

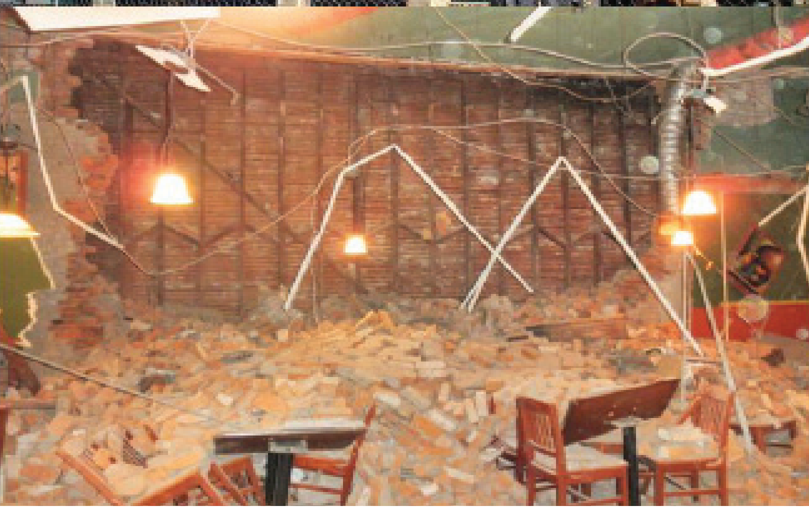


Performance of Buildings and Nonstructural Components in the 2014 South Napa Earthquake

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Preface

California is subject to frequent damaging earthquakes, and each one presents an opportunity to study the impacts, improve our understanding of how buildings perform when subjected to strong ground shaking, and update building codes and standards for improved building performance. The Federal Emergency Management Agency (FEMA) established the Mitigation Assessment Team (MAT) program to investigate post-disaster building performance and develop recommendations that address improvements in building design and construction, code development, enforcement, and mitigation activities that will lead to greater resistance to hazard events. The FEMA MAT program, however, is not currently set up to investigate the performance of buildings after earthquakes.

On August 24, 2014, a magnitude-6.0 earthquake occurred in Napa, California. In response to this earthquake, the Special Projects task of the National Earthquake Technical Assistance Program (NETAP) under FEMA Contract HSFE60-12-D-024 with the Applied Technology Council (ATC) was used to fund an investigation. At the time, this event had not yet been declared a federal disaster, and disaster funds were therefore not available. Because of limitations to this funding, some issues, such as performance of lifelines or building investigations in additional areas, could not be investigated.

Past earthquakes in California have resulted in significant improvements to national and local building codes, including the 1933 Long Beach earthquake (which affected schools and unreinforced masonry structures), the 1971 San Fernando earthquake (which affected hospitals and non-ductile concrete structures), the 1989 Loma Prieta earthquake (which affected soft story wood light-frame construction), and the 1994 Northridge earthquake (which affected steel moment frame structures). For the 2014 South Napa earthquake, work was focused on documenting the observed performance of buildings and nonstructural components in order to lead into future improvements in future building codes, and to do so within six months.

ATC is indebted to the vision of Michael Mahoney (FEMA Project Officer), and leadership of John Gillengerten and Maryann Phipps who served as Co-Project Technical Directors and principal authors for this work. Contributing authors Kelly Cobeen, Bret Lizundia, Joseph Maffei, Joshua Marrow, and Bill Tremayne assisted in the development of postearthquake observations and conclusions. Veronica Crothers, Sarah Durphy, Jonas Houston, Alix Kottke, Chiara McKenney, Karl Telleen, and Noelle Yuen were instrumental in the completion of field investigative work within the limited time available. The Project Review Panel, consisting of Dan Kavarian, Roy Lobo, Khalid Mosalam, Marko Schotanus, and Fred Turner, provided technical review of the report. The names and affiliations of all who participated on the project team are provided in the list of Project Participants at the end of this publication.

ATC gratefully acknowledges information provided by Brad Harward, Shawn Huff, Bruce Maison, Terrence Paret, Chris Tokas, Mark Quattrocchi, and Richard Weinert, who generously shared their observations with the report authors, and the residents of Napa who provided valuable information and assistance during the many visits to the region.

ATC also gratefully acknowledges Ayse Hortacsu for ATC project management, Veronica Heintz and Jack Lakes for assistance in compiling the field data, