with strong motion intensity and/or magnitude and distance of each event. It is important to assess the extent to which this variation is attributed to nonlinear site effects as it has yet to be quantified and remains a topic of research. The locations of the stations that studied herein and the epicenters of the intraslab earthquakes are shown in Figure 3.11. The events span a period from 2000 to 2017.

Next, Figure 3.12 shows the empirical amplification factors at the same stations of Figures 3.6 and 3.7, on the west side of the transition and outer lake zones. Station ES57 in the former shows no site period change for the last 17 years. Stations CI05 and CO56 in the lake IIIa zone, and less than 1 km apart shows no change in the site period, but significant change in the amplitude, an effect that could be attributed to changes in the viscosity (damping) rather than the stiffness of the soil (since the latter would also cause a shift in the resonant frequency, in addition to the amplitude). Approximately 1 km south and still in zone IIIa, station CJ03 experienced a 50% reduction in site period over 17 years; and lastly, 1km east, in zone IIIb, station BA49 experienced site period reduction 60%. The enormous variability of the site period time dependency is also indicative of variability in the hydrological setting, pumping well layout, rate of pumping and distribution of permeable sand lenses across the basin, which likely control the consolidation and secondary compression of the Mexico City clays.
Figure 3.10 (a) Empirically estimated site amplification factors compared to the predominant period zonation proposed by Arroyo et al. (2013) to a) the field H/V and surface wave measurements conducted by the advance and main UNAM-GEER teams and b) empirical SROR ratios.
Finally, Figures 3.12 and 3.13 show comparison of spectral amplification ratios (relative to TACY) and HVSRs (single station estimates) at 32 stations in the basin, each for the 4 crustal earthquakes of Figure 3.11, including the 2017 Mw 7.1 Puebla-Mexico City mainshock. Overall, the HVSRs and empirical amplification factors are consistent in terms of fundamental frequency estimates of the sites. The amplitude of empirical surface to rock outcrop transfer functions (SROR) varies strongly between adjacent sites; and the 2000 event in some stations has substantially lower resonant frequency compared to the post-2010 events (see for example stations CJ03 and BA49 in Figure 3.12).

At first glance, this phenomenon could be associated with sediment consolidation over the past 20 years, whereas the decrease in impedance contrast between the soft clay and underlying stiff sediments could explain, in part, the associated decrease in site response amplitude. It should be pointed out, however, that these results indicate that the shift in predominant period between 2000 and 2010 occurred only locally, suggesting that if consolidation is partially responsible for the shift, it is clearly not one-dimensional as Arroyo et al. (2013) suggested. Also, one should apply caution when interpreting modal shifts from strong motion records, because effects such as nonlinear response also manifest as shifts, sometimes irreversible, in predominant period.
Figure 3.11 (a) Strong motion stations used in the following analyses, and (b) epicenters of the four intraslab events (2017 included) used in the analysis
Figure 3.12 Empirical amplification factor at strong motion stations on the west side of the valley for four intraslab earthquakes
Figure 3.13 Horizontal-to-vertical spectral ratios at 32 stations in Mexico City, using 4 crustal events of similar epicentral distance shown in Figure 3.11
Figure 3.14 Surface to Rock Outcrop (TACY) amplification factors at 32 stations in Mexico City, using 4 crustal events of similar epicentral distance shown in Figure 3.11
Figure 3.16 depicts the location of the stations where Fourier spectral amplification ratios reveal systematic shift between 2000 and 2017, with most of the changes occurring between 2000 and 2010. For the remainder of the stations shown in Figure 3.16, in the transition and the lake zones, these analyses revealed almost no change in the resonant period of the site. The station location is overlain with an ESA satellite image of ground deformation in Mexico City, which was presented at the InSARap Workshop at ESA’s ESRIN Centre for Earth observation in December 2014. InSARap is a project under ESA’s Scientific Exploitation of Operational Missions (SEOM) Programme1. Although there is clearly a correlation between subsidence and material properties change, this is clearly not uniformly reflected in the resonant characteristics as calculated from the strong motion station recordings across the basin.

For the stations depicted in Figure 3.15, we computed the surface to rock outcrop amplification ratios of all recorded available events from 2010 to 2017, and traced changes in the fundamental frequency (location of first SROR peak) and in the amplification amplitude at that frequency. The SRORs at stations with pronounced changes in resonant characteristics are shown in Figure 3.17. Comparing the fundamental frequency (first mode) of these SRORs to changes in the resonant characteristics of the sites estimated by Arroyo et al. (2013), who used response spectral ratios of weak ground motion records and microtremors, we observed systematic differences:

In very few cases the estimation matches the period of the Arroyo et al. study. The source of this discrepancy is not clear, and more detailed analyses are needed to understand the evolution of clay properties with time and translate the phenomenological site period to microzonation maps. Examples of issues worth investigating in future refinements of the time evolution study include:

(i) Using Fourier complex amplitude spectral ratios as opposed to response spectra, to estimate the predominant site period. Response spectra operate as filters of the frequency content, do not preserve the phase of the ground motions, and may therefore lead to erroneous estimates of the predominant site period.

(ii) Understand the implications of using 1D consolidation theory to translate site period into ground deformation and water migration, as opposed to 3D. The latter could be particularly important in the transition zone, where the sediment depth is not constant and the rate of clay thickness increase varies across the valley.

(iii) Investigate the effects of water pumping rate changes when extrapolating the primary consolidation trend between 1985 and 2010 to more recent years. The concern here is that secondary compression may have become the governing mechanism of changes in soil properties reflected in dominant mode of dynamic site response.

1 http://www.esa.int
Figure 3.15 Location of all stations in Mexico City, where we computed the SROR predominant frequency peak from 2000 and 2017.

Figure 3.16 Satellite image of the 2013 ground deformation in Mexico City caused by ground water extraction produced by ESA’s Scientific Exploitation of Operational Missions (SEOM) Program\(^2\); overlain on Google Earth map is the subset of stations depicted in Figure 3.15 where we observed changes in the fundamental frequency over time, by tracing the peak of the empirical SRORs from four seismic events between 2000 and 2017.

\(^2\) http://www.esa.int
Finally, a comparison between strong motion spectral acceleration, amplification ratios and HVSRs at the five stations that the UNAM-GEER team had access to data from the 1985 Michoacan and the 2017 Puebla-Mexico City events are plotted. The stations and analyzed ground motion data are shown in Figure 3.18-20. As shown in Chapter 2 as well as in the foregoing sections of this Chapter, the 1985 subduction zone event clearly had a much longer period content, attributed to the source rupture and Lg path of body waves from the source to the basin, several hundred kilometers away.

Figure 3.17 Surface to rock outcrop amplification ratio for 2000 and 2017 events, which reveal considerable shifts in the predominant site frequency (or period).
From a phenomenological standpoint, SRORs at stations in the lake zone could arguably be associated with nonlinear site response: for example, compare the substantial shifts in the fundamental frequency and amplitude at stations SCT1,2 and SXVI to the minimal shifts at CO47 and AO24 when comparing the 1985 and 2017 strong motion recordings in Figures 3.19-20. In light of the results presented above, however, on the relationship between predominant site period and rapid site consolidation in Mexico City, such conclusions should be also drawn with caution, since with the information in hand, it is difficult to distinguish if the changes are due to nonlinear effects, consolidation, or a combination of the above.

Figure 3.18 Stations utilized for analysis in Figures 3.19 and 3.20. (recordings from the 1985 and 2017 events).
Figure 3.19 Spectral ratios, HVSR and spectral acceleration recorded at stations (co-located or adjacent) as captured by recordings from the 1985 and the 2017 events. (stations SCT and CUP)
Figure 3.20 Spectral ratios, HVSR and spectral acceleration recorded at stations (co-located or adjacent) as captured by recordings from the 1985 and the 2017 events. (stations AU11, CDOA, CDAF, CO47, SXVI, AO24, TACY)
3.6 A Simplified Model for Mexico City Basin Edge Effects and the Need for Future Research

The geotechnical conditions at Mexico City have repeatedly attracted analyses based on the concept that the vertical resonance of the thin, very soft, clay layer at the surface can be decoupled from the two-dimensional (2-D) effects of a much stiffer underlying sedimentary valley. Sáchez-Sesma et al. (1988) and Askar (1989), for example, proposed the simulation of site effects as a convolution between large scale and small scale effects, namely the representation of ground surface motion as the product of the incident motion and two transfer functions calculated independently. Bard and Chávez-García (1988) tested the approach by comparing the effects of a basin of homogeneous sediments convolved to the 1D site response of a soft clay layer; to the response of a complete model where the effects of shallow layer and 2D basement structure are accounted for simultaneously.

Their results showed that there exists significant coupling, even in such extreme conditions as Mexico City. More interestingly they showed that, in addition to the basin edges where the wavefield is dominated by diffracted waves on the edge of the clay deposit that the decoupled model cannot account for, these extremely slow, very dispersive surface waves affect the response event at the center of the basin, far from the valley’s edge. Their analysis suggested that in Mexico City, assuming homogeneous 2D or 1D layered site conditions alone (the latter being the standard practice of geotechnical engineers) may be misleading even at sites in the transition zone where the conditions would be characterized as ideal for 1D site response.

In the remainder of this chapter, a simplified third model proposed herein, is tested. Specifically, a dipping layer of soft clay (a wedge-shaped feature with mixed free-impedance boundary conditions) is compared to the wavefield pattern predicted by the complete 2D model of Bard and Chavez-García (1988). Compared to 1D models, the response of a dipping layer is characterized by two new phenomena: diffraction from the wedge tip and diverging/converging reflection and refraction from its lower boundary (Ishii and Ellis, 1970; Hong and Helmberger, 1977; Pao and Ziegler, 1982; and Ziegler and Pao, 1984). The diffracted field results from the discontinuity of the tip in the geometric elastodynamic field, i.e. when a set of reflection and refraction from the boundaries does not satisfy exactly the boundary conditions. For gently sloped dipping layers, like the basin edge of Mexico City, the extent and intensity of the diffraction field – which is determined by the wedge angle – is small, and thus barely affects the surface response especially for points far from the tip (considering the geometric attenuation of diffracted body waves as well as the small amplitude of diffracted Rayleigh wave). Furthermore, a dipping layer – as opposed to a horizontal layer – is able to change the angle of reflection-refraction as the trapped wave reverberates between the underlying stiff material and free surface. For a dipping layer at a small angle, however, the rate of change of the reflection-refraction angle is very small and thus minimizes the continuous generation of complex wave patterns at each reverberation.

Next the effects of a mild dipping layer similar to that of the Mexico City transition zone on the wave angle of incidence and reverberating wavefield is demonstrated, by performing a simple analysis using the following parameters:
1) Slope of dipping layer: 1% (0.6°) – depth 10m over 1km horizontal distance from the edge;  
2) Impedance contrast: 1:4 (due to computational constraints);  
3) Input: Vertically propagating single plane SV Ricker wave

The results of seismogram synthetics and amplification factors (compared to homogeneous flat half-space) are shown in Figures 3.21 and 3.22. The horizontal seismogram synthetics show the different components of the total wavefield:

- The direct wave that tilts towards the right as the thickness of upper soft layer increases;  
- A sequence of reflected waves originating from the same point (looks like a fan) whose angle of incidence becomes flatter in time and its amplitude decreases due to energy leakage;  
- Rayleigh waves of small amplitude on the top-right part of the figure that are grazing along the surface slowly (small slope of their trace).

The vertical seismogram synthetics on the right are useful to show the parts of the wavefield generated by material and geometric discontinuities that are not present in the 1D layer assumption. The amplitude of vertical motion that shows the negligible effect of dipping is 5% that of the horizontal motion amplitude.

\[ \text{Figure 3.21 Seismogram synthetics of horizontal (left) and vertical (right) acceleration} \]

The effect can also be seen in Figure 3.22, where the dipping layer response is compared to the amplification factor of an equivalent horizontal layer with space-varying depth. The abscissa shows the distance to the wedge tip normalized by the shortest propagating wavelength. At each point, the blue dash curve gives the horizontal acceleration of a horizontal layer over half-space whose thickness is the same as the local thickness of dipping layer. While the dipping layer can amplify the flat ground response by as much as 270%, its effect is essentially the same as that of a horizontal layer. The minor difference happens over a narrow region near the tip where the diffracted field evolves and then diminishes. The curve has a peak at thickness equal to \( \lambda/18 \) (maximum constructive interference between incident and reverberating waves for this
waveform), then falls to a point of discontinuity where the governing wavefield changes, and finally approaches the constant value of 1.6 which corresponds to the doubling of the transmitted wave at the surface, free of interference with the reflected waves:

\[ T = \frac{2Z_2}{Z_1 + Z_2} = \frac{2 \times 4}{1 + 4} = 1.6 \]

Normalized Amplitude @ Surface: \[ \frac{2 \times 1.6}{2} = 1.6 \]

Figure 3.22 (a-left) Horizontal amplification factor for a dipping layer and a horizontal (1D) layer with gradually increasing thickness (vertical axis represents the peak acceleration at each point, normalized by the peak free-field acceleration of a halfspace with flat ground surface); (b-right) Maximum amplitude of synthetic seismograms as a function of distance to the edge of the valley for the decoupled and the coupled 2D solution of Bard. The maximum difference between the two curves is about a factor of 1.7 (Bard and Chavez-Garcia, 1988)

Comparing the results of Figure 3.22a to the simulations of Bard and Chavez-Garcia (1988) (Figure 3.22b), one observes a qualitative similarity between the clay wedge (this study) and the uncoupled solution, which is because of model is missing the coupling with the underlying basin. In other words, a 2D clay layer is not enough to account for the effects of amplification near the basin edge –or in the middle of the basin for that matter.

Concluding remarks: The results from this chapter, more than anything, have highlighted the importance of understanding the 2D/3D heterogeneity of the basin sediments and the coupling between hydrological setting and site response, if we wish to capture the spatial variability of ground motion, the evolution of its properties with time and how this evolution affects the changes in the predominant period of the basin sediments. Further investigation is clearly also needed on the subject of coupling between the shallow clay layers with the deeper sediments, to understand the energy interaction between the deep basin edge and the shallow clay ‘energy trap’ that gives-- and will continue to give rise to fascinating wave propagation phenomena.
References


4. Effects of Event on Buildings

Damage to buildings due to the September 17, 2017 event were significant and extended throughout Mexico City, and the states of Puebla and Morelos. Although statistics are not complete, immediately after the event, government reporting to public media indicate that 9,722 buildings were affected by the earthquake, with 1,632 buildings suffering total collapse, 279 schools in need of repairs, and 17 hospitals in poor condition (CNNespanol, 2017). This chapter focuses on summarizing the effects of this event on buildings in the regions of Mexico City and the southern states of Puebla and Morelos; while documenting several important case study sites.

4.1 Damage Mapping Strategy

Buildings that experienced damage during the September 19 2017 earthquake were surveyed using a combination of UAV, LiDAR and walking (ground level) inspections. For certain sites, mapping of damage was evaluated using the Ground Failure Index presented Table 4.1 (Bray and Stewart, 2000); while assessment of structural damage utilized the indices presented in Table 4.2; (modification adopted by Bray and Stewart, 2000; of work by Coburn and Spence, 1992). Maps were generated via overlay with either low altitude aerial flyover imagery or the most recent satellite imagery available.

| Table 4.1 Ground Failure Index (after Bray and Stewart, 2000) |
|--------------------------|-----------------|-----------------|
| Index | Description | Interpretation |
| GF0 | No Observable Ground Failure | No settlement, tilt, lateral movement, or sediment ejecta |
| GF1 | Minor Ground Failure | Settlement, D < 10 cm; Tilt < 1 degree; no lateral movements |
| GF2 | Moderate Ground Failure | 10 cm < D < 25 cm; Tilt of 1-3 degrees; small lateral movements ( < 10 cm) |
| GF3 | Significant Ground Failure | D > 25 cm; Tilt of > 3 degrees; Lateral movement > 25 cm |

| Table 4.2 Structural damage index (modified from Coburn and Spence, 1992; as used by Bray and Stewart, 2000) |
|--------------------------|-----------------|-----------------|
| Index | Description | Interpretation |
| D0 | No Observable Damage | No cracking, broken glass, etc. |
| D1 | Light damage | Cosmetic cracking, no observable distress to load-bearing structural elements |
| D2 | Moderate Damage | Cracking in load-bearing elements, but no significant displacements across these cracks |
| D3 | Heavy Damage | Cracking in load-bearing elements with significant deformations across the cracks |
| D4 | Partial Collapse | Collapse of a portion of the building in plan view (i.e., a corner or a wing of the building) |
| D5 | Collapse | Collapse of the complete structure or loss of a floor |
In conjunction with the characterization between structural and foundation damage as discussed above, in the following sections, building usage was generalized into the following categories: Commercial, public, hospital, schools, housing (low, mid-, high-rise). Low-rise denoting buildings of 1-2 stories, mid-rise denoting buildings 3-11 stories and high-rise buildings with > 12 stories.

4.2 Ciudad de Mexico (CDMX)

Ciudad de Mexico (CDMX) or Mexico City proper is the capital of Mexico and centrally located in the country with a population of nearly 9 million (more than 21 million in Greater Mexico City) distributed within 16 municipalities. Resting in a valley with a minimum elevation of about 2200 m, it is surrounded by mountains and volcanoes and resides largely on former lake beds. The waters on the surrounding mountain slopes drain towards the basin center. CDMX and Greater Mexico City observed wide spread growth until the 1980s, after which decentralization policies were invoked to help reduce environmental pollution and better utilize the countries land. As a result, the net migration of CDMX from 1995 – 2000 was negative (INEG, 2013) and its greatest population (and construction) growth occurred well before the 1985 earthquake. Despite this, Mexico City is one of the most important economic centers in Latin America, with a GDP of nearly $400 million, it ranks as the wealthiest city in Latin America (PWC, 2009). Consequently, it is well known that CDMX has a long history as a densely populated area.

Buildings in the CDMX include a blend of low, mid, and high-rise structures, with a large portion constructed prior to the mid-1980s. Notably, according to some statistics more than 1400 high rise buildings (>12 stories) exist in Mexico City (Emporis, 2017), including the first constructed high-rise in all of Latin America (Torre Latinoamericana, a 40-story skyscraper, constructed in 1956). Intermediate height buildings, between 5 – 15 stories, and generally with period ranges between 0.5 – 1.5 sec, were severely impacted during the 1985 earthquake with 210 documented building collapses in Mexico City (Meli and Miranda, 1985; Meli and Avila, 1989). Buildings that collapsed during that event were characterized by large lateral flexibility, with their natural periods of vibration close to site periods in the lake bed of Mexico City, where accelerations were amplified to as high as 1g (Meli and Avila, 1989). Tall buildings, with a fundamental period of > 2 sec were less affected by this event; and generally low-rise buildings, with fundamental periods of < 0.5 sec, had adequate strength to resist the lateral earthquake demands in CDMX. Nonetheless, enhancement to building flexibility occurred in many cases due to foundation rocking and the highly deformable soil, which in some cases resulted in structural collapse. The most predominant failures that led to collapsed buildings during this event was columns or flat slabs failing in shear due to inadequate ductility. In addition to the more than 200 collapsed buildings, the main shock of September 19, 1985 resulted in 10,000 lives lost and 50,000 residents left homeless (Esteva, 1989).

The devastating impact of both the M7.9 1957 and particularly the M8.1 1985 Michoacan earthquake triggered major modifications to the Mexico City code. Notably, the advances to the Mexico Federal District building code after the 1985 event explicitly incorporated normal-faulting events to the hazard estimation in Mexico City (Rosenblueth et al., 1989; NTCS-87, 1987). In addition, this event triggered a finer discretization of soil types within the city, and more stringent structural design requirements. Unfortunately, as will be discussed, many existing buildings
constructed prior to 1985 collapsed or were severely damaged during the 2017 event. A preliminary report by Galvis et al. (2017) notes that buildings that collapsed from the 2017 event had one or more of the following characteristics: 1) constructed prior to 1985 of non-ductile reinforced concrete, 2) had a lateral resisting system consisting of flat slab-concrete columns and 3) had a soft story. The UNAM-GEER Advance and Main teams spent several days during their reconnaissance documenting collapsed and other building sites in the CDMX (Figure 4.1). Overall damage statistics and select sites documented by the teams are discussed in the following sections.

4.2.1 Building Damage Statistics
Maps of damage created by civil engineering brigade inspections organized by the Colegio de Ingenieros Civiles de Mexico (CICM, 2017) are overlaid with the geo-zonations described in Chapter 3, in Figures 4.2-4.5. At the time of preparation of this report, these data indicate that 38 buildings were tagged as collapsed (Edificios Colapsados; Figure 4.2), 340 buildings were identified as high risk buildings (riesgo alto; Figure 4.3), and 273 buildings were identified as having uncertain security (seguridad incierta; Figure 4.4). It is important to note that the four zones of inspection were of predominant focus of the CICM brigades (Figure 4.5), thus some
regions where heavy damage was documented by the UNAM-GEER team (such as Xochilmilco and Tlahuac to name a few) are not included in the CICM database. Interested readers are referred to the evolving government database of buildings to-be demolished: Plataforma de la Ciudad de México¹. The totality of severely damaged buildings as included in the CICM database (2017) by geo-zone is summarized in Figure 4.2. These maps and associated tabular data consistently demonstrate that most of the major severe structural damage in Mexico City was located in the west and southwest Transition (Zone II) and Lake zones, IIIa, and IIIb (in contrast to the distribution of collapsed buildings during the 1985 event, which were largely concentrated in the softest lake bed sediments in Zone IIIb/c; Figure 4.6). It is important to note however that the majority of these structures (~90%, Galvis et al., 2017) were constructed prior to 1985, thus it is suspect that many of the severely damaged buildings in the 2017 event may have been affected in the 1985 event, but lacked ample inspection/retrofit. The outer dark green boundaries in these Figures represent stiff/dense Hill Zone (Zone I) sediments and volcanic rocks. The geo-zones were heaviest structural damage was observed exhibit 1D predominant periods ranging from 0.8 to 1.5 s according to the map by Arroyo et al. (2013), which accounts for the changes in predominant periods due to regional subsidence effects (Figure 4.7). It is noted that this map is slightly different from that included in the Mexico City Seismic Design Code, NTCs.

Issues of site response on a regional scale are discussed in greater detail in Chapter 3. However, based on the mapped damage and the field observations from the UNAM-GEER team, the neighborhoods of Mexico City that were most severely impacted by site response and structural damage/collapse from the September 19th event appeared to be Cuauhtemoc, Juarez, La Condesa, Roma, Hipodromo, Hipodromo Condesa, Roma Sur, Roma, Col del Valle NTE, Narvarte Poniente, and Col del Valle Centro.

Table 4.3 Number of severely damaged building cases reported in each geo-seismic zone (CICM, 2017)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Number of buildings</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Collapsed</td>
<td>High Risk</td>
<td>Security Uncertain</td>
</tr>
<tr>
<td>I</td>
<td>2</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>II</td>
<td>9</td>
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<td>36</td>
<td>35</td>
</tr>
<tr>
<td>IIId</td>
<td>None</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>Total</td>
<td>38</td>
<td>340</td>
<td>273</td>
</tr>
</tbody>
</table>

¹ https://www.plataforma.cdmx.gob.mx/sig
Figure 4.2 Collapsed buildings as included in the CICM database as of October 24, 2017 overlaid with geo-zones in GoogleEarth (CICM, 2017). (blue = zone IIId, light green = zone IIIc, yellow = zone IIIb, red = IIIa, orange = zone II, and dark green = zone I).
Figure 4.3 Riesgo alto (high risk buildings) as included in the CICM database as of October 24, 2017 overlaid with geo-zones in GoogleEarth (CICM, 2017).
Figure 4.4 Riesgo alto (high risk buildings) and Seguridad Incierta (security uncertain buildings) as included in the CICM database as of October 24, 2017 overlaid with geo-zones in GoogleEarth (CICM, 2017).
Figure 4.5 Inspection zones of the CICM brigades in white (for context, note that some areas of heavy damage observed by the UNAM-GEER team were not included in the CICM inspection zones).
Figure 4.6 Collapsed buildings in the central region of Mexico City as mapped following the September, 19 1985 event and the September 19, 2017 event (per CICM, 2017 database)
Figure 4.7 Map of predominant periods proposed by Arroyo et al. (2013) as a modification to the NTC
4.2.2 Seismic Testing in CDMX

The UNAM-GEER teams conducted various seismic tests throughout CDMX, including Multi-Channel Analysis of Surface Waves (MASW), Microtremor Array Measurements (MAM), and horizontal-to-vertical spectral ratio (HVSR) testing (see Chapter 7). These tests consistently demonstrate the increasing site period towards the deepest lake zone consistent with previous studies, ranging from 0.9sec in the transition zone soils to greater than 2.0sec in the softest regions of Del Mar (Figure 4.8; Table 4.4). The methodology for performing these tests is presented in Chapter 7 and data from relevant sites summarized in Table 4.4 are utilized on a regional scale to draw inference to site response issues in this chapter.

![Figure 4.8 Overview map of sites within CDMX where HVSR was conducted by the UNAM-GEER teams. See Chapter 3 and 7 for more details.](image)

<table>
<thead>
<tr>
<th>Site Name</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Frequency (Hz)</th>
<th>Period (sec)</th>
</tr>
</thead>
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<td>1.35</td>
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<td>0.92</td>
<td>1.09</td>
</tr>
<tr>
<td>Location</td>
<td>Latitude</td>
<td>Longitude</td>
<td>Min. A</td>
<td>Max. A</td>
</tr>
<tr>
<td>-------------------</td>
<td>------------</td>
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<td>N/A</td>
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<td>2.08</td>
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</tr>
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4.2.3 Condesa Region

The UNAM-GEER team undertook several walking and aerial surveys within the Condesa neighborhood (Figure 4.9). This region is largely in Lake Zone soils (zone IIIa) and includes buildings ranging in state from collapsed to undamaged. Acceleration measurements are also available within this zone, approximately 0.8km northeast of the survey zone (Figure 4.10).
Measurements from CI05 comparable motion demand in both EW and NW shaking direction, and the strongest spectral acceleration of 0.4g in both EW and NS directions at 1.5sec. Importantly however CI05 is located within zone IIIb soils, while the survey zone is within slightly thinner layer of clay soils, within the geo-zone IIIa. It is likely that the slightly thinner clay in the survey zone observed its peak acceleration demands at slightly lower periods. CI05 reported a peak ground acceleration of 0.12g and strong duration of shaking of Td = 49sec. The maximum horizontal acceleration amplification measured at CI05 occurred at T = 1.58sec with an amplitude of 4.2. It is noted that aerial surveys (Figure 4.9) were used to generate ortho-rectified images as well as 3D models for navigating through the region (Figure 4.11-Figure 4.12). Processing of these 3D models is described in Chapter 7.

Figure 4.9 UNAM-GEER team survey areas (white areas show walking surveys; blue boundaries show UAV surveys, overlay includes HVSR and SW measurements) within the Condesa and Hipodromo neighborhoods; with overlaid CICM severely damaged building data (2017). Blue markers identify collapsed buildings from the 1985 event.
Figure 4.10 Accelerations measured at station CI05 within the Condesa neighborhood. (CI05 is approximately 0.8km NE of the center of the survey site (Figure 4.9), however it is in zone IIIb soils.)
4.2.4 La Condesa South
The La Condesa South region included a survey of 56 buildings along Calle Amsterdam and Calle Mexico (Figure 4.13). Buildings within this colonia of Hipódromo consisted of a mix of residential, office and commercial buildings, with the majority of structures of residential occupancy.
Buildings in this survey area ranged in height from 2 – 14 stories; with most constructed of reinforced concrete. The vintage of the buildings appeared to be from pre-1960s to present. Two states of floor or full collapse (D5) were observed within this survey; 3 cases of D3 (significant damage to structural components); and 3 cases of minor damage to structural components (D2); in totality about 14% of the buildings in this survey zone observed moderate-significant structural damage. All of these buildings were between 6-14 stories in height. The majority (56%) of the buildings in this survey region were 3 or less stories in height. The tallest building in this survey region was categorized as a structural damage state D3, with damage to its load carrying structural members evident from the exterior, though these appeared repairable (Figure 4.14). The most significant structural damage included an 8-story building (Avenida Sonora 149) which suffered a complete floor level collapse, likely due to discontinuity in its wall area (Figure 4.16-Figure 4.17). No ground failure was evident at the perimeter of this building. Moreover, ground failure was not substantial in the entirety of this survey area, with all foundations categorized as G0, with the exception of five cases, one denoted as a G2 (e.g., Figure 4.18); and four cases of minor foundation damage (G1). A concentration of ground failure was observed along Calle Amsterdam, with one 9-story building suffering moderate foundation movement-induced tilt (Figure 4.18).
Figure 4.13 Damage survey in La Condesa South (Hipodromo), along Calle Amsterdam and Calle Mexico (19.4130°, -99.1702°) (viewpoints A-E shown in subsequent Figures). Orthomosaic image created with UAV flyover conducted on 9/30/2017. Building footprints are approximate and obtained from Google Street View.
Figure 4.14 14-story building in La Condesa South (Hipodromo) (view A from Figure 4.13). Left: before (google street view) and right: after (note damage to structural walls). (19.4149°, -99.1693°).

Figure 4.15 Foundation movement at Avenida Sonora 149, a 14-story building in La Condesa South survey area (D3-G1) (19.4149°, -99.1693°). (<10cm settlement)
Figure 4.16 8-story building with intermediate story level collapse (19.41456°, -99.16905°) (G0-D5). Left: before and right: after (view B from Figure 4.13).
Figure 4.17 8-story building with intermediate story level collapse (19.4145°, -99.1690°) (G0-D5). (bottom: view C from Figure 4.13; back and side view to articulate lateral shift of collapsed floor)
Figure 4.18 Building cases along Calle Amsterdam (8, 9, and 3-story buildings) with 9-story building articulating worse case (G2) ground failure. (view D and E from Figure 4.13). (19.4143°, -99.1708°)

4.2.5 La Condesa North
A survey was conducted in La Condesa within the quadrant Av. Sonora – Av. Amsterdam - Calle Cacahuamilpa - and Av. Alvaro Obregon. A total of 51 buildings with heights ranging from 2 to 11 stories were inspected for structural damage and evidence of ground failure, as shown in Figure 4.19. This neighborhood is mostly residential but also has commercial and office buildings along Av. Sonora and Av. Alvaro Obregon. The oldest structures date from the early 1900’s and most (if not all) of the low rise buildings were built before the 1985 earthquake. One full collapse (D5) were observed within this survey; 2 cases of D4 (partial collapse); 4 cases of D3 (significant
damage to structural components); and 4 cases of minor damage to structural components (D2); in total, about 22% of the buildings in this survey zone observed moderate to significant structural damage. All of these buildings were between 2-9 stories in height. No significant ground failures were observed in this neighborhood. Only minor geotechnical damage was observed at four locations: in Av. Amsterdam the rocking of an 11-story building caused minor damage to the sidewalk and the ceramic tiles in the 1st floor. At the time of the reconnaissance the building tilt was estimated to be approximately 1 degree, but further analysis using Google Street View showed that this tilting existed before the earthquake (Figure 4.20). Likewise, the sidewalk along Av. Amsterdam in front of a 9-story building cracked and settled, apparently due to a poor compaction of the materials underneath and possible seismic rocking of the building. According to the neighbors, this RC moment frame building was retrofitted after the 1985 Michoacan earthquake. The building suffered light-to-moderate damage during the 2017 earthquakes, including concrete spalling on several floors, large diagonal cracks on partition walls, and damage to utilities (Figure 4.21).

The catastrophic collapse of a 7-story office building in Av. Alvaro Obregon #286 killed 49 people and left many injured. The structure was built in the 1970s and consisted of open-plan floors with thick concrete slabs and insufficient load bearing elements. The upper five stories collapsed on top of the 2nd floor (Figure 4.22) and caused a partial collapse of the adjacent building (Av. Alvaro Obregon #284). The first floor and the parking garage withheld the rubble and no ground failure was apparent. A secured perimeter was established around this site and the rescue teams occupied several blocks, making this one of the largest relief operations witnessed by the main team in CDMX.

Just 60 m south-east of Alvaro Obregon #286, the southern wing of a 5-story building collapsed, as shown in Figure 4.23. This building was made of unconfined masonry and concrete slabs; the remaining portion of the building seemed to have minor structural damage. Despite these collapses, most buildings in the survey area performed well, especially 2 and 3 story stiff structures, with a few exceptions. For instance, a 2 story masonry house in the south-west corner of Av. Amsterdam and Cacahuamilpa had large diagonal cracks in the 2nd floor walls and large residual deformations along the cracks (Figure 4.24). Damage was noted in several structures which appeared to be due to impact between adjacent structures. The “Complementary Technical Norms for Earthquake Resistant Design” (subsection 6.1) establishes a minimum separation from site boundaries equal to 0.006 × height + 50 mm for buildings located in zone III of CDMX (e.g., in La Condesa), and the total separation between adjacent bodies should be computed by adding the values from both bodies. Thus, for instance, the total gap between two buildings of 3-stories each should be in the order of 20 cm, which is significantly more than separations observed by the main team in this area.
Figure 4.19 Damage survey in La Condesa North along Av. Amsterdam and Av. Alvaro Obregon (19.4160°, -99.1683°) (viewpoints A-E in subsequent figures). Orthomosaic image created with UAV flyover conducted on 9/30/2017. Building footprints are approximate and obtained from aerial images and Google Street View.

Figure 4.20 Permanent rotation of 11-story building in La Condesa. Left: before the earthquake (Google Street view photo from 2016), Right: after the earthquake (view A from Figure 4.19).
Figure 4.21 Seismic induced settlement of sidewalk and damage to building foundations (D1-G1) (view B from Figure 4.19).

Figure 4.22 Left: Complete collapse of 7-story building at Alvaro Obregon #286 (view E from Figure 4.19), and right: partial collapse of adjacent building (Alvaro Obregon #284)
4.2.6 La Plaza Condesa

The advance and main UNAM-GEER teams examined the performance of the Plaza Condesa located in the Condesa neighborhood of CDMX (19.4129°, -99.1722°; Figure 4.25-Figure 4.26). This structure is supported on end-bearing, controlled piles (Figure 4.27), which are periodically adjusted to offset differential settlement of the foundation. The building consists of an outer U-
shaped portion which is 12 stories tall and a shorter inner portion which serves as a concert hall. The building was occupied and in use at the time of the UNAM-GEER reconnaissance.

Figure 4.25 UAV image of Plaza Condesa showing the direction of tilt and viewports A through E which are shown in subsequent figures. (19.4130°, -99.1722°).

Figure 4.26 UAV-generated 3D model with view of the Plaza Condesa (view looking North East).
The structure sustained mostly cosmetic damage during the earthquake as evidenced by broken windows and damage to the building façade (Figure 4.28). The interior of the building was not inspected. The advance team noted that the north side of the building appeared to have some tilt to the north. The Theodolite app was used to optically measure this tilt as approximately 1 degree to the north (Figure 4.29), but it is not clear how much of this tilt was due to the earthquake or uneven leveling/maintenance of the controlled pile foundation.

**Figure 4.27 End-bearing control piles supporting the Plaza Condesa building (19.4132°, -99.1721°)**

**Figure 4.28 Cosmetic damage to the façade of Plaza Condesa (view A in Figure 4.25).**
Differential movement of the building and the surrounding ground was observed and measured at several locations (Figure 4.30-Figure 4.33). The general pattern of displacement indicated the building settled approximately 1-2cm along the north side relative to the surrounding ground and uplifted 1-2 cm along the south side relative to the adjacent buildings and surrounding ground. This conclusion was reached based on measurements taken at several locations. Along the north side of the building, there was little observed settlement of the surrounding ground and in some locations the building had appeared to have settled 1-2cm relative to the curb and surrounding ground (Figure 4.31). Along the east side of the building, there was significant differential movement between the structure and the sidewalk, but it was unclear how much was due to possible uplift of the building and how much was due to settlement of the surrounding ground. There was one location where settlement did appear to occur around a utility box (Figure 4.32), but Google Street View images show that some of this settlement occurred prior to the earthquake. At the southeast corner of the Plaza Condesa, the team observed cracking and offset at the interface with the adjacent structure to the south (Figure 4.33). At the southwest corner of the building, the team measured approximately 2 cm of uplift of the Plaza Condesa based on offsets in the sidewalk and a gas line entering the building (Figure 4.34). All of these observations are consistent with some global tilting of the building to the north, but the team was unable to determine how much of these movements occurred during to the earthquake.
Figure 4.30  Differential settlement at the northwest corner of the Plaza Condesa (view B in Figure 4.25). The front of the building and attached sidewalk had settled approximately 1.5 cm relative to curb and planter area.

Figure 4.31 Cracking along the edge of the sidewalk at the northeast corner of the Plaza Condesa (View C in Figure 4.25).
Figure 4.32 Settlement around a utility box (View D in Figure 4.25) after the earthquake (left) and in Google Street View images from January 2017 (right). The cracks had a maximum offset of approximately 5 cm after the earthquake. Some of the cracks are visible in the Street View image, but have no apparent offset.

Figure 4.33 Offset and cracking at the southeastern corner of the Plaza Condesa (Figure 4.25). The tape measure shows the location of a missing wall tile. The tape measure shows inches.
4.2.7 Escocia and Rebsamen/La Morena
The Escocia neighborhood is in Zone II (Transition soils); and contains a blend of residential and commercial buildings. Two buildings within this neighborhood collapsed (Edimburgo 4 and Escocia 4); in addition to several partially collapsed buildings. The UNAM-GEER team conducted a walking and aerial survey of this neighborhood as well as seismic testing (Figure 4.35). A contrasting site Rebsamen/La Morena, approximately 1.3km North of Escocia was also surveyed in similar detail as several buildings collapsed within this neighborhood and this site is within the deeper Lake zone IIIa soils geo-zone. Stations AU46, SI53, and SCT2 are the closest stations to these two sites, with stations AU46 and SI53 being within the same geo-zones as Escocia and Rebasmen/La Morena, respectively. Elastic response spectra from these motions demonstrate the elongating predominant period transitioning West to East, which varied from 0.8sec (AU46); 1.2sec (SI53), to 1.7sec (SCT2) (Figure 4.36-Figure 4.38); consistent with the increase in soft clay soil layers (transitioning from geo-zones II, IIIa, IIIb). Seismic tests confirm similar site periods (0.93sec and 1.28sec in Escocia and Rebsamen/La Morena, respectively, which are sites within Zone II and IIIa).
Figure 4.35 UNAM-GEER damage survey in the La Escocia and Rebsamen/La Morena neighborhood. Bounding blue box articulates UAV coverage area, site periods denoted were obtained by seismic testing conducted during the reconnaissance (see Chapter 8).

Figure 4.36 Elastic response spectra at station AU46, 0.5 km SW of Escocia 4, in zone II (Transition) soils
4.2.8 Escocia – Colonia del Valle
The neighborhood of Escocia has a high concentration of residential dwellings, with many multi-family buildings. While most buildings are low-rise (<3 stories), there are concentrations of mid-rise buildings (5 – 10 stories), with most of these constructed on a podium style concrete moment frame with an open first floor area for parking, retail, or other functionality. Although an extensive damage map of the neighborhood was not generated, the most severely damaged buildings were documented by the team (Figure 4.39). In addition, a walking survey and several UAV flights were conducted across this region to understand the building vintage and typology of adjacent structures (Figure 4.40). Overview isometric views from the UAV model generated from the imagery demonstrate the similar building stock within the neighborhood, though the vintage of construction varied. Notably a cluster of 6-8 story buildings offering residential dwellings are within just one block of the most severely damaged buildings and low-rise buildings are also just adjacent to the most severely damaged buildings (Figure 4.41). Neighbors mention that the cluster of 6-8 story buildings just north of the damaged buildings were constructed within the past 10 years. The most severe damage within the region outlined in Figure 4.39 and Figure 4.41 included two collapsed buildings (D5) (Escocia 4 and Edimburgo 4) and three partially collapsed buildings (D4). Importantly, no geotechnical (foundation or exterior hardscape surficial)
Measurements taken by station AU46 during the September 19th event (Figure 4.36) indicate that the NS and EW component of shaking were of approximate equal acceleration demand, with peak ground accelerations of between 0.08 – 0.1g. The maximum elastic spectral demands were at approximately 0.8-0.9 seconds (Figure 4.36a), with an amplitude of 0.4g (or a 4-5 fold acceleration amplification). The period range in which largest spectral accelerations concentrated is consistent with site period measurements taken by both the advance and main UNAM-GEER teams, which indicate the period at the site was 0.87sec and 0.93sec. Moreover, the observed damage concentration to weaker mid-rise buildings in this neighborhood is consistent with the period region of greatest acceleration content during this event at this site. Assuming $T_1 \approx 0.1N$, the pair of mid-rise buildings that collapsed in this neighborhood likely had elastic fundamental periods between 0.7 – 0.9sec, and even if softened due to pre-event damage or aging/poor construction the wide period band of the most significant spectral content would be sufficiently damaging to these buildings.

Imagery taken before and immediately after the event of Escocia 4 demonstrate that this concrete moment frame with brick infill 7-story building suffered a multi-floor pancake style collapse, leading to the loss of 19 lives (Figure 4.43). In addition, the collapse of Escocia 4 precipitated significant damage to Escocia 6, a similar vintage and structural style 5-story building, which neighbors reported was slightly newer vintage (though likely still pre-1985) (Figure 4.44). Neighbors believed that Escocia 4 may not have received sufficient inspection and retrofit following the 1985 event.

The second completely collapsed building in this neighborhood was a 9-story multi-family dwelling also constructed of a concrete moment frame brick infill load resisting system. This building, at Edimburgo 4 suffered collapse of its upper 5 floors (see before and after photographs, Figure 4.45). The overview UAV model of this and its immediately adjacent buildings offer insight into its relationship with adjacent buildings (Figure 4.46). Notably the multi-floor catastrophic collapse of Edimburgo 4 precipitated damage onto the adjacent mid-rise building at Escocia 29, an 8-story building and the low-rise (3 story) building at Edimburgo 6, both of which were constructed within the same vintage as Edimburgo 4 (pre-1985). Damage to Edimburgo 6 (Figure 4.47-Figure 4.48) was induced by stripping of the exterior walls and subsequent loss of floor load bearing support at floors 2 and the roof. While damage to Escocia 29 was limited to localized floor collapse at the northeast corner of the building at level 2 (Figure 4.49). Notably, other mid-rise buildings (particularly many 6-8 story buildings) remained fully intact and functional following the event. Neighbors reported that these buildings were all constructed after 1985 (e.g., Figure 4.50-Figure 4.51) though in several cases similar open first floor designs were utilized (Figure 4.51). This again speaks to the importance of careful post-event inspection and adoption of repair actions.
Figure 4.39 Damage map of the Escocia neighborhood. Orthomosaic image created with UAV flyover conducted on 10/1/2017. Building heights are approximate and obtained from Googlesreet view. (19.3876°, -99.1632°).
Figure 4.40  Snapshot from the UAV-generated model (flight conducted 10/1/2017). (far view)

Figure 4.41  Snapshot from the UAV-generated model (flight conducted 10/1/2017). (near view)
Figure 4.42 Escocia 4 during various stages of demolition (left – UAV aerial image 09/27/2017) and right, view A of Figure 4.39, note seismic testing by GEER team at front of building, 10/1/2017). (19.3877°, -99.1637°)

Figure 4.43 Before (top left) and after (top right) images of the Escocia 4 (image on left courtesy of Google Street View taken in 2016; image on right courtesy of Marco Antonio Cruz, Proceso, 2017). (19.3877°, -99.1637°)
Figure 4.44 View B showing the remaining collapsed floors at Escocia 4 and exterior damage to adjacent building (View B of Figure 4.39). Photo taken on 10/1/2017. (19.3877°, -99.1637°)
Figure 4.45 (left) Google earth streetview image of Edimburgo 4 (before, taken in 2016) and social media image (available within CICM, 2017) taken immediately after the event articulating the upper multi-floor collapse. (19.3873°, -99.1633°)

Figure 4.46 Snapshot from the UAV-generated model (flight conducted 10/2/2017). View of collapsed building Edimburgo 4 and adjacent (2) partially collapsed buildings (Escocia 29 and Edimburgo 6). Approximate central GPS coordinates of the region: 19.3873°, -99.1633°).
Figure 4.47 (top) Before from google street view and (bottom) after images of the Edimburgo 4, 6, and 8 Buildings (Edimburgo 4 fully collapsed; Edimburgo 6 partially collapsed due to Edimburgo 4 and Edimburgo 8 remained in tact and functional) (19.3872°, -99.1633°). Bottom image is view E of Figure 4.39.
Figure 4.48 View C of Figure 4.39 showing the podium of the collapsed building at Edimburgo 4 remaining (center), partial collapse of Escocia 29 (left 8-story building) and partial collapse of Edimburgo 6 (right 3-story building). Photo taken on 10/1/2017.

Figure 4.49 View D of Figure 4.39 showing (left) the partial floor collapse of Escocia 29 adjacent to Edimburgo 4 (only podium floor remaining) and (right) zoom-in view of floor collapse (temporary shoring visible). Photo taken on 10/1/2017.
Figure 4.50 Six story ("1990s constructed") 6-story building at Escocia 14 (19.3875°, -99.1634°) within the most severely damaged area. Building remained in tact and operational immediately after the event. This building had a lower single level parking/basement. (cordoned off region at street level to right is Edimburgo 4).
Figure 4.51 Cluster of six story buildings just north of Escocia 4 (View F Figure 4.39), (constructed ~2000) undamaged and reported to be fully functional after the event. (19.3880°, -99.1637°).

4.2.9 Enrique Rebsamen/La Morena – Colonia Navarte Poniente

The neighborhood along La Morena, particularly at the intersection of Enrique Rebsamen/La Morena is approximately 1.3km northeast of the aforementioned Escocia survey area, however, it is situated on a thicker deposit of lake soils, therefore rests within geo-zone IIIa. This region began its development in the 1940s, and contains a blend of residential multi-family, commercial and public buildings. Similar to the aforementioned Escocia survey area, most buildings within the immediate area surveyed by the UNAM-GEER team are low-rise (<3 stories), however, there are a number of mid-rise 6-8 story buildings which suffered significant damage (Figure 4.52). UAV flights over this neighborhood (Figure 4.53) offer insight into the relationship between different buildings and damage distribution. The most significant damage included a concentration of buildings just north of La Morena along Enrique Rebsamen. The isometric view of the UAV-3D model shown in Figure 4.54 shows an overview of the most significant cluster of damaged buildings. Notably, Enrique Rebsamen 249, 241, and 237 suffered structural damage categorized as D4, D4, and D5, respectively. In contrast, Enrique Rebsamen 245 suffered no damage, and
according to occupants, was functional immediately after the event, though access was limited due to demolition efforts for the adjacent buildings. Of the significantly damaged buildings, Enrique Rebsamen 249 is an 8-story concrete moment frame building with brick infill that articulated a soft first story (Figure 4.55). Poor detailing of columns combined with the large open first floor to accommodate vehicular parking resulted in extensive damage to the first floor columns and wide shear cracking of interior infill walls in this building (Figure 4.56). Exposed, damaged columns reveal insufficient confinement (wide spacing of lateral ties), leading to buckling of longitudinal rebar (Figure 4.57). Directly adjacent to this building are two 6-story buildings of approximately the same vintage (according to neighbors); Enrique Rebsamen 245 and 241. Enrique Rebsamen 241, a 5-story building, suffered a complete collapse of its ground floor (Figure 4.58) and was being demolished at the time of the teams visit (Figure 4.59); whereas building 245 was undamaged. While the contrast in these three buildings is unique, the asymmetric geometric layout and lack of ductile detailing precipitated the failure modes observed in the pair of buildings (249 and 241) which suffered the most significant damage (Figure 4.60). Building 245 appeared to be a lighter building, with less wall lines (lower stiffness) at its exterior, street front in the upper floors and vertical symmetry (upper floor wall lines aligned with ground floor walls lines with little overhang) (Figure 4.61). Other buildings with similar layout (exterior walls at the perimeter in the upper floors); yet symmetric vertical wall lines, performed well, with only minimal damage to structural walls at the upper floors (e.g., Figure 4.62). It is important to note that the collapse of the shorter 2-story residential dwelling at Enrique Rebsamen 237 was due to the collapse of its adjacent building 241 (Figure 4.63). As shown in the damage maps in Figure 4.52, no notable ground failure were observed along Enrique Rebsamen; however, one building just one block east of La Morena/Enrique Rebsamen did observe appreciable ground failure (La Morena 716). This 8-story concrete moment frame corner building suffered appreciable rocking-induced movement (Figure 4.64). The dynamic rotations were significant enough to result in pounding-induced damage to a 3-story building (La Morena 710; Figure 4.65). Evidence of rocking manifest in the form of foundation-hardscape damage at the front (corner) wall lines of the building (Figure 4.66). The vintage of this neighborhood, in addition to discussions with neighbors, suggests that the most significantly damaged buildings in this survey were of 1960s vintage or older.

Measurements taken by station SIS3 during the September 19th event (Figure 4.37) indicate that the EW component of shaking was slightly stronger than the NS component, with peak ground accelerations of 0.13 and 0.18g, respectively. In addition, the EW component observed its maximum elastic spectral acceleration at a lower period of 1.2sec, compared with 1.4sec for the NS component (Figure 4.37a); nonetheless both larger periods than the nearby Escocia site sensor (AU46 – in zone II soils). The elongation in period of peak spectral accelerations compared with AU46 is consistent with the deeper deposits of soil at sensor location SIS3, which is in the zone IIIa lake soils. Again, the period range in which the largest spectral accelerations concentrate is consistent with site period measurements taken by both the advance and main UNAM-GEER teams, which indicate the period at the site was 1.28sec. The relatively longer period of peak spectral demands may be why taller (8-story) buildings in this neighborhood were so vulnerable. This surface level recording also offers an appreciable spectral content at 0.5sec period, with coincident appreciable vertical motion content at this period as well. In fact, at a period of 0.5sec,
an acceleration amplification (acceleration at zero period relative to acceleration at 0.5sec) of 3.0 and 2.8, resulting in accelerations of 0.4 and 0.5g in the NS and EW directions, respectively, are observed.

Figure 4.52 Damage map of the Enrique Rebsamen/La Morena neighborhood. Orthomosaic image created with UAV flyover conducted on 10/1/2017. Building heights are approximate and obtained from Google Street View.
Figure 4.53  Snapshot from the UAV-generated 3D model of Enrique Rebsamen/La Morena neighborhood (flight conducted 10/1/2017). (far view) Approximate central GPS coordinates of the region: 19.3984°, -99.1587°.

Figure 4.54  Isometric view of building damage cluster along Enrique Rebsamen, snapshot from the UAV-generated 3D model of Figure 4.53). Approximate central GPS coordinates of the region: 19.3986°, -99.1587°.
Figure 4.55 Enrique Rebsamen 249 elevation: (left) Google street view (2017) and (right) view A of Figure 4.52. (19.3986°, -99.1588°)
Figure 4.56 Enrique Rebsamen 249: (left and bottom) first floor shoring and (right) interior column and shearwall showing thru shear hinge articulating soft story collapse. (19.3986°, -99.1588°)
Figure 4.57 Local view of detailing of Enrique Rebsamen 249: (left) moment frame column-infill wall and (right) moment frame columns. (19.3986°, -99.1588°)

Figure 4.58 (left) Before images (Google Street view, 2016) and (right) images of the Enrique Rebsamen 241; (19.3989°, -99.1588°).
Figure 4.59 (upper): UAV images of Enrique Rebsamen 249 during demolition, also shown are 241 (left most 8-story building, being temporarily shored) and 245 (untouched) (19.3988°, -99.1587°). Flyover on 09/28/2017. (lower): street level images of Enrique Rebsamen 241 during demolition.

Figure 4.60 Images of Enrique Rebsamen 249, 245, and 241 (left to right); 249 first floor soft story collapse – building shored, 245 no damage and functional, 241 first floor soft story collapse – demolition complete) (19.3988°, -99.1587°). View B of Figure 4.52.
Figure 4.61 Enrique Rebsamen 245: building undamaged and functional after the event. (19.3988°, -99.1588°).
Figure 4.62 Four-story building with minor damage in the form of column and beam shear cracks. Building constructed in 1940s. (19.3996°, -99.1588°)

Figure 4.63 Enrique Rebsamen 237 (2-story building on right of image): building damaged during collapse of Enrique Rebsamen 241 (central remaining debris pile). (19.3990°, -99.1588°). View C of Figure 4.52.
Figure 4.64 La Morena 716 (Eight-story building with rocking-induced damage). (left) Google street view (2016) and (right) elevation after event (view E of Figure 4.52). (19.3986°, -99.1582°)

Figure 4.65 La Morena 710 (left 3-story building) and 716 (right 8-story building). (left) before – google street view in 2016 and (right) after event showing impact damage at floors 2-roof (view D of Figure 4.52). (19.3986°, -99.1582°)
4.2.10 Performance of Building Foundations

In general, the UNAM-GEER team observed three types of structural foundations commonly used in the affected areas of Mexico City: (1) end-bearing piles, (2) friction piles combined with mat foundations, and (3) shallow or excavated mat foundations with floating superstructure. The end-bearing pile foundations that were observed were either permanent and fixed (e.g., Figure 4.67), or incorporated adjustable controls (termed “control pile”, e.g., Figure 4.68) to mechanically lower the superstructure incrementally in an effort to level the structure with the ground surface, which is settling at an average rate of 10 cm per year due to groundwater extraction from beneath the city.
Figure 4.67 An end-bearing pile structure built in 1966 that experienced approximately 3 cm of settlement in the soil surrounding the structure; otherwise, the structural components of the building were generally undamaged. Note that the building entrance used to be level with the sidewalk, but is now 1.25 meters above the sidewalk elevation, due primarily to pre-event settlement ($19.4146^\circ, -99.1705^\circ$).
Performance of the different types of foundations was observed to be the worst in the affected western portions of Mexico City in the Transition Zone II soils and Lake Zone IIIb soils. End-bearing piles generally performed well, with occasional settlements ranging from 3 to 8 cm in the ground or hardscape surrounding the structure, as shown in Figure 4.69 (visible at stairs). Within CDMX, the team observed appreciable tilt in an end-bearing pile structure only at La Plaza Condesa (19.4129°, -99.1722°; see Figure 4.70 and Figure 4.34), which is supported by controlled piles. While up to 1 degree of tilt was measured on the north side of the building, the team was not able to determine whether this tilt is due to the earthquake or to uneven maintenance or adjustments of the controlled pile foundations.
Figure 4.69 Settlement of approximately 15 cm in the soil surrounding an end-bearing pile-supported structure (19.4146°, -99.1684°)

Figure 4.70 The 12-story La Condesa building (19.4129°, -99.1722°), which was tilting approximately 1 degree to the north.
Many structures founded on friction piles with mat foundations did not perform well in the earthquake. While little to no differential settlement between the structure and the surrounding soil was generally apparent because the structure tended to settle over time with the surrounding ground, there were observed cases of permanent structural tilt following the earthquake, as shown in Figure 4.71. This damage likely occurred as the rocking structure with its corresponding friction piles weakened the underlying lacustrine clays sufficiently to induce cyclic softening and reduced shear strengths beneath the structure and along the friction piles. As a result, piles on one side of the building were uplifted, and the mat foundation on the opposite side of the structure caused shear-induced deformation and bearing capacity failure in the underlying clay. While is it certainly possible that many of the damage that affected areas of Mexico City were caused by structural deformations following the 1985 and 2017 earthquakes, many of the tilted buildings that the UNAM-GEER team inspected in La Condesa showed no signs of structural distress (e.g., cracking, exposed rebar, spalling, etc.). The team did not address whether the observed damage was due to the 1985 or 2017 earthquakes; however, future efforts should do so to refine the observations that were gathered.

Figure 4.71 Permanent tilt of approximately 2 degrees in the friction pile-supported structure on the left; the structure on the right is vertical (19.4118°, -99.1711°)
Relatively few structures in the affected areas of western Mexico City appeared to be constructed on shallow mat foundations. Those that were confirmed as such were usually more than 70 years old and less than two stories in height. Among these structures and their foundations, the ones that were inspected by the advance team performed relatively well.

4.3 Colonia Del Mar, Iztapalapa, Tlahuac
Ground subsidence has been a major problem for many areas in Mexico City. Specifically, in Iztapalapa, where ground subsidence of up to 40 cm per year has been observed mainly due to water pumping from deep aquifers. The National Risk Atlas developed by UNAM Center of Geosciences classify the area as high risk for ground fracturing and plots the cracks mapped in the region due to ground subsidence (Figure 4.72). The mapped region is located at the foothills with clayey, possibly transitional sediments between the lake soils and the rocky soils close to the hills in the south. Following the September 19, 2017 Mw 7.1 Puebla-Morelos Earthquake the Colonia del Mar neighborhood, located in the southern part of Iztapalapa, experienced additional ground subsidence of up to 1 m and suffered a series of additional cracks that damaged both sewage and fresh water pipelines (Figure 4.73; pipeline damage detailed in Chapter 6). As a result, depression bands crossed streets and entire city blocks. The black points shown in Figure 4.72 indicate locations of significant ground failure observed in the area by the advance and main GEER teams following the September 19, 2017 Mw 7.1 earthquake, which coincide with previously mapped ground cracks in the region indicating that majority of the observed ground failures occurred at previously subsiding locations.

Figure 4.72 Ground subsidence cracks mapped by UNAM Center of Geosciences in the Colonia Del Mar Neighborhood. Black dots indicate the locations of significant ground failure observed in the area by the advance and main GEER teams following the September 19, 2017 MW 7.1 earthquake.
4.3.1 Colonia del Mar South (Tlahuac)
Several GEER Main team members (denoted herein as Team 1 and Team 2) visited Colonia Del Mar on October 1, 2017 for initial reconnaissance and damage assessment of the neighborhood. The surface tracks of team 1 were largely dedicated to the southern region of Colonia Del mar, and are shown in Figure 4.74, together with the locations of recorded cases where surface cracks were observed. Table 4.5 provides a summary of the surface cracks observed at each location and the associated ground and building damage index per Bray and Stewart (2000). At the time of the Main Team’s visit, sewage pipeline repairs were underway. Most of the roads with pipeline breakage and ground settlement were closed to traffic. Fresh water supply in the neighborhood was initially provided through water trucks following the September 19, 2017 earthquake, but partial pipeline repairs enabled freshwater supply at the time of the visit. This section provides detailed description of observed damage and surface cracks for each case. UAV flights in the region offer an overview of the damage patterns from an aerial perspective (Figure 4.75-Figure 4.76). In addition, two HVSR measurements were taken by the GEER main team, very close together, but one outside of an existing cracked region (19.2883°, -99.0635°) and one adjacent to a cracked region (19.2887°, -99.0633°); in the former a site period of 2.0sec was estimated; while the later observed a slight elongation in site period to 2.08sec.
Figure 4.74 Team 1 surface tracks and locations of documented case histories (cases 1-9)

Figure 4.75 Orthomosaic image of the Del Mar region (refer to Figure 4.74). UAV flyover on 10/1/2017.
Figure 4.76 Ortho mosaic image of the Del Mar region (refer to Figure 4.74). UAV flyover on 10/1/2017.

Table 4.5 Summary of documented cases of damage and surface cracks in Colonial Del Mar (team 1)

<table>
<thead>
<tr>
<th>Case ID</th>
<th>Ground Damage Index</th>
<th>Building Damage Index</th>
<th>Street</th>
<th>Crossing Street(s)</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Crack Settlement (cm)</th>
<th>Crack Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>GF2 to GF3 (mostly GF3)</td>
<td>D0 to D1</td>
<td>Curel</td>
<td>Aleta</td>
<td>19.2838</td>
<td>-99.0578</td>
<td>75</td>
<td>22</td>
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<td>2</td>
<td>GF3</td>
<td>D1</td>
<td>Almeja</td>
<td>Sirena &amp; Aleta</td>
<td>19.2839</td>
<td>-99.0572</td>
<td>30</td>
<td>21</td>
</tr>
<tr>
<td>3</td>
<td>GF2</td>
<td>D1</td>
<td>Langosta</td>
<td>Ostion &amp; Aleta</td>
<td>19.2840</td>
<td>-99.0567</td>
<td>50</td>
<td>30</td>
</tr>
<tr>
<td>4</td>
<td>GF3</td>
<td>D1 to D3</td>
<td>Sirena</td>
<td>Pampano &amp; Atun</td>
<td>19.285</td>
<td>-99.0579</td>
<td>20 to 35</td>
<td>14</td>
</tr>
<tr>
<td>5</td>
<td>GF3</td>
<td>D2 to D3</td>
<td>Gitana</td>
<td>Pez Villa</td>
<td>19.2847</td>
<td>-99.0577</td>
<td>30 to 40</td>
<td>18</td>
</tr>
<tr>
<td>6</td>
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<td>D4</td>
<td>Gitana</td>
<td>Aleta</td>
<td>19.2878</td>
<td>-99.0634</td>
<td>50 to 75</td>
<td>24</td>
</tr>
<tr>
<td>7</td>
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<td>D0 to D1</td>
<td>Gitana</td>
<td>Aleta</td>
<td>19.2874</td>
<td>-99.0637</td>
<td>55 (stepped)</td>
<td>97</td>
</tr>
<tr>
<td>8</td>
<td>GF3</td>
<td>D3</td>
<td>Piguene</td>
<td>Sirena &amp; Aleta</td>
<td>19.2887</td>
<td>-99.0633</td>
<td>&gt;100</td>
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<tr>
<td>9</td>
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<td>D4</td>
<td>Sirena</td>
<td>Salmon</td>
<td>19.2895</td>
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<td>40</td>
<td>50</td>
</tr>
</tbody>
</table>
**Case 1:**
The GEER team observed significant ground settlement along Curel Street (St), which was approximately 22 meters (m) in length. The most significant settlement was observed at the intersection of Curel and Aleta streets, where the measured settlement was up to 75 centimeters (cm). A ground failure index (GFI) of GF2 to GF3 was assigned to this site due to major settlements observed at the site. The sidewalks along Curel Street were damaged as they were detached from the curbs (Figure 4.77). Generally, the sidewalk was at the same elevation as the houses in this block, while the street had settled considerably. In addition, a surface crack was observed crossing Curel St in a diagonal direction to the west of Aleta St. (Figure 4.78a). A local resident indicated that settlement was observed immediately after the September 19 earthquake. The resident also indicated she first felt the vertical motions, then a primarily rotational horizontal movement that made her dizzy. The waste water pipeline two streets south of Curel Street broke due to ground subsidence and caused service interruptions in the area. A trench was opened at the intersection of Aleta and Curel Streets for inspecting the waste water pipelines; no significant damage to the pipelines was observed at that location (Figure 4.78b). The houses in the area did not suffer any observed damage or settlement in this case, even though the street had suffered large relative displacements. Accordingly, a building damage index (BDI) of D0 to D1 was assigned to the structures in the area.
Figure 4.77 Ground settlement along Curel Street caused damage in sidewalks. The houses in the vicinity did not experience significant damage. (Case 1; 19.2838°, -99.0578°)

Figure 4.78 (a) Surface crack across Curel St, to the west of Aleta St., (b) Open trench at the intersection of Curel and Aleta Streets for the inspection of pipeline. (Case 1; 19.2838°, -99.0578°)
Case 2:
Settlements were observed in an approximately 21-m long block along Almeja St between Sirena and Aleta St, which resulted in surface cracks crossing Almeja St perpendicular to each other on the northwest and southeast ends of the block (Figure 4.79a). Ground settlements of up to 30 cm were observed at this location. Sidewalks along the Almeja St experienced settlement related damage. The structures near the area suffered slight tilting and ground settlement related damage. On the northwest end, where larger ground settlements were observed, a tilted wall hit an adjacent structure, causing cracks in its supporting column due to pounding. Cracks in the masonry walls were also observed at a few locations (e.g., Figure 4.80). The main GEER team followed surface cracks on the southwest end into a residence. The one-story structure on the northwest side of Almeja St experienced damage due to ground settlements of up to 25 cm. Observed damage at the site included cracks in the foundation slab and in the masonry walls (Figure 4.81). In the backyard, an adjacent one-story structure was found to experience tilting and rotating, potentially related to the ground deformation (Figure 4.82). Considering up to 30 cm of ground settlement, the area was classified as GF3. Although some of the structures experienced cracks in the masonry wall as a result of tilting, no major cracks were observed in the load-bearing elements, and therefore structures in this area were classified as D1.

Figure 4.79 Surface cracks observed at (a) northeast and (b) southwest ends of 22 m long block along Almeja Street between Sirena and Aleta Streets where ground settlement up to 30 cm was observed. (Case 2; 19.2839°, -99.0572°)
Figure 4.80 Structural damage due to tilting of a wall and possibly pounding on the adjacent structure. (Case 2; 19.2839°, -99.0572°)

Figure 4.81 Ground settlement related damage resulting in differential settlement of the foundation slab, cracking in the foundation slab, and cracking in the masonry wall. (Case 2; 19.2839°, -99.0572°)
Case 3:
The wastewater pipeline along Langosta Avenue, between Ostion and Aleta Streets, broke at three different locations within a 30-m long block that experienced ground settlements up to about 50 cm (Figure 4.83). Broken pipelines interrupted the service provided in the Colonia del Mar neighborhood. At the time of the main GEER team visit, the broken pipeline was not restored and dewatering efforts were in progress. The area was classified as GF2 due to ground settlement and-related damage observed which affected the pipeline. The structures along Langosta Avenue did not show evidence of structural damage; occasional cosmetic damage was observed on some structures. Therefore, the structures in this location were classified as D1.
Case 4:
A 30-m long block along Sirena St. between Atun and Pampano Streets experienced ground settlements up to about 35 cm (Figure 4.84). Large ground settlements damaged the sidewalks with differential settlements up to 37 cm and surface cracks up to 45 cm in depth (Figure 4.85). Sirena St was closed to traffic between Atun and Pampano Streets. Considering the significant seismically-induced settlements in excess of 25 cm and deep cracks, the site was classified as GF3. Most of the buildings in the vicinity experienced light to moderate damage, while one building experienced sagging floors and shear cracking that could have led to collapse. The 4-story Saloon Social Place shown in Figure 4.86 experienced around 1 degree tilt toward the southwest direction due to ground movements shown in Figure 4.85. This tilted structure was supported by temporary wooden braces on its sidewalls. The Saloon Social Palace was classified as D3. Several other structures that experienced lighter damage in the vicinity were classified as D1 to D2. Additionally, the main GEER team visited a 2-story structure that suffered partial wall damage with shear cracks on the walls and the foundation slab (Figure 4.87). This structure was classified as D3. In this region, LiDAR was used to map the local ground cracking pattern (Figure 4.88).
Figure 4.84 Settlement of approximately 30-m long block along Sirena St. Ground Settlement measured up to 35 cm on both ends of the 30-m long block. (Case 4; 19.2851°, -99.0579°)
Figure 4.85 Damage to sidewalk along Sirena St. where measured differential settlement is up to 37 cm and surface crack depth up to 45 cm. (Case 4; 19.2851°, -99.0579°)

Figure 4.86 4-story building that experienced approximately one-degree tilt towards southeast direction due to ground movement. Note that the building was supported by wooden braces to prevent further movement. (Case 4; 19.2851°, -99.0579°)
Figure 4.87 2-Story building with sagging floors and shear cracks along the walls and the foundation slab. (Case 4; 19.2851°, -99.0579°)
Figure 4.88 Case 4 LiDAR model showing ground failure exemplified by shift in garage doors. (Case 4; 19.2851°, -99.0579°)

Case 5:
An 18-m long block along Gitana St. experienced ground settlements up to 30 to 40 cm, near Pez Villa Street. The road along Gitana St. was closed due to ground deformation, as sidewalks had suffered significant damage (Figure 4.89). Considering the significant settlement, in excess of 25 cm, the site was classified as GF3. The structures near the settled ground suffered light to moderate damage. Some single- to two-story structures experienced a tilt up to 3-4 degrees (Figure 4.90). A building damage index of D2 to D3 were assigned to these structures.
Figure 4.89 An 18-m block settled around 30 to 40 cm on Gitana St, near Pez Villa St. Street was closed to traffic, sidewalks suffered significant damage (Case 5; 19.2847°, -99.0577°)
Case 6

Ground settlements of up to approximately 75 cm were observed on Gitana street near its intersection with Aleta St, where an approximately 24 m-long block suffered extensive relative settlement (Figure 4.91). A two-story, unreinforced masonry structure on Gitana St experienced major damage including partial collapse of a non-structural roof element and some of the walls in the courtyard (Figure 4.92). The majority of the settlements were observed in the courtyard next to a six-story structure with a heavy water tank on its roof. The adjacent six-story structure appeared to have settled uniformly, while its heavy weight caused large differential settlements on the shorter masonry structure. The large differential settlement in the courtyard, due to interactions with the adjacent taller building, resulted in significant shear cracks and a partial collapse of the courtyard walls and balconies. The two-story masonry structure was classified as D4, while a ground failure index of GF3 was assigned to the block. In this region, LiDAR was used to map the local ground cracking pattern (Figure 4.93).
Figure 4.91 Approximately 24 m long block settled up to 75 cm on Gitana St near Aleta St. intersection causing differential settlement induced damage in the roadway, sidewalks, and adjacent houses. (Case 6; 19.2878°, -99.0634°)
Figure 4.92 Two-story structure damaged due to ground shaking and ground shaking induced differential ground settlement of up to 75 cm as well as due to downdrag of the adjacent 6-story structure. (Case 6; 19.2878°, -99.0634°)
Figure 4.93 Ortho and oblique views of LiDAR model (case 6), showing ground failure extending along sidewalk; oblique view shows the depth of the pavement shift. (19.2878°, -99.0634°)
Case 7
A surface crack that was approximately 97 m in length formed on Gitana St. to the northwest side of Aleta St after the earthquake, where settlements up to 55 cm were observed on the southern edge of the street and 15 cm along the centerline (Figure 4.94). A resident indicated that the observed cracks along Gitana St. lie along the existing water collector running beneath the street. The resident also mentioned that there was a stairwell access to the water collector, which had been closed due to safety concerns prior to the earthquake. A ground failure index of GF3 was assigned to the area. Slight tilting of the buildings and minor cracks on the walls and driveways were observed in some structures (Figure 4.95). Structures were therefore classified as D0 to D1, since they generally experienced no to light damage. It is possible that soil nonlinearity and cyclic softening in the lower clay deposits reduced the amplitude of ground shaking, preventing notable damage to the superstructure, even in cases where structures were not well designed. Differential settlements between the sidewalk and the road caused significant cracks, however.

Figure 4.94 Stepped ground settlement up to 55 cm on the edge and 15 cm in the middle of Gitana St. was recorded along the water collector pipeline to the northwest of Aleta St. Major damage to sidewalks due to differential settlement was observed. (Case 7; 19.2874°, -99.0637°)
Select structures along Gitana St. experienced slight tilt and minor cracks, while others did not show any visible damage. (Case 7; 19.2874°, -99.0637°)

Case 8
Differential settlements of up to 1-m along Pigueno St between Sirena and Aleta Streets caused major damage to foundations and structures along the street as well as the road (Figure 4.96). The largest ground settlements were observed at this location compared to other cases recorded by the GEER team. Ground failure observed along Pigueno St was classified as GF3. The structures experienced up to 3 degrees tilt and very severe damage. Some structures were supported by wooden braces to prevent partial collapse at the time of the main GEER team’s visit. The structures in this area were classified as D3. The residents pointed to the existence of pre-earthquake settlements, but notable settlement was observed immediately after the earthquake leading to extensive damage in this block. The block had continued to settle at a greater rate than its static condition after the event. One of the residents indicated that his house with reinforced slabs did not experience settlement-related damage; however, the basement below the slab experienced damage due to large ground subsidence and differential movements (Figure 4.97).
Figure 4.96 Significant settlement up to 1-m was observed along a 27-m long block on Pigueno St between Sirena and Aleta Streets. (Case 8; 19.2887°, -99.0633°)

Figure 4.97 Structures along Pigueno St experienced significant damage due to ground settlement. (a) structure supported by wooden braces to prevent partial collapse, (b) residents indicated reinforced slab did not experience settlement but the basement settled a significant amount, (c) cracks on masonry wall, and (d) cracks on the slab due to ground settlement. (Case 8, 19.2887°, -99.0633°)
Case 9

A 50-m long block at the intersection of Sirena and Salmon Streets experienced ground settlements on the order of 40 cm (Figure 4.98). The observed ground failure was classified as GF3, which manifested in settlement induced damage to sidewalks. Both Salmon and Sirena Streets were closed to vehicle traffic due to excessive and non-uniform ground settlements. Most of the structures in the vicinity experienced light to no damage except for a one-story structure located at the southern corner of the intersection (Figure 4.99). The one-story building experienced up to 3 degrees of tilt and rotation. A building damage index of D4 was assigned to this structure while the other structures are classified as D0 to D1. The severe rotation of this short building at the corner of the block was due to large differential settlements across the two cross streets. The nature of these non-uniform movements may indicate large variations in the thickness of the underlying clay deposit.

Figure 4.98 A 50-m long block experienced on the order of 40 cm settlement near the intersection of Salmon and Sirena streets. Near the intersection, both streets were closed to vehicle traffic. Slightly sloping ground conditions near the intersection, towards the direction of the observed ground settlement, indicated potential seismic slope stability problems that can might have caused the observed ground cracking and the settlement. (Case 9; 19.2895°, -99.0639°)
4.3.2 Colonia del Mar North (Tlahuac)

In addition to the cases documented in cases 1-9 in the aforementioned section, a second UNAM-GEER team (denoted here as Team 2) surveyed a region extending approximately 2-3km north of Colonia del Mar, extending across transitional (zone II) soils (Figure 4.100). These cases are summarized in Table 4.6.

**Case 10:**
The northernmost ground crack observed by the team was adjacent to the Parroquia La Inmaculada Concepción in the center of the Plaza Juárez (Figure 4.101). This extensive surface feature measured approximately 80m in length and observed an appreciable 20-30cm vertical offset. Locals indicated this was induced during the earthquake, though no damage to the Parroquia was noted. Surrounding buildings in the plaza observed only superficial damage due to the earthquake.

**Case 11:**
Team 2 conducted a drive-through survey of zone IIIb soils transitioning from case 10 South (traversing east-west) towards case 12 (Hospital General Tlahuac). In general, little to no damage was observed through this neighborhood. Locals indicated that some minor surface road damage occurred due to the earthquake, however the most pronounced foundation damage, which induced structural damage occurred to dwellings that had already suffered extensive damage due to existing pre-seismic settlements (e.g., Figure 4.102). The team measured a permanent rotation of 4.5 degrees of this two-story dwelling, however, locals report that this and its neighboring building across the street (Figure 4.103) had an appreciable rotation due to differential settlement pervasive in the area, prior to the earthquake. A large dip in the road running between these two dwellings was indicative of that observed throughout this neighborhood. Some of these were exasperated due to the earthquake and had already been repaired.
Figure 4.100 Team 2 tracks Colonia del Mar and north, including UAV survey region and HVSR locations (cases 11-14)

Table 4.6 Summary of documented cases of damage and surface cracks in Colonial Del Mar North (as documented by Team 2)

<table>
<thead>
<tr>
<th>Case ID</th>
<th>Ground Damage Index</th>
<th>Building Damage Index</th>
<th>Street</th>
<th>Crossing Street(s)</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Crack Settlement (cm)</th>
<th>Crack Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>GF2</td>
<td>D0</td>
<td>Juarez</td>
<td>Aquiles Serdan</td>
<td>19.2983</td>
<td>-99.0368</td>
<td>20-30</td>
<td>30</td>
</tr>
<tr>
<td>11</td>
<td>GF3</td>
<td>D3*</td>
<td>Alestis</td>
<td>Guillermo Dufay &amp; Bartolmoe de Escobedo</td>
<td>19.2839</td>
<td>-99.0572</td>
<td>10</td>
<td>7</td>
</tr>
<tr>
<td>12</td>
<td>GF2</td>
<td>N/A</td>
<td>Gabriela Mistral</td>
<td>Pulpo</td>
<td>19.2865</td>
<td>-99.0520</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>13</td>
<td>GF3</td>
<td>D1</td>
<td>Pulpo</td>
<td>Gabriela Mistral</td>
<td>19.2850</td>
<td>-99.0579</td>
<td>20</td>
<td>14</td>
</tr>
<tr>
<td>14</td>
<td>GF3</td>
<td>D1</td>
<td>Gabriela Mistral</td>
<td>Juana Ines de la Cruz</td>
<td>19.2825</td>
<td>-99.0549</td>
<td>20 to 50</td>
<td>160°</td>
</tr>
</tbody>
</table>

*Building damage confirmed by locals as being present prior to earthquake

*Crack was tracked across two schools and one football field, though aerial images were not taken, this extensive length does not represent a continuous crack (as it could not be reliably tracked; the longest possible on-foot surface crack that could be tracked had a length of approximately 60m)
Figure 4.101 Case 10: Northernmost surface crack observed by the UNAM-GEER main team within the Plaza Juarez, upper photo shows the Parroquia La Inmaculada, which was undamaged during the earthquake. (19.2983°, -99.0368°)
Figure 4.102 Case 11: Two-story dwelling observing permanent tilt of 4.5 degrees (19.2925°, -99.0529°)
Case 12:
A pair of ground surface cracks were observed at the entrance to a private multi-family dwelling neighborhood; directly adjacent to the Hospital General Tlahuac (case 13). Locals noted that the adjacent football field had expressed pre-earthquake surface settlements traversing through it for many years. The ground surface cracks ran transverse to Gabriela Mistral, however, did not traverse through any buildings, therefore their primary impact was to disrupt entry traffic into the private neighborhood (Figure 4.104). Residents in this private neighborhood commented that surficial cracks had exasperated over the years as the community was enlarged; and that the most pronounced surface cracking became more apparent during the construction of the General Hospital Tlahuac (case 13); approximately 5-7 years ago.
Figure 4.104 Case 12: Pair of ground surface cracks adjacent to the Hospital General Tlahuac parking area (left), along Gabriela Mistral. Note that these cracks were observed to extend into the neighboring football field (right); though no buildings spanned across this surficial damage of approximately, which at its largest was approximately 20cm settlement. (19.2865°, -99.0520°)

Case 13:
The Hospital General Tlahuac is a modern hospital constructed 5-7 years prior to the September 17th 2017 event (Figure 4.105). It is one of the largest in the region and services Col del Mar and surrounding regions, including offering urgent care services. The two most pronounced surface cracks adjacent to the hospital were observed along the southern boundary of the property on Pulpo street. Spaced at approximately 7m apart, this pair of cracks ran transverse to the street traffic direction and had a measured settlement between the cracks of about 20cm (Figure 4.106-Figure 4.107). Locals reported that water pipes at the location of these cracks were damaged. The hospital was designed to accommodate differential movement therefore, despite the extension of these cracks onto the hospital property and movement of one of its building units (Figure 4.108), locals noted that the hospital was closed for only two days and patients were not evacuated.

Seismic testing was performed along Pulpo street directly adjacent to the surface cracks and depressed regions observed (Figure 4.109). These tests indicate that the site period is much longer than other regions of CDMX, with a value of 2.0sec. In addition, UAV imagery was utilized to create a 3D model of this area, the boundaries of the flight path are shown in Figure 4.110. Perspective views of the 3D model created using this imagery are shown in Figure 4.111. Aerial imagery allows a useful view of the damage ensued at the interface of the two buildings that had to straddle the ground surface cracks and depressions that developed along the south boundary of the hospital (Figure 4.112).
Figure 4.105 Case 13: General Hospital Tlahuac (image of front of hospital, along La Turba – courtesy of Google Earth). (19.2872°, -99.0527°)

Figure 4.106 Case 13: Pair of ground cracks along Pulpo street, southern border to the Hospital General Tlahuac. (19.2869°, -99.0529°).
Case 13: Settlement measurements at the boundary fence between Pulpo street and the General Hospital Tlahuac property (hospital is in gray in background). (19.2869°, -99.0529°)
Figure 4.108 Case 13: Differential movement of pair of adjacent buildings which are part of the General Hospital Tlahuac property: (left) image at ground level (note the ground depression and damage to services entering building and (right) view of top of building at space between buildings (19.2869°, -99.0529°).
Figure 4.109 Site photo of Hospital Gral Tlahuac (case 13) showing the locations of active and passive surface wave arrays, and HVSR location. (19.2869°, -99.0529°)

Figure 4.110 Perspective looking south at the Hospital General Tlahuac showing the locations of case 12 and 13 relative to the hospital. Note the football field in the lower left corner was reported to have the earliest noted surficial cracks due to regional subsidence, dating back to the 1985 event (according to locals). Approximate central GPS coordinates of image: (19.2871°, -99.0524°). UAV flight 10/2/2017.
Figure 4.111 Perspective looking north at the Hospital General Tlahuac showing the locations of case 12 and 13 relative to the hospital. Football field in the upper right corner was reported to have the earliest noted surficial cracks due to regional subsidence, dating back to the 1985 event (according to locals). Approximate central GPS coordinates of image: (19.2860°, -99.0528°) UAV flight 10/2/2017.

Figure 4.112 Perspective looking onto the top of the Hospital General Tlahuac showing the repair areas at the interface between two buildings within the hospital complex that were isolated and moved differentially due to surface ground cracks (see Figure 4.105-Figure 4.107) (19.2872°, -99.0530°). 3D model created with UAV flight 10/2/2017.
Case 14:
South of the boundary of the UAV flight taken to capture the Hospital General Tlahuac, an extensive series of surface ground cracks were observed that extended through two schools and one surface concrete finished football field (Figure 4.113). Denoted case 14, damage due to this ground failure feature was documented only on foot (due to time constraints an additional UAV survey could not be conducted). A zoomed-in view shown in Figure 4.114 is assembled to guide in the documentation of this feature and its ensuing damage to the football field and adjacent schools. The open field and visibility of the concrete surfaced football field offered a clear view of the feature and its extent of damage (Figure 4.115-Figure 4.117). Measurements of ground settlement ranged from 20 – 50cm along the length of the most pronounced feature traversing across the lateral dimension (east-west) of the football field. The most significant settlement of 50cm was measured at the visible end of the feature within the adjacent park (Figure 4.118). This photograph also shows the shallow depth of the soft clay, approximately 40cm below ground surface. This feature was tracked west of the football field and through the adjacent primary schools (Figure 4.119-Figure 4.120). Resident caretakers of the westernmost school remarked that the school was closed immediately after the event and remained closed at the time of the GEER teams visit due to lack of water services to the facility (Figure 4.121). Damage to both school buildings was cosmetic (D1), with hardscape damage features distributed throughout both facilities (Figure 4.122), despite the extensive exterior damage to surrounding screen walls (Figure 4.123).

Figure 4.113 Case 14: Extensive series of surface cracks extending through two schools and a football field. (19.2825°, -99.0549°). Blue shows UNAM-GEER team tracks.
Figure 4.114 Case 14: Extensive series of surface cracks extending through two schools and a football field. (19.2825°, -99.0549°). (zoomin of Figure 4.113 used to articulate image views in subsequent image sequence). Blue shows UNAM-GEER team tracks.

Figure 4.115 Case 14: Surface crack running through football field (damage to screen wall due to ground failure feature, view A of Figure 4.114). (19.2825°, -99.0549°).
Figure 4.116 Case 14: 28cm depression measured at one location along ground failure feature running through football field (view B of Figure 4.114) (19.2825°, -99.0549°).

Figure 4.117 Case 14: Ground failure feature at boundary of football field (view C of Figure 4.114) (19.2825°, -99.0549°).
Figure 4.118 Case 14: Most significant measured ground settlement of 50cm in grassy field adjacent to concrete football field (clay layer is visible at 40cm below ground surface) (view D of Figure 4.114). 
(19.2825°, -99.0545°)
Figure 4.119 Case 14: Ground failure extending across Gabriela Mistral street at southwest entrance to football field (view E Figure 4.114). (19.2825°, -99.0552°)

Figure 4.120 Case 14: Two-story school at corner of Sor Juana Ines de la Cruz and Gabriela Mistral (view F of Figure 4.114). (19.2825°, -99.0552°)
Figure 4.121 Case 14: Ground failure feature extending into school – ground movement damaged the electrical and water supply system, whose central services shown here. School was closed due to lack of services. (view G of Figure 4.114). (19.2831°, -99.0557°)

Figure 4.122 Case 14: Ground failure feature extending into school – school damage was confined to cosmetic damage, however, surrounding hardscape disrupted services to the school (view H of Figure 4.114). (19.2830°, -99.0551°)
Figure 4.123 Case 14: Ground failure feature extending into school – view looking west at school perimeter screen fence (view I of Figure 4.114). (19.2830°, -99.0551°)

Additional Cases: Maneuvering through the 3D model generated with the UAV flight is revealing, and in this case allows the analyst to identify other cases of ground depression and associated damage in the region. For example, a subsequent case was identified using the 3D model (denoted as Case 15 in Figure 4.124). A zoom in to this region indicates that during the time of flight, road and piping repairs were underway (Figure 4.125); though the extent of this damage onto adjacent dwellings is unknown as the team did not visit this site during its field reconnaissance.
Figure 4.124 Perspective view northeast at the Hospital Gral Tlahuac showing the locations of new cases identified using the UAV-generated 3D model (case 15?). Flythrough of the model indicates additional ground failure features and potentially ruptured water pipes occurred at this location (see Figure 4.125). Approximate central GPS coordinates of image: (19.2860°, -99.0528°) UAV flight 10/2/2017.

Figure 4.125 Zoom-in of Figure 4.124 to damage region identified by maneuvering through the UAV-generated 3D model (case 15?). The GEER team did not have time to visit this site, however, the road and pipe repairs underway are apparent by viewing the model, which was created following the teams field reconnaissance. (circled region at: 19.2867°, -99.0544°)
4.4 Western Trek towards Epicenter (State of Morelos)

The state of Morelos is located in South-Central Mexico and bordered by the states of México to the north-east and north-west, Puebla to the east and Guerrero to the southwest. Mexico City is situated north of Morelos. The state is the second smallest in the nation and its capital is Cuernavaca, a northern most city in the State nearest to Mexico City. It has a population of just under 2 million, and its economy is largely based on agriculture. The states topography is very diverse: 42% is mountainous, 16% hilly land, and 42% flat terrain. The state falls between two main geographic formations, the Trans-Mexican Volcanic Belt in the north and east and the Sierra Madre del Sur, which stretches south and west from Cuernavaca and Jiutepec.

In general, Morelos is divided into several soil formations, however, the main ones are the Cuernavaca formation, the Chichinaultzin basalt series and the Morelos formation (Fries 1958). The Cuernavaca formation has conglomerates, fanglomerates, alluvial deposits, volcanic ashes, diatomaceous earth, peat, marl and travertine. In contrast, the Chichinaultzin basalt series is composed by its name sake, that is primarily basaltic and andesitic lava flows with minor tuff, breccia and alluvium. The Morelos formation is also formed by limestone and dolomite with anhydrite in the basal part.

According to the Manual of Civil Works Design, Earthquake Design, of the Federal Commission of Electricity (CFE), Morelos is located in an intermediate seismic zone where earthquakes are not as frequent or there are areas affected by accelerations that do not exceed 70% of the acceleration of the soil. Figure 4.126 shows a seismic hazard map of the state of Morelos. It is noted that both the main and advance teams documented damage in several cities in the state of Morelos, including severe damage in some locations which are identified in this map as areas of “very high” seismic danger, such as populated regions within the cities of Tlaquiltenango and Jojutla (Figure 4.126). The most devastated municipalities in the state of Morelos were Jojutla, Cuernavaca, Tecamac, Miacatlan, Yecapixtla, Yautepc, Cuatla, Xochitepec, Axochiapan, Tlayacapan, Oculta, and Zacatepec, as reported by the general Secretary of Government of Morelos (Animal Politico, 2017; Emporis, 2017). In the municipality of Jojutla, at colonia Emiliano Zapata, at least 4-blocks of downtown were completely destroyed, and by some estimates 80% of the city is destroyed or severely damaged. Approximately 20,000 houses were damaged, to some extent, in 22 out of 33 municipalities, which represents 60% of the state. At least 185 schools experienced damage (Animal Politico, 2017; Emporis, 2017).

Importantly, damage severity increased when traveling from north to south of the state, with the largest concentrations of damage in the two southern-most municipalities of Tlaquiltenango and Jojutla, which are closest to the epicenter. These cities are located approximately 70-80 km from the epicenter, therefore a more detailed survey was performed in these particular locations to examine the damage patterns therein.
Figure 4.126 Seismic hazard map of the state of Morelos (modified after Morelos government 2013)
Figure 4.127 UNAM-GEER team western trek towards epicenter including summary of HVSR measurements from Mexico city to Jojutla. Estimated site period is shown for each measurement location. Blue shows UNAM-GEER team tracks.

The survey methodology consisted primarily of visual surveys to document structural and geotechnical damage, which was supplemented with UAV surveys of damaged churches in Tlaltizapan and Tlaluitenango and UAV surveys of a damaged church and residential neighborhood in Jojutla. The UNAM-GEER main teams trek towards the epicenter is shown in Figure 4.127 along with locations of HVSR testing conducted in various towns along this trek. It is noted that the data only showed clear peaks in the towns of Cuernavaca, Emiliano Zapata and Jojutla. The other towns had no HVSR peak which is not surprising given the hilly terrain in these areas. The results clearly indicate a much smaller site period in the towns of Emiliano Zapata and Jojutla, which may correlate with the increased damage observations in these cities; in addition the site period is much smaller in Emiliano Zapata (0.09sec) than in Jojutla (0.27sec), where damage was extensive. Reasons for this difference in site period may include the different geological conditions in each of the towns. Notably, Emiliano Zapata is founded on primarily igneous rock while Jojutla is founded on alluvial soils (Figure 4.128). Detailed descriptions of the damage from each of the towns visited along the Western trek within the state of Morelos taken by the GEER team are described below.
4.4.1 Emiliano Zapata (80 km from epicenter)

The main and advance teams observed very little damage in the town of Emiliano Zapata. A walking survey was performed near the city center (18.8410, -99.1833) and only minor cosmetic damage was noted to the buildings (Figure 4.129). Most of the structures in this area appeared to be 1-2 story masonry-infill construction (Figure 4.129). Local residents informed the team that no major damage occurred within the city. In addition, the main hospital in this area, a modern hospital constructed within the last 5 years, ISSSTE, locals reported as undamaged and functional immediately after the event (Figure 4.130). Minor damage to the bell tower of the church (Iglesia San Francisco de Zacualpan) was observed (Figure 4.131) and the church was closed to the public. The sign outside of the church indicated it was constructed in 1755.
Figure 4.129 Little to no damage was observed near the city center in Emiliano Zapata (left; 18.8410°, -99.1834°). Much of the construction in this area was masonry in-fill (right; 18.8409°, -99.1841°).

Figure 4.130 Modern hospital undamaged and remained functional immediately after the event (ISSSTE, Palo Escrito, 62765 Emiliano Zapata, Morelos, Mexico; 18.8484°, -99.1964°).
4.4.2 Santa Rosa Treinta (74.5 km from epicenter)
The town of Santa Rosa Treinta was visited by the main team. This town had a mix of masonry infill construction and adobe homes. Only cosmetic damage was observed in the masonry infill homes (Figure 4.132), however, several collapses were observed in adobe homes (Figure 4.133). The bell towers of the church (Parroquia de Santa Rosa de Lima) had partially collapsed, but no structural damage was observed from the exterior (Figure 4.134). The church was closed at the time of the teams visit.

Figure 4.132 Masonry in-fill buildings surveyed in Santa Rosa Treinta showed little to no damage (left; 18.6968°, -99.1796°) (right; 18.6977°, -99.1796°).
Figure 4.133 Damage to adobe buildings in Santa Rosa Treinta showing complete collapse (left; 18.6976°, -99.1796°) and partial collapse (right; 18.6966°, -99.1794°).

Figure 4.134 Damage to the Parroquia de Santa Rosa de Lima in Santa Rosa Treinta. The tops of the bell towers had collapsed, but the church did not appear to sustain significant structural damage (18.6968°, -99.1776°).

4.4.3 Tlaltizapan (68 km from epicenter)
Both the main and advance teams visited the town of Tlaltizapan. The town did not sustain widespread damage, but both the main and advance teams noted several structures in the town which collapsed. No geotechnical damage was observed. The main team performed a detailed survey of damage to structures around the town center (Figure 4.135) Figure 4.138). In this area, only cosmetic damage was noted to the municipal building and masonry infill structures (Figure 4.136); while three collapses of adobe structures were observed (e.g., Figure 4.137). On the southern edge of town, a masonry structure was found with significant structural damage (Figure 4.138), although this structure did not appear to have any reinforcement. This represented the
first non-adobe home with significant structural damage that was observed by the main team in any of the towns surveyed on this trip.

Figure 4.135 Damage map for Tlaltizapan showing structural and geotechnical damage categories. The numbers on the buildings indicates the number of stories and the building footprints were defined using satellite images and should be considered approximate. (viewpoints A-E in subsequent figures). (18.6834°, -99.1185°).

Figure 4.136 The municipal building in Tlaltizapan showed only cosmetic damage (view A in Figure 4.135). Masonry in-fill structures near the town center also performed well (view B in Figure 4.135).
Figure 4.137 Collapse of two adjacent adobe structures (view C in Figure 4.135).

Figure 4.138 Significant structural damage to unreinforced masonry structure on the southern edge of Tlaltizapan.

Significant damage occurred at the church, Parroquia San Miguel Arcangel, including damage to the bell tower and large cracks in the exterior walls (Figure 4.139). The church was closed at the time of the GEER teams survey. A UAV survey of the church was performed to document this damage, and video documentation were generated from this imagery survey (see NHERI database; and references in Chapter 7).
Figure 4.139 Damage to Parroquia San Miguel Arcangel in Tlaltizapan showing damage to the bell tower (left, view E in Figure 4.135) and cracks in the exterior walls below the tower (right, view F in Figure 4.135).

4.4.4 Tlaquiltenango (71.5 km from epicenter)
Both the main and advance teams visited the town of Tlaquiltenango, which suffered significant damage including the collapse of several 1 to 3 story buildings. Damaged structures included both masonry in-fill and adobe construction, with many of approximately 100-200 years of age (according to locals) (Figure 4.140). No geotechnical damage was observed. The adobe municipal building in the center of the city had partially collapsed (Figure 4.141a) and the nearby church (Parroquia Santo Domingo de Guzman; Figure 4.141b), which observed large cracks that extended through its full height. A full damage survey was not performed in Tlaquiltenango, however, a UAV survey of the church was performed to document its performance. A 3D model was generated from the UAV survey; and in particular articulates the extensive damage from an aerial perspective (roof collapsed regions evident in Figure 4.142).
Figure 4.140 Collapsed structures in Tlaquiltenango included both masonry in-fill (left; 18.6387°, -99.1602°) and adobe (right; 18.6300°, -99.1604°) construction.

Figure 4.141 Partial collapse of the municipal building (left; 18.6290°, -99.1612°) and the Parroquia Santo Domingo de Guzman (right; 18.6294°, -99.1600°).
Figure 4.142 Viewpoints from the UAV model of the Parroquia Santo Domingo de Guzman (top: isometric view of damaged belltower, forefront of image; and bottom: aerial (roof) view – note the visible roof collapse in several regions of the structure, collapsed bell tower visible in upper left of this image). (18.6294°, -99.1600°).
4.4.5 Jojutla (~70 km from epicenter)
Both the main and advance teams visited the town of Jojutla, which was one of the hardest hit areas in the state of Morelos. The municipality of Jojutla, with an estimated population of 57,000 reported the loss of 30 lives. Although official statistics of collapsed structures were not available; satellite image analysis indicate that more than 80% of the downtown commercial district and greater than 50% of the surrounding dwellings suffered complete collapse (UNITAR, 2017; Figure 4.143). On-ground surveys by the GEER team confirm the widespread damage; which although most pronounced to adobe and masonry in-fill residential dwellings, was also pervasive in modern buildings within the commercial district (1980s vintage), where nearly four blocks were completely destroyed.

![Figure 4.143 UNITAR-UNOSAT (2017) satellite damage assessment of the town of Jojutla (image analysis October 3, 2017). Red dots denote predicted severely damaged structures (1,102 damaged structures identified). (top left: 18.5953°, -99.1596°; bottom right: 18.6341°, -99.2022°).](image)

The main team performed a detailed survey of a residential area to the east of the commercial district (Figure 4.144). No geotechnical damage was observed in this area, however, large regions of this neighborhood had been destroyed by the earthquake. It is noted that the collapsed buildings had already been cleaned up by the time the team arrived. This made it difficult to identify the height and number of structures which had been present in this neighborhood before
the earthquake. However, Google Street View images from 2015 (the most recent available) indicate the buildings ranged in height from 1 to 3 stories similar to adjacent blocks which were surveyed (e.g., Figure 4.145-Figure 4.147). Other structures in the survey area performed better. Of the remaining structures, the team observed only one partial collapse and three buildings with major structural damage, all of which were two story structures. In other areas of Jojutla, residential structures suffered much more severe complete lower floor collapse, with the upper floors remaining intact. Structures with such severe damage were of mixed construction, and often with the addition of an upper floor at a later date atop of an older, softer lower story of brittle construction (adobe or unreinforced brick walls, e.g., Figure 4.148).

Figure 4.144 Damage survey in the residential area of Jojutla showing structural and geotechnical damage categories (18.6146°, -99.1754°). The numbers on the buildings indicates the number of stories. The zones marked with “***” represent multiple collapsed buildings which had been cleaned up prior to the survey. It was not possible to confirm the height or exact number of individual buildings in these areas.

Figure 4.145 Complete (left, view A in Figure 4.144) and partial collapse (right, view B in Figure 4.144) of masonry in-fill structures in Jojutla.
Figure 4.146 Two-story masonry building (left, view C in Figure 4.144) which experienced structural damage to the interior walls (right). This damage was not visible from the exterior.

Figure 4.147 Damage to two two-story structures showing moderate (D2, left) and heavy (D3, right) damage (view D in Figure 4.144)
Figure 4.148 Collapsed (pancake) two-story dwelling (left: before imagery, courtesy of Google street view, 2010 and right: after image). The adjacent 3-story building to the left remained in tact. (18.6112°, -99.1820°).

The commercial area in downtown Jojutla experienced major damage. Most of the streets in this area were closed due to safety concerns at the time of the main team’s visit. Due to this, the main team’s survey of the commercial district was limited to a single block along Av. Manuel Altamirano between Francisco Leyva and Zayas Enriques (Figure 4.149). In this block, the team surveyed nine buildings which ranged in height from 1 to 5 stories. All of the buildings were rated as D3 indicating heavy damage with the exception of one two story building which was rated D2 and one two story building which was rated D4 (Figure 4.144). The majority of the buildings in this area appeared to be masonry in-fill construction with non-ductile concrete moment frames (e.g., Figure 4.150). Notably, large story front openings were commonplace within the business district, and lack of ductile detailing at critical structural elements were visible (e.g., Figure 4.151). Locals commented that the majority of this commercial district was constructed in the mid-1980s-early 1990s.
Figure 4.149 Damage to commercial buildings along Av. Manuel Altamirano in the downtown Jojutla commercial district. (18.6149°, 99.1793°).

Figure 4.150 A five story (left) and one story (right) masonry in-fill structure which sustained major damage in the Jojutla commercial district. (18.6149°, 99.1793°)
Figure 4.151 Two-story commercial building with large open entry and heavy upper floor façade. Poorly detailed lower floor columns are evident in the close-up view of the central columns (right). (18.6149°, -99.1793°)

The main team also performed a UAV survey of Parroquia San Miguel Arcangel in Jojutla, built in 1884, it had survived previous earthquakes with little damage. During this event it experienced significant damage and will likely need to be completely torn down in some regions (Figure 4.152-Figure 4.153). A UAV survey was performed of the church to document the damage, as access to the church was prohibited due to its excessive damage (Figure 4.154).

Figure 4.152 Damage to the Parroquia San Miguel Arcangel in Jojutla (18.6122°, -99.1815°).
Figure 4.153 Aerial view of the Jojutla church (immediately after the event)\(^2\) (18.6122°, -99.1815°).

\(^2\) Image courtesy of: https://www.razon.com.mx/temblor-toma-sorpresa-a-morelos/
Further south in Jojutla, the advance team observed approximately 5 to 10 collapsed buildings on either side of the Río Apatlaco to the south of the commercial district (Figure 4.155). A bridge connecting the two sides of the river also suffered significant damage. One of the banks along the river failed, showing signs of ground failure potentially due to loss of strength due to the
seismic load. The bank failure damaged a two-story structure and a significant portion of the sidewalk leading to this structure (Figure 4.155).

![Figure 4.155 Map of Jojutla with heavily affected area by Rio Apatlaco highlighted in red (top) and partially collapsed buildings close to Rio Apatlaco (bottom, 18.6123°, -99.1817°).](image)

### 4.5 Eastern-Central Trek towards Epicenter (State of Puebla)

The state of Puebla is in the highlands of south-central Mexico, and approximately 80-100km south-east of CDMX. UNAM-GEER teams spent several days traveling east of CDMX due to the states proximity to the epicenter (approximately 80-90km north east). Media reports also noted significant impacts due to this event to the communities within the state of Puebla. For example, the Secretaría de la Gobernación of Puebla declared a state of extraordinary emergency in 112 municipalities in the state, which translates in 51% of the state. Puebla’s governor, José A. Galil Fayad, stated on September 20, 2017 that 1,700 houses in Puebla were completely destroyed by the earthquake and had to be demolished (Animal Político, 2017). The majority of the destroyed dwellings are located in the cities of Atlixco, Izúcar de Matamoros and Mixteca. The Secretaría de Educación reported that approximately 213 schools (4%) experienced some level of damage. However, at the time of reporting this statistic, only 46% of the schools had been inspected. More than 350 prisoners from the Penitentiaries of Atlixco and Izúcar de Matamoros in the state of
Puebla had to be relocated due to the damaged suffered in their infrastructure. Survey methods used by the UNAM-GEER teams in the state of Puebla included driving and walking surveys, UAV flights, and seismic testing to document the damage distribution. It is noted that two accelerometers recorded the event, stations SXPU and PHPU, which are located approximately 1.8km and 3.0km west and east of the central plaza in downtown Puebla (Figure 4.156).

![Figure 4.156 Main GEER team tracks through region of Puebla (surveys conducted 10/4 and 10/5/2017)](image)

Event measurements from PHPU and SXPU demonstrate comparable peak acceleration demand, with amplitudes of PGA ranging from 0.12 – 0.14g in both horizontal components (Figure 4.157 and Figure 4.158). Neither directional component (NS nor EW) in either measurement was notably stronger in terms of its peak ground acceleration. Although the PGA, maximum elastic spectral acceleration (Sa,max) and duration of strong shaking Td are comparable amongst this pair of sensors, the long period energy content varies (Table 4.7). For example, highlighted yellow cells in Table 4.7 show that the period at which the maximum spectral acceleration (T@Sa,max) occurs is 0.42s for the eastern most sensor (PHPU), whereas T@Sa,max occurs at between 0.1 – 0.18sec for the westernmost sensor (SXPU). Similarly, the amplitude of Sa(T=0.5sec) is about 2-3 larger for the easternmost sensor (PHPU). In contrast, the western region of the city, with measurements from sensor SXPU report appreciable spectral content particularly for the NS component, at periods between 1.7 ~ 2.2 sec. The later would explain why moderate damage to mid- and high-rise buildings in Santa Fe was observed (west of the plaza) as will be discussed later.
Table 4.7 Summary of measurements of two sensors in the region of Puebla (for locations see Figure 4.156; summary histories and spectra are shown in Figure 4.157 and Figure 4.158, respectively).

| Station | Component | PGA (g) | PGV (cm/s) | PGD (cm) | T6 (s) | T6Sa,max (s) | Sa,max (g) | max AMP | Sa (T=0.2s) (g) | Sa (T=0.5s) (g) | Sa (T=1.0s) (g) | Sa (T=1.5s) (g) | Sa (T=2.0s) (g) |
|---------|-----------|---------|------------|----------|--------|-------------|------------|--------|----------------|----------------|----------------|----------------|----------------|----------------|
| PHPU    | NS        | 0.14    | 8.88       | 2.46     | 24.7   | 0.43        | 0.51       | 3.53   | 0.28           | 0.41           | 0.12           | 0.07           | 0.06           |
|         | EW        | 0.14    | 9.48       | 1.72     | 21.0   | 0.42        | 0.50       | 3.47   | 0.26           | 0.34           | 0.08           | 0.10           | 0.05           |
|         | Vertical  | 0.09    | 4.99       | 1.70     | 21.3   | 0.22        | 0.27       | 3.48   | 0.25           | 0.35           | 0.04           | 0.04           | 0.02           |
| SXPU    | NS        | 0.12    | 17.02      | 5.53     | 16.3   | 0.19        | 0.33       | 2.76   | 0.25           | 0.14           | 0.12           | 0.11           | 0.18           |
|         | EW        | 0.14    | 12.14      | 3.03     | 24.3   | 0.18        | 0.49       | 3.47   | 0.43           | 0.19           | 0.18           | 0.08           | 0.12           |
|         | Vertical  | 0.11    | 7.36       | 2.62     | 24.3   | 0.08        | 0.35       | 3.23   | 0.20           | 0.15           | 0.05           | 0.05           | 0.08           |

Figure 4.157 Acceleration histories of each component of (a – left) SXPU (west of Puebla centro) and (b – right) PHPU (east of Puebla centro)
Figure 4.158 Elastic acceleration, velocity, and displacement response spectra at: (a) station SXPU, ~1.8km west from Puebla city plaza and (b) station PHPU ~3.0km east from Puebla city plaza.

4.5.1 Centro Puebla
The city of Puebla, being the central municipality within the state, has a rich history, being founded in 1531 by the Spanish. Thus, many of the buildings have significant historical context, and are correspondingly constructed of adobe or brick. The most historical and important community icons include a number of churches which were damaged (e.g., Figure 4.160). Most prominent in the city of Puebla, the Puebla Cathedral, constructed between 1575-1690, was moderately damaged during the earthquake. This iconic Cathedral includes a library, temple and church amongst other occupancies. With its tall bell towers, it overlooks the central square of Zocala (Figure 4.159). The team inspected the interior of this Cathedral and spoke with locals about its damage. In addition to damage to the interior hall arch damage (Figure 4.159), the top of the bell tower was damaged, though this was believed to be minor. Many of these churches have withstood harsh environmental conditions for centuries and have undergone very few structural modifications, this cathedral appeared to have no structural modifications.

The main GEER team conducted several walking surveys within a region close to and including the central plaza in downtown Puebla. Buildings within this survey area were primarily 1-2 story,
long block style structures, with multiple occupancies within one long (spanning from block-block) structure. Seismic measurements and UAV surveys were also taken in the plaza and approximately 0.5km east along Calle 6. Within the city plaza the estimated site period was 0.78sec, whereas along Calle 6, the most heavily damaged region observed by the team, the site period was slightly elongated at 0.89sec. Residents believed an old river may have run along Calle 6, which may explain the larger site period at this location.

The resulting UAV model generated from imagery collected in the central plaza of the city of Puebla is shown in Figure 4.161-Figure 4.163, with a plan view of the model, highlighting the most damaged region in the plaza (a two-story building whose roof suffered collapse). This and one other structure in the plaza were damaged, and in each case temporary shoring was in place to support the upper floor which had suffered partial collapse during the earthquake (e.g., Figure 4.164). No notable ground or foundation surface damage were observed within the central plaza.
Figure 4.159 City of Puebla – historic Puebla Cathedral (constructed between 1575-1690) (exterior - top and interior – bottom, damage to arches in church) (19.0449°, -98.1897°)
Figure 4.160 City of Puebla – historic church front of building (entryway-right image) separating from remainder of building (19.0453°, -98.1913°)

Figure 4.161 3D model of Puebla centro plaza using UAV flyover imagery collected on 10/4/2017.
Figure 4.162 Damaged 2 story business on right (tarped roof circled region) (19.0443°, -98.1986°) (plan view from UAV model)

Figure 4.163 Damaged 2 story business on right (tarped roof circled region – isometric view from the UAV model) (19.0443°, -98.1986°)
4.5.2 Calle 6 Norte

A four block region along Calle 6 Norte was believed to be the most heavily damaged region within the central city of Puebla (Figure 4.165). Long single block 2-3 story buildings are characteristic of this area, with multiple businesses and residential dwellings rental units along a single block. Placards on various buildings summarize their historical context (some being several hundreds years old). Damage along Calle 6 Norte manifested in the form of tearing away of the front façade of buildings on the northern side, towards the street. The form of the failure, and its unique extent along this single street, supported the theory that an old river bed may have run along Calle 6 Norte. Various roof-level views documented via UAV survey flights articulate the pull away of the building facades south towards the street (e.g., Figure 4.166-Figure 4.169). On-ground views of the building demonstrate its lean, which extends along nearly the entirety of Calle 6 Norte (to different extents, the most extreme examples shown in Figure 4.170-Figure 4.172).
Figure 4.165 Main GEER team tracks around the city plaza and along Calle 6 Norte (most heavily damaged region outlined in red box, survey conducted on 10/4/2017).

Figure 4.166 3D model of along Calle 6 Norte using UAV flyover imagery collected on 10/4/2017. Note the temporary bracing used to support the central building along this block. (19.0431°, -98.1934°)
Figure 4.167 Plan view of the 3D model of along Calle 6 Norte using UAV flyover imagery collected on 10/4/2017. (zoom in from model shown in Figure 4.166) (19.0431°, -98.1934°)

Figure 4.168 Calle 6 Norte (isometric view of 3D model created with UAV flyover imagery collected on 10/4/2017). (19.0439°, -98.1929°)
Figure 4.169 Calle 6 Norte (plan view of the 3D model created with UAV flyover imagery collected on 10/4/2017). (19.0439°, -98.1929°)

Figure 4.170 On-ground damage photographs of temporary shoring along two-story portion of buildings along Calle 6 Norte (building in plan view of Figure 4.167) (19.0439°, -98.1929°)
Figure 4.171 On-ground damage photographs of temporary shoring along two-story portion of buildings along Calle 6 Norte (building in plan view of Figure 4.169) (19.0422°, -98.1941°)

Figure 4.172 On-ground damage photographs of temporary shoring along two-story portion of buildings along Calle 6 Norte (front elevation views of building in Figure 4.171) (19.0422°, -98.1941°)
4.5.3 Cholula

The town of Cholula is located in the state of Puebla, approximately 60 km northeast of the epicenter. The main team visited the town to document damage to the churches in the city. The GPS track of the team in Cholula is shown in Figure 4.173. The team found little overall damage to residential and commercial structures and no evidence of geotechnical damage. The majority of the structures surveyed were one to two stories with adobe construction. The team surveyed several blocks around the city center and found only three structures which had moderate structural damage (D2, e.g., Figure 4.174). All of the other structures had no damage (D0) or only cosmetic damage (D1). This observation contrasts with the widespread damage observed in some cities in the state of Morelos (e.g., Jojutla) which were a similar distance from the epicenter. The team observed multiple blocks of adobe structures in Cholula which sustained no visible damage (e.g., Figure 4.175).

![Figure 4.173 Map of Cholula showing the tracks of the main GEER team and locations of the surveyed churches and Great Pyramid of Cholula.](image-url)
Figure 4.174 An adobe structure in Cholula (19.0640°, -98.3053°) which sustained damage to both the exterior (left) and interior (right). The steeple of the Parroquia de San Pedro Cholula can be seen in the background (left).

Figure 4.175 Two blocks of primarily adobe structures in Cholula (left: 19.0630°, -98.3033°; right: 19.0612°, -98.3054°) which sustained only cosmetic damage due to the earthquake.

The town of Cholula is one of the oldest in Mexico and has an archeological area near the town center around the Great Pyramid of Cholula (Figure 4.176). The team was unable to inspect the interior of the archaeological area, but did not observe any damage to the exterior of the pyramid on the western edge. At the top of the pyramid stands the church Nuestra Senora de los Remedios. The team was unable to visit the church, but could see that the tops of both bell towers had fallen in the earthquake (Figure 4.177).
Figure 4.176 Views of the western edge of the Great Pyramid of Cholula (19.0580°, -98.3010) which did not appear to sustain damage to its exterior during the earthquake.

Figure 4.177 Damage to the tops of the bell towers at the Iglesia de Nuestra Senora de los Remedios (left; 19.0580°, -98.3018°) and Iglesia de San Miguel Tianguisnahuitl (right; 19.0606°, -98.3032°).

Many other churches are located in Cholula and the team was able to survey the exteriors of six of them (locations in Figure 4.173). These churches were Iglesia de San Miguel Tianguisnahuitl, Convento de San Gabriel (Figure 4.178), Parroquia de San Pedro Cholula (Figure 4.179), Capilla de Santa Cruz Jerusalén (Figure 4.180), Iglesia de San Juan Bautista Calvario (Figure 4.181) and Iglesia de San Pablo Tecamac (Figure 4.182). Damage to the steeples or bell towers was observed at five of the seven churches surveyed (Figure 4.178-Figure 4.182), including the Iglesia de Nuestra Senora de los Remedios. The other two churches had minor damage to portions of the exterior walls, but no visible damage to the bell towers. The team was not able to inspect the interior of any of the churches as all were closed at the time. Local residents informed the team that damage did occur to the interiors of the Parroquia de San Pedro Cholula and Iglesia de San Juan Bautista Calvario, however the extent of this damage is unknown. No damage was noted to the exterior of the domes at any of the churches visited by the team.
Figure 4.178 Photos of damage to the Convento de San Gabriel (19.0623°, -98.3044°). The top of the bell tower had fallen (left) and cracks were observed at several buttress connections (right).

Figure 4.179 The Parroquia de San Pedro Cholula (19.0637°, -98.3065°) sustained damage to the exterior walls (left) and cracking in the bell tower (right) due to the earthquake. Streets around the church were closed due to fear of falling debris. Security guards at the church said the interior sustained some damage, but the extent of this damage is unknown.
Figure 4.180 The Capilla de Santa Cruz Jerusalén (left; 19.0655°, -98.3104°) had minor cracking in the bell tower (right) and some damage to the wall around the main gate which had been repaired. No damage to the exterior of the dome was observed (left).

Figure 4.181 The Iglesia de San Juan Bautista Calvario (19.0667°, -98.3129°) did not appear to sustain damage to the bell tower (left), although some damage to the exterior walls was observed (right). The church was closed at the time due to damage to the interior, but the extent of this damage is not known.
The Iglesia de San Pablo Tecamac (19.0526°, -98.3068°) did not appear to sustain damage to the bell tower (left), but some minor cracking was observed in the exterior walls and the spire on top of the dome had fallen (right).

4.5.4 Atlixco
The town of Atlixco is located in the state of Puebla, approximately 40 km northeast of the epicenter. The main team performed a detailed damage survey of structures near the town center (Figure 4.183). The team surveyed 60 structures of which the majority were 1-3 story adobe buildings. The buildings in Atlixco sustained significantly more damage than those in Cholula with one full (Figure 4.184) and three partial collapses (e.g., Figure 4.185) in the survey area. All of these buildings were adobe construction. Of the surveyed buildings that did not collapse (56), 38% sustained some sort of structural damage that was visible from the exterior. Examples of heavy damage included cracking and collapse of portions of exterior walls (Figure 4.186). Moderate structural damage was observed in one masonry in-fill building near the town square (Figure 4.187). The remaining buildings surveyed had either cosmetic or no visible damage. The municipal building also sustained some damage including cracking of the parapet wall and partial collapse of the clock tower (Figure 4.188).
Figure 4.183 Damage survey around the town center in Atlixco (18.9097°, -98.4346°). The three churches near the town center are also labeled. Viewpoints A through E are shown in subsequent figures.

Figure 4.184 Complete collapse of the second story of an adobe building in Atlixco (view A in Figure 4.183).
Figure 4.185 Partial collapse of a two-story adobe building in Atlixco (view B in Figure 4.183).

Figure 4.186 Heavy damage to a two-story adobe structure (view C in Figure 4.183).

Figure 4.187. Cracking in one of the columns on the fourth floor of a masonry in-fill structure near the town center in Atlixco (view D in Figure 4.183).
Three churches were located in the survey area (Figure 4.183) and the team documented damage to the exteriors of each. The churches are the Conjunto Conventual de Santa Clara (Figure 4.189), Parroquia de Santa María de la Natividad (Figure 4.190) and Iglesia de San Agustín. All three churches sustained damage including cracking of the exterior walls and two of the three had damage to the bell towers. The most severe damage was observed at Parroquia de Santa María de la Natividad, which had a large crack along the edge of the west wall. The spires for the bell tower and above the door had also fallen. The interiors of the churches were not surveyed.
4.5.4 Santa Fe
Continuing southwest at the outskirts of the Puebla City, modern mid- and high-rise buildings sprinkle the region. The team conducted several surveys of these areas to assess the impact of the event on more modern construction. One such more detailed survey areas was the area of Santa Fe, a small affluent community approximately 85km northeast from the epicenter, and 8km southwest from the central plaza in the city of Puebla. Within Santa Fe, a street Casiopea was surveyed due to the wide range of building heights observed in this multi-family residential community (Figure 4.191). The team conducted walking surveys along this region and note that a group of three 22-story buildings, 12 story buildings, and various low-rise buildings (Figure 4.192). Site seismic tests were also conducted, however, they were inconclusive. Nonetheless, visible damage was observed, primarily in the mid-rise (12 story) and high-rise (22-story) buildings. This damage was being painted over at the time of the teams visit therefore the survey was conducted rapidly. The team was also not allowed access to any of the buildings; and security attendants informed the team that occupants were allowed back into the dwellings 1-2 days after the event. The high-rise buildings realized a pattern of shear cracking within the wall-bracing along the height of each building (Figure 4.193), which appeared wide enough to categorize the damage as D3; while the mid-rise (12-story) buildings appeared to suffer rocking induced damage at the wall-ends as apparent from compression cracking along floor lines (Figure 4.194); the damage to this structure may be estimated as D2-D3. Neighboring buildings under construction indicate that the underlying building skeleton of these modern construction was likely of concrete moment frame, with brick infill (Figure 4.195). The contrasting performance of this modern constructed area highlights the importance of this event and in particular the vulnerability of modern construction methods under continued use.

*Figure 4.190 Damage to Parroquia de Santa María de la Natividad (left; 18.9116°, -98.4334°) included significant cracking in the exterior walls and the toppling of two spires. Damage to Iglesia de San Agustin (right; 18.9077°, -98.4352°) was limited to cracking at the tops of the exterior walls.*
Figure 4.191 Map of main GEER team survey area in Santa Fe. (19.0158°, -98.2582°)
Figure 4.192 Santa Fe, group of three 22-story buildings (central buildings); 12-story buildings (far buildings); and 3-story buildings (appeared undamaged). The 22-story buildings suffered moderate damage in the form of shear cracking of wall piers along their height, the 12-story buildings suffered rocking induced damage at wall ends, while the 3-story buildings appeared undamaged. (19.0158°, -98.2582°)
Figure 4.193 Distributed shear cracking up the height of 22-story building at Casiopea 4002 (image on bottom is a zoom-in of elevation image on top) (19.0158°, -98.2582°)
Figure 4.194 Distribution of floor-line cracking along 12-story building at Casiopea 4004 (right image is a zoom in of left image) (19.0150°, -98.2587°)
Figure 4.195 Neighboring buildings under construction (left – bare skeleton; and right – finished structure, each within same complex). (19.0158°, -98.2582°)
References


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5. Observations of Slope Instability

5.1 General Rockfall and Landslide Observations
Several instances of slope instability were investigated as part of the reconnaissance efforts. Relatively moderate slope stability problems were observed within the boundaries of Mexico City (CDMX). The main and advance teams investigated one site located near Xochimilco and several more in the state of Morelos to the south of Mexico City. Limited slope instabilities were reported near the epicenter (e.g., Mount La Malinche slide), which caused no visible damage. The locations and descriptions for each of the sites investigated by the team are shown in Figure 5.1 and Table 5.1. In addition to the observed instances of slope instability, some activity of the Popocatepetl Volcano was noted immediately after the earthquake as summarized at the end of this chapter.

The impact of the observed landslides on the communities varied significantly. The slide in Xochimilco closed a major roadway in southern CDMX leading to traffic jams and large detours. The slide also caused damage to water lines, retaining walls, roadways, sidewalks and buildings. Landslides in more rural areas affected smaller populations, but also damaged buildings and roadways. The damage to the Yautepec bridge in Morelos led to a 5 km detour for the residents of the town of Estacas. The landslides in Totolapan and Tlayacapan did not cause major damage to homes, but left behind unstable slopes which may pose a greater risk to residents in the future. Not all of the impacts were negative and, in the case of Atlatlahucan, the slides actually loosened material which made mining activities easier. The direct impacts of the increased activity of Popocatepetl has been minimal and the damage near the base of the volcano has been attributed to the earthquake shaking rather than volcanic activity. Further details on these observations are discussed in the following sections.

<table>
<thead>
<tr>
<th>Name of City/Region</th>
<th>Approx. Location</th>
<th>Type of Instability</th>
<th>Date(s) Visited (2017)</th>
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<tr>
<td>Xochimilco</td>
<td>19.2467, -99.0872</td>
<td>Landslide/Subsidence</td>
<td>9/25, 9/26, 10/2</td>
</tr>
<tr>
<td>Tlayacapan</td>
<td>18.9486, -98.9837</td>
<td>Rockfall</td>
<td>9/27, 10/2</td>
</tr>
<tr>
<td>Totolapan</td>
<td>18.9816, -98.9246</td>
<td>Slope failure in old quarry</td>
<td>9/27, 10/2, 10/3</td>
</tr>
<tr>
<td>Atlatlahucan</td>
<td>18.9378, -98.8784</td>
<td>Series of landslides</td>
<td>9/27, 10/3</td>
</tr>
<tr>
<td>Yautepec River</td>
<td>18.7307, -99.1194</td>
<td>Slope failure near bridge</td>
<td>10/3, 10/4</td>
</tr>
<tr>
<td>Rio Apatlaco</td>
<td>18.6123, -99.1817</td>
<td>Failure of river bank</td>
<td>9/27</td>
</tr>
<tr>
<td>Lake Tequesquitengo</td>
<td>18.6313, -99.2538</td>
<td>Movement along lake shore</td>
<td>9/27</td>
</tr>
<tr>
<td>Tejalpa-Zacatepec</td>
<td>18.7146, -99.1835</td>
<td>Embankment failure</td>
<td>9/27</td>
</tr>
</tbody>
</table>
5.2 Xochimilco Slope Instability (19.2467°, -99.0872°)
The UNAM-GEER advance and main teams visited an unstable slope that had developed in the hills of Xochimilco (Figure 5.2), a municipality in southern CDMX. The site is located in seismic zone III-a (RCDF 2004), is densely populated, and the slope has a maximum inclination of 33%. Local residents informed the main team that the slope had been moving slowly for many years and attributed the movement to the CDMX water well that was located at the toe of the slope. In early 2017, a void had opened in Carret Xochimilco-Tulyehualco near the base of the slope which was repaired in March 2017. The cause of this void is unknown. The 2017 Puebla-Mexico City earthquake appeared to accelerate the slope movement and caused rupture of several water lines to the west of the slope (Figure 5.2), significant damage to the pavement, partial collapse of multiple retaining walls and rotation of several light poles. The earthquake occurred at the end of the rainy season (with average precipitation of 230 mm/month) and runoff from this rainfall led to erosion in the areas of damaged pavement. Some rainfall had occurred in the time between the main team and advance team’s visit, but it was not apparent how much additional movement, if any, had occurred.
The maximum slope displacements were located just south of the water pumping station, at the intersection of Av. Cocoxochitl and Vieja Xochimilco-Tulyehualco (19.2469,-99.0871). In this area, the asphalt cracks had 20 cm of vertical offset and 4 cm of lateral displacement (Figure 5.3 view A). This surface crack extends for at least 250 m along Vieja Xochimilco-Tulyehualco (Figure 5.3, view B) and ran parallel to a system of cracks observed in Av. Acueducto. A second system of cracks was located further west along Vieja Xochimilco-Tulyehualco and Ave. Las Flores (Figure 5.4 and 5.5). Multiple sections of the retaining walls in this area had failed including one that damaged a public library (Figure 5.6, view F). A local resident showed the main team a second area of movement to the west of the main slope instability. While no large surface cracks were visible, there was movement which led to the rupture of three water lines running approximately parallel to the western edge of the slope (Figure 5.2). The resident informed the team that this same area had moved during the 1985 earthquake and had continued to move ever since.

There was additional evidence of slope movement and subsidence which did not appear to be related to the earthquake. This includes the repaired void in Carret Xochimilco-Tulyehualco discussed previously. The team also noticed that 15 cm diameter concrete piles were previously placed behind the retaining wall along Carret Xochimilco-Tulyehualco in an apparent effort to mitigate the effects of the slope creep. The team observed significant subsidence in the park to the west of water pumping station (Figure 5.6, view G). Settlement in this area was concentrated in narrow bands and measured as much as 30 cm. The established vegetation in this area indicated this settlement occurred prior to the earthquake. These observations suggested to the team that there was more of a global stability problem in this area, likely due to subsidence, which may have been exasperated by the earthquake.

\[\text{Figure 5.2. Xochimilco slope instability showing locations of pumping station and previously repaired void. (19.2471°, -99.0882°).}\]
Figure 5.3. Left: Damaged area of Av. Las Flores (view B in Figure 5.2; 19.2470°, -99.0874°). Cracks in the retaining wall to the right had been painted pink. Right: Large residual displacements in asphalt pavement (view A in Figure 5.2; 19.2467°, -99.0870°).

Figure 5.4. Left: Pre-existing cracks in the Av. Las Flores just above a failed retaining wall (view C in Figure 5.2; 19.2468°, -99.0898°). Right: Lateral cracks and void had developed in Av. Las Flores (view C in Figure 5.2; 19.2470°, -99.0895°).
Figure 5.5. Left: Retaining wall supported with wooden cross braces in Vieja Xochimilco-Tulyehualco (View D in Figure 5.2; 19.2470°, -99.0897°). Center: Failed section of retaining wall in Vieja Xochimilco-Tulyehualco (view D in Figure 5.2; 19.2470°, -99.0895°). Right: Failed retaining wall and lamp post with 3 degrees tilt (view E in Figure 5.2; 19.2474°, -99.0893°).

Figure 5.6. Left: Damage to library windows due to failed retaining wall (view F in Figure 5.2; 19.2470°, -99.0897°). Right: Pre-existing cracking and subsidence in park next to water pumping station (view G in Figure 5.2; 19.2474°, -99.0879°).

5.3 Tlayacapan Rockslide (18.9486°, -98.9837°)
A significant rockfall occurred in the state of Morelos on the outskirts of the town of Tlayacapan (Figure 5.7). The rockfall occurred when a large portion of the ridge above Tlayacapan dislodged during the earthquake. Geologic maps of the area show that this ridge is composed primarily of sedimentary breccia which was consistent with the observations of the team. The advance team performed a UAV flight at this location and the main team hiked up the path created by the boulders (Figure 5.8). The homes at the base of the rockfall were occupied at the time of the reconnaissance. The local residents had placed gravel under the edge of several large
boulders in an attempt to prevent further movement. The rock face itself appeared to still be unstable and large cracks were visible in the face when the main team reached the top of the slope (Figure 5.9).

The boulders from the rockfall traveled approximately 240 m down the slope, which had an average grade of 41% (Figure 5.8). The size of the boulders varied significantly with the largest being several meters in diameter. Flattened trees were visible all along the slope. The velocity of the boulders appeared to have been slowed by the dense vegetation, the very soft soil conditions along the slope and by large boulders apparently left behind by previous rockfalls. Large impact craters were visible at several points along the slope and the soil in these areas was very soft and wet with an organic odor (Figure 5.10). The boulders appeared to both break into small pieces and change direction after impacting remnants from previous rockfalls (Figure 5.11). The majority of the boulders were stopped before reaching the occupied houses along the roadway beneath the slope. Only damage to the one residential house was observed and this structure was unoccupied and under construction at the time (Figure 5.12). The approximate dimensions of this boulder were 230 cm x 250 cm x 270 cm. It left a hole in the CMU wall that measured 130 cm high by 200 cm wide.

*Figure 5.7. Photographs from main highway (left) and UAV (right) (18.9486°, -98.9837°)*
Figure 5.8. Top: Location and profile of rockfall slide. Viewpoint A shows the location of Figure 5.9. Bottom left: “Before” view from Google Earth. Bottom right: Current view after earthquake with a circle indicating the mass which had fallen during the earthquake (18.9486°, -98.9837°).
Figure 5.9. Visible cracks in the rock face at the top of rockfall (view A in Figure 5.8; 18.9486°, -98.9859°).

Figure 5.10. Imprint left in the soft soil by a large boulder (18.4986°, -98.9846°). The houses along the street can be seen in the background.
Figure 5.11 Boulders from the recent rockfall were broken up and redirected upon impacting a large boulder apparently from a previous rockfall (18.9486°, -98.9842°).

Figure 5.12 Damage to a residential building under construction by massive boulder (~15 m$^3$) from rockfall (18.9488°, -98.9840°)

5.4 Totolapan Slope Failure (18.9816°, -98.9246°)
A slope failure was observed in the walls of an old quarry which is currently being used as a landfill in the city of Totolapan. This site was investigated by both the main and advance teams. Local residents informed the team that the quarry was used to mine lava rock (Tezontle) in the past. The walls of the quarry were very steep and possibly undercut in locations before the earthquake (Figure 5.13). During the earthquake, the sides of the quarry failed in multiple locations and soil from these failures covered approximately half of the landfill (Figure 5.13). UAV photos showed substantial cracking along the east side of the quarry and near some homes constructed above the quarry (Figure 5.14). These homes were occupied at the time of
the reconnaissance. During the advance team’s visit, a moderate size slope failure occurred along the walls of the quarry. Given the steep quarry walls and the cracks, additional instability is anticipated at this location.

Figure 5.13 Left: photo of the abandoned Tezontle quarry and current landfill taken before the earthquake (Google Street View from February 2017). Right: The same area after the earthquake (18.9816°, -98.9246°).

Figure 5.14 Photographs showing failed walls of the former lava rock quarry after the earthquake (18.9816°, -98.9246°).
5.5 Atlatlahucan Landslides (18.9378°, -98.8784°)
Over six “landslides” were observed by the advance and main teams along the hills that run parallel to Route 115, to the SE, over a distance of about 2 km (Figure 5.15). The advance team learned that these slides were associated with old mining quarries based on Google Earth images from 2011 – 2017. This was confirmed by the main team which observed mining activities at multiple “landslide” locations. Excavators and dump trucks were actively removing material (soil and rock) from the site (Figure 5.16). According to local workers, the area has been mined for construction materials for over 20 years by various companies. Natural slides do occur and provide accessible loose material for mining. These areas have been expanded by the mining operations of the various companies. The workers informed the main team that additional sliding and rock falls had occurred during the earthquake, but that no one was injured. The advanced team observations using “before and after” imagery also indicated potential for additional slide movement at all individual mines. In addition to individual slides, substantial cracking of the surface at the top plateau of the landslides was observed. Vegetation regrowth was observed along some of the scarps and slide areas indicating these areas last moved prior to the earthquake. The main team and advance teams performed UAV flights of the area to better observe cracking/scarps along the plateau/ridge and understand the full extent of the slides (Figure 5.17).
Figure 5.15 Top: Extent of the observed Atlatlahuca landslide. Bottom: Individual slides are indicated by yellow arrows (18.9409°, -98.8709°).
Figure 5.16 Top: Mixed soil/rock slide area with construction road and excavator for mining purposes (18.9283°, -98.8801°). Bottom: Rock outcrop with excavator (18.9439°, -98.8669°).
Figure 5.17 Aerial UAV images of several of the Atlatlahucan landslides and cracking on top of the plateau (18.9409°, -98.8709°)
5.6 Other observations of instability

5.6.1 Yautepec River

While traveling to Jojutla from Tetela de Volcán, the main team discovered a damaged bridge over the Yautepec River, near the towns of Estacas and Yautepec (Figure 5.18). A circular sliding failure occurred adjacent to the bridge near the west abutment in a clayey material. The slide extended about 40 meters (135 feet) south of the abutment and encompassed a gabion basket retaining wall constructed along the river bank (Figures 5.19 and 5.20). The slide scarp and multiple tension cracks extending behind the slide were observed. The slide also led to damage of the west roadway approach and southwest wing wall of the bridge, as well as the rotation of a roadside power pole. Soil underneath the bridge deck in front of the abutment experience vertical settlement of about 0.45 meters (1.5 feet). The main team collected terrestrial lidar data, UAV imagery and performed HVSR measurements at this location. The bridge and roadway were under repair at the time of the main team’s visit. Documentation of the bridge damage is provided in Chapter 6 of this report.

Figure 5.18 Location of damaged bridge and landslide at the Yautepec River (18.7307°, -99.1194°)
Figure 5.19 Point cloud data created using structure from motion (SfM) from UAV imagery (18.7306°, -99.1194°) showing the extent of the landslide, west side of the bridge and gabion basket wall (units m).

Figure 5.20 Left: Scarp of landslide with gabion basket wall and river to the left (18.7306°, -99.1195°). Right: Looking back at gabion basket wall and Yauatepec River from the east side of the bridge. Note the bulging and build-up of soil at the base of the wall (18.7307°, -99.1193°).

5.6.2 Río Apatlaco
One of the banks along the river failed in the southern part of Jojutla, showing signs of ground failure potentially due to loss of strength due to the seismic load. The bank failure damaged a two story structure and a significant portion of the sidewalk leading to this structure (Figure 5.21). The bridge connecting the two banks suffered significant damage. Additional discussion of the structural damage in Jojutla is included in Chapter 4 of this report.
Figure 5.21 Top: Map of Jojutla with heavily affected area by Río Apatlaco highlighted in red. Middle: Partially collapsed buildings close to Río Apatlaco (18.6123°, -99.1817°). Bottom left: structure with failed first story and second story fallen on top (18.6113°, -99.1821°). Bottom Right: Apatlaco River bank failure and affected structures (18.6117°, -99.1830°). Refer to Chapter 4 for additional documentation of damage in Jojutla.
5.6.3 Lake Tequesquitengo
Failure and lateral displacement of the lake bank was observed along the northeast shore of Lake Tequesquitengo. This led to large cracks and significant displacement of the roadway (Figure 5.22). The side of the road closer to the lake settled about 0.61 meters (2 feet), and its horizontal displacement was about 0.3 meters (1 foot). The length of the failure, as measured from the cracks on the pavement surface, was approximately 52 meters (170 feet).

Figure 5.22 Top left: Cracks along pavement on Lake Tequesquitengo. Top right: Vertical displacement of road. Bottom: Large cracks opened on pavement \(18.6313^\circ, -99.2538^\circ\).
5.6.4 Embankment failure in Tejalpa-Zacatepec Highway
A portion of an embankment partially supported by a retaining wall suffered significant damage, which manifested as medium to large cracks on the pavement for a length of 21 meters (70 feet) (Figure 5.23). The failure occurred on the section of the embankment that was not supported by the retaining wall, indicating a failure due to suboptimal design. The failed portion of the road was closed due to the damages.

![Figure 5.23 Significant deformations along pavement due to embankment deformation (18.7146°, -99.1835°).](image)

5.7 Popocatépetl Volcano (19.0224°, -98.6279°)
Popocatépetl, meaning “smokey mountain”, but also called El Popo or Don Goyo by locals, is an active volcano with a peak elevation of 5,426 meters (17,797 feet). The volcano has erupted more than 15 times since 1519, but has been more active over the last 23 years (since 1994). Volcanic activity on November 20, 2017 is now the most recent, however, the volcano also released a low ash content plume shortly after the September 19, 2017 earthquake (Global Volcanism Program [GVP], 2017c). The volcano has been highly active (Figure 5.24) since the earthquake, with as many as 10 explosions from September 26-30, and numerous others, including almost daily emissions through November 20. Volcanic activity on and after September 19 is not believed to be linked to the earthquake (GVP, 2017a-d).

A local resident of Puebla (about 61 km [38 miles] east of El Popo) informed the main team that the eruption and earthquake led to the initiation of rockfall and landslides around the volcano. There were no deaths reported in the nearest town of San Antonio, however, about 80% of the buildings were destroyed or extensively damaged there, including many adobe homes, an elementary school, and a church constructed in 1781 (LA Times, 2017). It was reported that 15 people died in Atzitzihuacán at the foot of El Popo when a church collapsed during the earthquake (Forbes, 2017).
Figure 5.24 Top: Ash spewing from Popocatépetl Volcano. Middle: Night time eruption on October 1, 2017. Bottom: El Popo erupting on November 10, 2017 (19.0224°, -98.6279°).
References


6. Effects of Event on Lifelines and Critical Infrastructure

6.1 General Observations
This chapter summarizes the extent of damage to lifelines and critical infrastructure as observed and evaluated by the Advance and Main GEER teams. Lifelines (defined here as bridges, roads, pipelines, utilities, and evacuation routes) that experienced damage during the September 19 2017 earthquake were surveyed using a combination of walking inspection, low altitude imagery collected using UAVs, and on-ground LiDAR. When possible, mapping of damage was evaluated using the Ground Failure and Structural Damage Indices presented in Chapter 4, which was used for buildings. Although in general terms lifelines performed well during the 2017 Puebla-Mexico City earthquake, several cases are worth discussing. It is also noted that the performance of airports is included in this chapter, as they are a component of critical infrastructure.

6.2 Performance of Pipelines and Utilities
Water and power utilities were significantly affected during this event. In terms of water utilities, media reports claim that over 6 million users lost potable water services after the earthquake (Excelsior, 2017a). A total of 64 municipalities were directly affected by this, with the states of Puebla and Morelos being the most affected, with 30 and 20 municipalities, respectively, being affected. In CDMX, five regions, predominantly Iztapala and Tlalpan, were severely hit by the loss of water, with a total of more than 2,300 leaks in the distribution network. The State Water Commission (CEAGUA) of the state of Morelos reported 8 dams, 50 water reservoirs, 39 wells, 27 water treatment plants and 20 waterways were somewhat damaged.

In terms of the electrical power grid, the Federal Commission of Power (CFE) reestablished almost entirely the electricity infrastructure to all affected customers within the first two weeks following the earthquake. CFE reported that their service to 33% of its customers (representing 5 million people) were affected by the earthquake in the state of Mexico City, Morelos, Puebla, Oaxaca, Gurrero, and Tlaxcala. Within CDMX, a total of 1.8 million customers lost power (Noticieros Televisa, 2017). In totality, over 900 electricity posts, 750 power transformers, and 19 high power tension lines were severely affected.

Figure 6.1 shows the shaking intensity map of Mexico City in terms of spectral accelerations at periods of 0, 0.3, 1, and 2 sec. Figure 6.2 shows the approximate distribution of damage to major fresh water pipelines, which occurred as a result of strong shaking or permanent ground movements. The majority of damage to pipelines was concentrated in the Colonia Del Mar area in Mexico City, with widespread evidence of surface cracks, terrain settlement (and at times lateral displacement) crossing several city blocks (Figure 6.3). The general settlement patterns and ground subsidence in this area had been noticed over a long period prior to the earthquake. However, ground shaking amplified the magnitude of vertical and horizontal ground deformations, likely due to sharp variations in the thickness of soft clay deposits underlying these neighborhoods (transitioning sediments between the lake soils and rocky soils close to the hills in the south). Such large differential settlements (in the order of 1 to 3 feet)
caused damage to sewage and fresh water pipelines (e.g., along the Langosta and Carmen streets and others).

Figure 6.4 - Figure 6.6 show typical cases of pipeline failure in the Del Mar area of CDMX due to excessive ground deformations. Large sewage pipelines (5 to 6m in diameter) and fresh water pipelines (up to 1m in diameter) broke in several sections either through shear or compression along parallel streets that were separated by several blocks. The depression bands crossed streets and city blocks, affecting the pipelines in this area.

By the time the UNAM and GEER teams arrived on the ground, many of the damaged pipelines had already been repaired or being repaired, as shown in Figure 6.7. Many of these same pipes were previously damaged and repaired after the 1985 earthquake with a stiff steel section. This method of repair is known to increase the pipe’s compressional stiffness and hence, vulnerability to compression and damage in subsequent earthquake events. As expected and predicted by O’Rourke and Liu (1985), these repaired pipes were damaged again in 2017, and subsequently repaired with the same method. The GEER team expects to gain access to more detailed data on the performance and damage of pipelines across Mexico City from the Department of Water, which will be added to the report in future versions of this document.

Figure 6.1 Shaking intensity map of Mexico City in terms of spectral accelerations at periods of 0, 0.3, 1, and 2 sec (courtesy of Instituto de Ingenieria UNAM).
Figure 6.2 Approximate distribution of damage to structures and major fresh water pipelines in Mexico City (courtesy of Instituto de Ingeniería UNAM).

Figure 6.3 Google map zoomed into the Colonia Del Mar area in Mexico City where extensive damage to pipelines was observed.
Figure 6.4 Photographs of pipeline damage common in the Colonia Del Mar area in Mexico City due to excessive ground deformations (courtesy of Professor Gustavo Ayala Milian of UNAM, approximate coordinate for pictures: 19.2837°, -99.0568°)
Figure 6.5 Photographs of pipeline damage and type of repair common in the Colonia Del Mar area in Mexico City (courtesy of Professor Gustavo Ayala Milian of UNAM, approximate coordinate for pictures: 19.2837°, -99.0568°)
Figure 6.6 Photographs of pipeline damage due to excessive ground deformations and repair efforts common in the Colonia Del Mar area in Mexico City (courtesy of Professor Gustavo Ayala Milian of UNAM, approximate coordinate for pictures: 19.2837°, -99.0568°)
6.3 Performance of Transportation System and Bridges

6.3.1 Overall Transportation Network Impact
According to Secretaria de Comunicaciones y Transportes (SCT), severe highway infrastructure damage was reported on bridges on Oaxtepec-Cualuta highway, Amacameca-La Alborada highway, and Mexico-Aapulco highway (La Razon, 2017). Minor embankment failures in the Puebla-Oaxaca highway near Leon-Huajuapan and Huajuapan-Nochixitan and Santa Barbara highway in Izucar de Matamoros were also immediately reported after the event (La Razon, 2017). Six out of the 195 metro stations (Tezonco, Olivos, Npalera, Zapotitlan, Tlaltenco, and Tlahuac), suffered structural damage and were out of service immediately after the earthquake. They were estimated to be out of service for at least four weeks after the event (Excelsior, 2017b).

6.3.2 Specific Bridge Performance Observed by UNAM-GEER Team
More than 70% of the bridges in Mexico were constructed before 1970 without any seismic design (Landa-Ruiz, 2008). Moreover, no bridge retrofit program for these old bridges has occurred since then to address their vulnerability. In contrast, most new bridges are designed using the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications (BDS), which includes some seismic design criteria. Moreover, they also
have to comply with local codes such as the Normas Técnicas Complementarias para Diseño por Sismo (Complementary Technical Standards for Earthquake Design) (NTS), which is part of the Mexico City Building Code or with the CFE (Civil Engineering) Seismic Design Manual.

In general, bridges performed well during the September 19th 2017 Puebla-Morelos earthquake with a few cases of damaged and collapsed bridges reported. Only a few cases of damage associated with seat displacement observed (e.g., Metro Viaduct in CDMX). Despite the overall good bridge performance that was observed, some bridge damage and even bridge collapse did occur. The advance and main GEER teams visited several bridge locations. Locations and descriptions for each of the sites investigated by the reconnaissance teams are shown in Figure 6.8 and Table 6.2. Short descriptions of the observed damage are included in the following sections.

Figure 6.8 Map showing location of damaged bridges visited by the GEER teams
Table 6.2. Observations of bridge damage by the UNAM-GEER team

<table>
<thead>
<tr>
<th>Structure Name/Type</th>
<th>Approx. Location</th>
<th>Type of Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circuito Interior Avenida Rio Churubusco</td>
<td>19.2913, -99.1105</td>
<td>Parallel box girder viaducts on pier walls rocked</td>
</tr>
<tr>
<td>Metro Elevated Train Viaduct (Site1)</td>
<td>19.2982, -99.0339</td>
<td>Damage to ‘S’ shaped section</td>
</tr>
<tr>
<td>Metro Elevated Train Viaduct (Site2)</td>
<td>19.3017, -99.0520</td>
<td>Severe Damage at Column Base</td>
</tr>
<tr>
<td>Rio Yauatepec Bridge</td>
<td>18.7306, -99.1193</td>
<td>Collapsed abutments</td>
</tr>
<tr>
<td>Cuernavaca Bridge</td>
<td>18.7624, -99.2344</td>
<td>Complete deck collapse</td>
</tr>
</tbody>
</table>

This is a pedestrian overcrossing crossing Boulevard Adolfo Ruiz Cortines in Mexico City. The bridge collapsed during the earthquake, unfortunately falling onto a taxi (Figure 6.9).

![Figure 6.9](image1.jpg)

Figure 6.9 Left: Collapsed POC across Blvd. Adolfo Ruiz Cortines (19.2913°, -99.1105°), Right: POC (taken from Google Earth) shown before the earthquake (19.2913°, -99.1105°).

Circuito Interior Avenida Rio Churubusco (19.2913°, -99.1105°)
Collaborator Prof. Miranda of Stanford shared photos of the Circuito Interior Avenida Rio Churubusco (19.3696°, -99.1225°) in Mexico City. This five span undercrossing supported on pier walls appeared to have rocked during the earthquake (the pier walls were too stiff to bend) and an abutment constructed of unreinforced masonry with a rock facing was damaged (Figure 6.10). The GEER team subsequently visited this site and felt that some of this damage may have been existing, such as the hardscape damage at the interior bent foundations, which offer the impression that the bends rocked during the event, though this hardscape damage was certainly exacerbated due to the earthquake. Other bends along the undercrossing had no notable hardscape damage and thus left no evidence of rocking (Figure 6.11). During the UNAM-GEER team visit, notable damage was evident in the rockwall of the Northwest abutment (lower image shown in Figure 6.12). This on-ramp deck appeared to have moved at its abutment support (upper image show in Figure 6.12).

![Figure 6.10](image2.jpg)

![Figure 6.11](image3.jpg)

![Figure 6.12](image4.jpg)
Figure 6.10 Damage to the Circuito Interior Avenida Rio Churubusco (19.2913°, -99.1105°). Photographs courtesy of Prof Eduardo Miranda, Stanford University.
Figure 6.11 Bents directly adjacent to those shown in Figure 6.10 (no surficial damage as was evident in the lower left of above Figure were observed). (19.3692°, -99.1230°). Others along the length of this undercrossing were in similar good condition.
Figure 6.12 Northwest abutment onramp at the Circuito Interior Avenida Rio Churubusco (19.3690°, -99.1232°)
Metro Elevated Train Viaduct (19.2982°, -99.0339°) and (19.3017°, -99.0520°)

Two damage locations were reported for the Metro Train Viaduct. Site one showed damage at the superstructure (pounding) and ground cracking due to column rocking (floating foundations), while at site two, a support column was severely damaged. Both sites are summarized below.

Site 1: Damage to ‘S’ shaped section of viaduct: 19.2982°, -99.0339°

In this site damage was observed in a portion of the viaduct superstructure composed of simply supported steel girders supporting a concrete deck. The wide deck carries two sets of tracks and an occasional train station. The substructure is 7 ft in diameter, single column bents and big hammerhead bent caps, supported on hollow floating foundations to protect the structure from sinking (Figure 6.13). The columns have a 5-inch notch for a drainage pipe that prevents rainwater from accumulating on the roadway. This detail is considered poor since it requires at least 5 inch of concrete cover to provide increased column stiffness. Support spacing varies around an average of 60 ft.

Severe cracking in the road indicated the columns rocking during the earthquake (see Figure 6.14). Rocking in this case may have helped protect the columns from more serious damage during an earthquake. Shear Key Damage was observed on the west side of Juan de Dios Peza (19.2982°, -99.0339°), as well as damage to the bent caps (Figure 6.15). In addition, minor damage cracks and spalls from the spans banging together at expansion joints was noticed (19.2982°, -99.0339°).

No interruption of traffic and Metro Service, engineering assessment of road cracking and superstructure damage underway during visit. Given the unique foundation system of the bridge pier (Floating foundations), it would be of interest to further evaluate the foundation for potential foundation damage. Furthermore, the available amount of bridge seat at the ends of the girders would be of interest to assess if the girders have any restraint (or enough seat) left to prevent them from falling.
Figure 6.13 Photo showing ‘S’ shaped viaduct (19.2982°, -99.0339°)

Figure 6.14 Roadway cracks as a result of column rocking during the earthquake (19.2982°, -99.0339°)
Figure 6.15 Damage to the bent cap due to shear plate movement during the earthquake (left) and spalling at expansion joints (right) (19.2982°, -99.0339°)

Site 2: Severe Damage at Column Base on west side of Amado Nervo (19.3017°, -99.0520°)
In this site cracks and spalls, observed at the base of the column, exposed the reinforcement, removed the cover concrete and penetrated deeply into the core (Figure 6.16). The main reinforcement was coupled in the plastic hinge zone, which meant it had very little strain capacity. Almost no transverse reinforcement was visible at the column, isolated hoops were loosely wrapped around the column.

The bridge was supported by a large steel frame until repair measures had been identified (Figure 6.17). Local information hinted similar construction issues were anticipated in the adjacent columns of the bridge segment. X-ray measurements are anticipated to ensure this column is an anomaly and not typical along the viaduct.

Figure 6.16 Cracks and spalls at column base on west side of Amado Nervo (19.3017°, -99.0520°)
Figure 6.17 Cracks extending into the column core (shown on top) and superstructure shoring with tubular steel frame sections (shown on bottom). (19.3017°, -99.0520°).

Rio Yautepec Bridge (18.7307°, -99.1194°)
This bridge located near the towns of Estacas and Yautepec in the state of Morelos just 110 km from the earthquake epicenter, is a single span structure crossing the Yautepec river in the East-West direction (Figure 6.18). The bridge is commonly used by heavy trucks transporting crops. The bridge superstructure is composed of five reinforced concrete girders supporting a continuous concrete slab. The seat abutments are composed of precast abutment walls and stone and cement wing-walls. Although the construction date is unknown, locals indicate the bridge was built 20 years ago and was retrofitted four years ago when a concrete skirt and protective structure was added to the west embankment and a new carpet installed (see Figure 6.19).
Soils in the vicinity of the bridge are soft clays and fine silts. The meandering Yautepec river overflows regularly flooding large extensions of land. Newspapers and news releases indicate the zone has suffered extensive flooding damage over the years. The water table is elevated and soils are saturated. Due to its characteristics, this land is actively used for crops. The river banks have been dredged along the river and in several have been protected with gabion wall structures.

*Figure 6.19 Yautepec River Bridge: Schematic of observed damage*
During the Sept 19 2017 Puebla-Morelos earthquake the bridge and surrounding soils suffered stability failure with consequent large deformations. A schematic of the observed damage is shown in Figure 6.19 and Figure 6.20 shows photos of the bridge before and after the Morelos-Puebla earthquake event. The river bank on the west and east sides suffered rotational slope stability failures. The bridge abutments and rock-cement wing-walls worked as expected reducing substantially the bridge displacements and settlements. Nevertheless, damage at both ends was important and resulting in the closing of the bridge to traffic.

![Bridge before earthquake](image1)

![Bridge after earthquake](image2)

Figure 6.20 Yautepec River Bridge (a)Before the Puebla-Morelos Earthquake (from Google Street View) and (b) After the event (photo from the west side) (18.7307°, -99.1194°)

**West Abutment**

The west abutment suffered severe damage with estimated vertical settlements of 0.5 m and lateral deformations near 1.0 m. Figure 6.21(a) shows the west bank slope failure mechanism. A typical deep seated circular failure mechanism is observed in the Figure with visible scarps defining moving blocks and estimated lateral displacements of 4.0 - 5.0 m. A gabion wall (marked in red in the figure) prevented larger deformations and possibly the collapse of the bridge. Figure 6.21(b) shows the response of the abutment wall with clear structural rocking and lateral displacements (and a peculiar curved shape of the wall).

![Abutment damage](image3)

Figure 6.21 Yautepec bridge West abutment damage: (a) Circular slope stability failure, (b) Settlements and lateral deformation of bridge abutment - (18.7306°, -99.1194°)
East Abutment
The east abutment also suffered damage and river bank also showed indication of rotational slope stability failure but smaller settlements and lateral displacement in the north-west direction almost parallel to the river bank. The failure zone extended almost 70 m towards east with an estimated settlement of 10 cm (Figure 6.22a). The wind-wall completely detached form the embankment and moved laterally in the same direction as the surrounding soils but did not collapse (Figure 6.22b).

![East Abutment Damage](image)

*Figure 6.22 Yautepec bridge East abutment damage: (a) Extension of affect soils, (b) East abutment damage (18.7307°, -99.1194°)*

The GEER main team collected terrestrial LiDAR data, UAV imagery and performed HVSR measurements at this location. The HVSR measurements were made on the west abutment and a site period of 0.83 seconds was estimated for the area. Figure 6.23 shows a 3D reconstruction of the West abutment obtained using structure-from-motion photogrammetric imaging techniques. The Figure shows the concrete skirt built four years ago to protect the abutment. At the time the main team visited the site local officials were evaluating damage, compacting the embankment soils on the west side and evaluating global stability of the area. Figure 6.24 shows the compacted material on the west side. It was expected to open the bridge to traffic in a few weeks.
Figure 6.23 3D Image of Yautepec bridge West abutment. The figure clearly shows the observed damage to the soil and structure. It also shows the presence of a concrete skirt recently built to protect the structure (18.7307°, -99.1194°).

Figure 6.24 Yautepec bridge repair construction work (18.7307°, -99.1194°).

Highway-95D Cuernavaca bridge (18.7624°, -99.2345°)
This highway bridge on Mexico City-Acapulco Highway 95D, south of Cuernavaca in the state of Morelos collapsed during September 19, 2017 Mw 7.1 earthquake, as shown in Figure 6.25. Unfortunately, the bridge was demolished at the time of the main GEER team visit on October 4, 2017. So, all perishable information was lost. The team observed that the site conditions were composed of low plasticity sandy silt, and it seems one of the bridge abutments gave up with the consequent collapse of the bridge deck. A local video of the collapse a few minutes after the event became viral and can be viewed at: https://www.youtube.com/watch?v=4MnZ0dm7CvE.
6.25 (a) Cuernavaca Bridge collapsed during the September 19, 2017 Mw 7.1 earthquake. 
(b) Bridge was demolished at time of GEER team reconnaissance (18.7627°, -99.2342°) 

6.4 Performance of Airports 
At least 180 flights into and out of the City of Mexico were affected after the earthquake. The airport runways and tarmac did not suffer structural damage or ground failure, however an access ramp at terminal 2 suffered a ground crack which promoted the closing of this area for rapid repairs (Figure 6.26). The airport was closed for only a few hours (Excelsior, 2017c). 

6.4.1 New Aeropuerto Internacional de Ciudad de Mexico (NAICM) 
The UNAM-GEER advance team was specifically requested to investigate the performance of the New Aeropuerto Internacional de Ciudad de México (NAICM) construction site. This multi-billion dollar project is located in the northeastern portion of Mexico City on top of very soft
and deep Texcoco Lake clays. Figure 6.27 shows an approximate site vicinity map of the NAICM site and its relative location to Mexico City.

At the time of the UNAM-GEER team site visit at NAICM, the reinforced concrete slab for the main terminal was being constructed (Figure 6.28). The slab consists of 1.6 meter thick concrete heavily reinforced with #12 high strength steel rebar. The slab had already settled 8 cm at its center, causing it to pond water from the rain the night before the team visit (see the visible water on the slab in Figure 6.28). The slab is connected by reinforced grade beams to various driven friction piles across the site. The team observed that many of the piles located around the perimeter of the site were bent out of plum, as shown in Figure 6.29. The site foreman informed the team that these piles were out already tilted before the earthquake, however, the earthquake made them worse.

![Figure 6.27 Site vicinity map showing the approximate location of the NAICM airport project in white](image-url)
Figure 6.28  UAV image of the NAICM terminal construction site at the time of the UNAM-GEER team site visit (19.5052°, -98.9967°)

Figure 6.29  Tilted driven pile near the perimeter of the excavation for the NAICM terminal construction
Flights with the UAV around the perimeter of the 5 m deep excavation showed numerous small tension cracks forming in places. Many of these cracks had apparently been in place before the September 19th 2017 earthquake, but the earthquake widened existing cracks and also precipitated new cracks. An example of these observed cracks are shown in Figure 6.30.

![Figure 6.30 Tension crack observed in the NAICM excavation from a UAV flown at an altitude of 50 meters; driven pile is visible towards the left of the image (19.5050°, -98.9957°)](image)

The final part of the NAICM construction project that was visited by the UNAM-GEER advance team was the construction site for the new control tower. At the time of the team visit, the foundation was under construction, and no superstructure was yet in place. The foundation design consisted of friction piles combined with a parabolic reinforced mat foundation. The system will eventually support the base-isolated control tower for the NAICM. An aerial image of the control foundation tower being constructed is presented in Figure 6.31.
Figure 6.31  UAV image of the construction of the new control tower foundation at NAICM (19.5223°, -98.9971°)

References


7. Advanced Survey and Testing Methods

This chapter summarizes the advanced survey technologies and seismic testing methods used in the field by the UNAM-GEER team. Data derived using this equipment is embedded in various sections of this report; therefore the focus of this chapter is to offer interested readers an understanding of the equipment, data processing, and dissemination methods used. It is noted that the NHERI-DesignSafe Data Depot is being utilized to disseminate data from this reconnaissance (PRJ-1690, 2017).

7.1 Terrestrial and Aerial Surveys

A broad range of terrestrial and aerial imaging surveys were conducted, collecting perishable data following the 2017 Puebla-Mexico City Earthquake. The UNAM-GEER reconnaissance, Advance team members from UC Irvine and BYU employed two unmanned aerial vehicles (UAVs), and Main team members from UC San Diego employed a terrestrial LiDAR system as well as two UAVs for aerial imaging. LiDAR (Figure 7.1) was utilized primarily to obtain groundtruth models of interior and exterior regions associated with observed damage patterns (buildings, ground features, etc.), while the UAVs (Figure 7.2) were used to obtain comprehensive, high-resolution aerial images in the visible range. UAV-collected imagery was also used to 1) generate ortho rectified images for damage maps and 2) to build 3D models of scenes for future analysis. The deployed survey tools are comparatively low cost and lightweight, and therefore ideal for a mobile reconnaissance. UAVs have emerged in recent years as a promising tool for capturing post-event damage patterns (e.g., Meyer et al., 2016).

Figure 7.1. LiDAR scanning in Colonia Del Mar neighborhood (19.2853°, -99.0577°)
Figure 7.2. UAV returning from a flight over the church in Jojutla (18.6122°, -99.1815°).

7.1.1 Sites Surveyed
UAV-based, low-altitude surveys were conducted at 23 different sites, during both the advance and main team survey days. In addition, LiDAR data was collected at five strategically selected sites in the Colonia del Mar area, which suffered from extensive pre-event subsidence-induced and event-induced ground settlement. The initial set of data assets includes 3D point-clouds for each LiDAR survey site as well as hundreds to thousands of photos for each UAV imaging site. In total, more than 10,000 photos were taken using low altitude UAV cameras, resulting in the generation of 260GB of image data. Table 7.1 summarizes the collected data assets per site.

Table 7.1 Summary of UAV and LiDAR sites surveyed by the UNAM-US GEER advance and main teams (note that some sites were surveyed by both teams; since the coverage was different, these are counted as different sites below, white filled boxes are those surveyed by the advance team; the grey boxes are those surveyed by the main team).

<table>
<thead>
<tr>
<th>Imaging Site</th>
<th>Date</th>
<th>Location [coordinates]</th>
<th>Raw data size [GB]</th>
<th>UAV [#photos]</th>
<th>LiDAR [#scans]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1        Escocia 4 (advance team)</td>
<td>9/25/2017</td>
<td>19.3877, -99.1637</td>
<td>0.13</td>
<td>23</td>
<td>-</td>
</tr>
<tr>
<td>2        Enrique Rebsamen and La Morena (advance team)</td>
<td>9/25/2017</td>
<td>19.3984, -99.1587</td>
<td>0.1</td>
<td>20</td>
<td>-</td>
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<tr>
<td>No.</td>
<td>Location Description</td>
<td>Date</td>
<td>Latitude/Longitude</td>
<td>Magnitude</td>
<td>Damage</td>
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<td>-----</td>
<td>---------------------</td>
<td>--------------</td>
<td>-----------------------</td>
<td>-----------</td>
<td>--------</td>
</tr>
<tr>
<td>3</td>
<td>La Condesa / Hipodromo</td>
<td>9/26/2017</td>
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<td>7.7</td>
<td>552</td>
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<tr>
<td>4</td>
<td>Xochimilco Slope</td>
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</tr>
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<td>Enrique Rebsamen and La Morena (main team)</td>
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<td>34.5</td>
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<tr>
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<td>Col. del Mar Case 4</td>
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<td>1.58</td>
<td>-</td>
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<td>19.2887,-99.0633</td>
<td>2.18</td>
<td>-</td>
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<td>28.9</td>
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<td>7.85</td>
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<td>Bridge Río Yautepec</td>
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<td>Atlatlahucan Landslide</td>
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<td>Downtown Jojutla</td>
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<td>18.6149,-99.1764</td>
<td>3.28</td>
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<td>Parroquia San Miguel Arcángel Jojutla</td>
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<td>22</td>
<td>Parroquia San Miguel Arcángel Tlaltizapán</td>
<td>10/3/2017</td>
<td>18.6830,-99.1196</td>
<td>4.53</td>
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<td>23</td>
<td>Parroquia Santo Domingo de Guzmán</td>
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<td>3.48</td>
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<td>24</td>
<td>Zócalo de Puebla</td>
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<td>25</td>
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<td>19.0433,-98.1935</td>
<td>13.4</td>
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<tr>
<td>26</td>
<td>Izuar de Matamoros (4 churches)</td>
<td>10/5/2017</td>
<td>18.6072,-98.4641</td>
<td>10.0</td>
<td>214</td>
</tr>
</tbody>
</table>

| Total | 260.15 | 10138 | 21 |
7.1.2 Equipment Specifications
Imaging equipment included a FARO FOCUS 3D LiDAR to capture localized features visible from the ground, such as cracks in facades, ground failures, or structural deformations. This LiDAR (Figure 7.1) has a theoretical range of 120m and accuracy of +/-2mm. It has an onboard camera for collecting images that can be texture mapped onto the point cloud during processing. The model used in these surveys collects about one million points per second and weighs 11 lbs. With a battery life of about 5 hours, it proved ideal for mobile field efforts. In this effort, when not all features were visible from one viewpoint, multiple overlapping scans were acquired and combined. The quadrotor UAVs included multiple DJI Phantom 3 Professional and DJI Phantom 4 Professional models, which were used to acquire images from vantage points which would be difficult or impossible to otherwise access. At weights below 1.5kg each, and a diagonal size of 350mm, the copters were ideal for traveling from site to site in a backpack, while providing 12 megapixel and 20 megapixel camera, respectively, in image resolution. The UAVs’ internal GPS was used for the initial georeferencing of the 3D point clouds and orthomosaics generated from structure from motion processing.

7.1.3 UAV Data Collection Strategy
Based on site characteristics, including location, scale, complexity, as well as desired level of detail, manual, semi-autonomous or fully autonomous flights were conducted, targeting overlap of around 75% overlap between photos. A total of 10,138 photos were acquired, ranging between 23 to 1,217 photos per site, with most sites captured with at least 200 photos. For the main team aerial flight imagery, each photo collection was subsequently processed using structure from motion (SfM) techniques, to create 3D site models, capturing geometry and setting in respect to the surrounding environment. To acquire the imagery for the 3D reconstruction, the copters were flown in a lawnmower pattern with the camera pointed downwards, with line spacing selected to be able to capture oblique views of both sides of any structures in the survey area. In the case of particularly tall structures or structures with damage most evident from an oblique view, the copters were flown in either an orbit or parallel to the surface with the camera pointed outwards and slightly down.

7.2 Model Creation and Synthesis
7.2.1 LiDAR-Based Models
LiDAR models were processed using FARO SCENE 7.0 (Faro, 2017). SCENE was used to filter and apply color to the point-cloud, as well as align multiple scan viewpoints to create a more comprehensive model of the scanned area. Point clouds were then exported for visualization and analysis using UC San Diego’s point-based visual analytics environment called Viscore (Petrovic et al., 2015), which is capable of interactively rendering billions of points. Viscore also supports the creation of derivative data assets such as an orthorectified image mosaics and high fidelity, video flythroughs. A large suite of orthorectified images were created to support creation of damage maps (particularly those shown in Chapter 4).
7.2.2 UAV-Based Models
Once acquired, the UAV-based images were converted into 3D models using SfM techniques, including commercially available tools such as Agisoft Photoscan 1.3.3 (Agisoft, 2017) and Bentley ContextCapture (Bentley, 2017). Data was initially processed at significantly reduced resolution on field laptops to ensure comprehensive coverage of the target area, and subsequently processed at full resolution on a computer cluster. Through this workflow, dense 3D point clouds were generated to represent the 3D geometry of the site, and high resolution orthomosaics were created to provide comprehensive views of the survey area.

7.3 Derivative Data Assets
Original as well as derivative data assets are accessible via NHERI’s DesignSafe (PRJ-1690, 2017), UC San Diego’s DroneLab data-repository1, and BYU’s PRISM Lab model gallery2, and may be accessed via these links. Data assets include image collections, interactive 3D point-cloud models, video fly-throughs as well as orthomosaics. Reference naming schemes are intuitively linked to the site names summarized in Table 7.1.

Interactive 3D Point-Cloud Models
- atlalahucan.html
- bridge.html
- calle6nte.html
- coldelmar-ext.html
- coldelmar.html
- condesa.html
- downtown-jojutla.html
- escocia.html
- hospital-tlahuac.html
- izucar-de-matamoros.html
- parroquia-jojutla.html
- parroquia-santodomingo.html
- parroquia-tlaltizapan.html
- paseodelasgalias-ext.html
- paseodelasgalias.html
- rebsamen.html
- zocalo-puebla.html

Video flythroughs
- atlalahucan.mp4
- bridge.mp4
- calle6nte.mp4
- catedral-de-puebla.mp4

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1 http://chei.ucsd.edu/selected-projects/2017-puebla-mexico-city-earthquake/
2 http://prismweb.groups.et.byu.net/gallery2/
7.4 In-Situ Methods, Equipment, and Locations of Testing (MASW, MAM, and HVSR)

7.4.1 Introduction
This section provides a summary of the testing location for in-situ dynamic measurements taken during the UNAM-GEER Team reconnaissance following the September 19th Puebla-Mexico City earthquake and details the specific methodology used to collect and process Multi-Channel Analysis of Surface Waves (MASW), Microtremor Array Measurements (MAM), and horizontal-to-vertical spectral ratio (HVSR) data collected as part of this study.
### 7.4.2 Sites Investigated

MASW, MAM, and HVSR measurements were taken at a total of 25 sites by the Main Team (University of Arkansas, Uark) and 18 sites by the advanced team (Universidad de Concepción and Pontificia Universidad Católica de Chile, UdeC-PUC) throughout the area affected by the Mw 7.1 Puebla-Mexico City earthquake. Table 7.2 summarizes the location of each test site and indicates the type of test performed.

**Table 7.2 MASW, MAM, and HVSR testing locations (University of Arkansas, Uark and Pontificia Universidad Católica de Chile, UdeC-PUC) and type of testing performed.**

<table>
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<tr>
<th>Site Name</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Active MASW</th>
<th>Passive MAM</th>
<th>HVSR</th>
<th>Collector</th>
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### 7.5 Specific Techniques and Equipment Used

MASW testing was typically conducted using a linear array of 24, 4.5 Hz vertical geophones with a uniform spacing between geophones of 2 meters (shorter receiver spacings were necessary at some site due to space limitations). Rayleigh type surface waves were generated using a 4 kg sledgehammer with source locations of 5, 10, and 20 meters from the first geophone in the array. Multiple source offsets were used to insure high quality data, allow uncertainty to be estimated, and ensure near-field effects do not corrupt the data. The surface wave propagation was recorded using a geometrics Geode seismograph. At each source location 10 sledgehammer blows were stacked in order to increase the signal to noise ratio of the recorded signals. A typically MASW field setup is shown in Figure 7.3. The active source data was analyzed using the frequency domain beamformer (FDBF) method (Zywicki 1999) coupled with the multiple source-offset technique for identifying near-field contamination and quantifying dispersion uncertainty (Cox and Wood 2011). Dispersion data were generated from each source-offset location (i.e., 5, 10, and 20 meters), and the maximum spectral peak for each
frequency was picked automatically using a Matlab code. The individual dispersion data from each source offset were combined to form a composite dispersion curve. After eliminating clear near-field data, the composite experimental dispersion curve was divided into 100 frequency bins spaced equally between 1 to 100 Hz on a log scale. The mean phase velocity and associated standard deviation were then calculated for each bin, resulting in an experimental dispersion curve with associated uncertainty for each frequency.

2D MAM was conducted using an L-shaped array of 24, 4.5 Hz vertical geophones with a spacing of between 4-5m between geophones depending on the available space at each particular site. Passive energy from urban noise or environmental energy from wind or other sources was utilized for the testing. The surface wave propagation was recorded using a geometrics Geode seismograph. A total of 30, 60 second records were recorded at each site for a total recording time of 30 minutes. A typically MAM field setup is shown in Figure 7.4. The passive L-array data (i.e., microtremor array measurements; MAM) were analyzed using the 2D frequency-wavenumber method as programed in the Geopsy software\(^3\). For the f-k method, the vertical vibrations recorded with the L-arrays were used to compute the Rayleigh wave.

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\(^3\) http://www.geopsy.org
dispersion data. Dispersion uncertainty was estimated by processing the time records in 60 second blocks. The phase velocities obtained from each block were used to obtain a mean and standard deviation at each frequency. The Rayleigh wave velocity at each frequency was computed by selecting the x and y wavenumber pair that resulted in the peak output power in the 2D wavenumber spectrum. The composite experimental dispersion curve was divided into 100 frequency bins equally spaced between 1 to 30 Hz on a log scale. Similar to the active source data, the mean phase velocity and associated standard deviation were calculated for each bin.

![Figure 7.4 Photograph showing typical passive MAM setup for testing.](image)

HVSR data was collected at selected sites using a Nanometrics Trillium Compact boardband three component seismometers. The sensor has a flat response from 100 Hz-20 sec. Waveforms were recorded using a Nanometrics Centuar three component digitizer. Passive energy was recorded for between 15-120 minutes. A typical HVSR field setup is shown in Figure 7.5. Data analysis was performed in Geopsy using the squared average of the north-south and east-west horizontal components. The time records were divided into 120 second blocks for processing. Time windows with excessive noise were rejected, and the remaining time windows were used to create a spectral average representing the response of each array. Data was processed in general accordance with guidelines developed by the SESAME project (SESAME 2004).
Once the Rayleigh wave dispersion estimates were obtained and the HVSR peak was determined (if observed), an inversion was performed using either the multi-modal joint inversion in Geopsy in combination with the effective mode inversion in WinSASW. WinSASW was used exclusively for sites which had a high velocity layer at the surface and velocity inversion at depth. These sites could not be inverted using the Geopsy software due to problems with the inversions calculations. For the Geopsy inversions, the Rayleigh dispersion data and HVSR peak were jointly inverted. Layer interfaces were only loosely constrained and allowed to find any appropriate depth. If layering information from invasive tests were available, these were used to constrain/inform the layering of the site. The shear wave velocities of each layer were constrained within reasonable velocities for geotechnical materials, which were defined as a function of material type and confining pressure. Poisson’s ratio was allowed to vary between 0.25-0.50 except where the location of the water table was known in which case the P-wave velocity was fixed at 1500 m/s. The density was varied for each layer from 1600-2000 kg/m3 depending on the estimated Vs of the layer. Multiple inversion analyses were conducted for each site to insure parameterization choices did not negatively influence the results Vs profile. The neighborhood algorithm in Geopsy (Wathelet 2008) was allowed to run with 1 million models searched in each analysis. The relative quality of fit was
quantified by the minimum misfit, which compares the theoretical data to the experimental data in terms of the collective squared error.

For dispersion data inverted in combination with WinSASW, Rayleigh dispersion data was fit by eye using the 2D theoretical solution. Poisson’s ratio was set to 0.3 except where the location of the water table was known in which case the P-wave velocity was fixed at 1500 m/s. The density of each layer was based on the suspected material type and shear wave velocity. The shear wave velocity profiles obtained from the inversions for each site were limited to the maximum experimental wavelength divided by two (i.e., lmax/2).

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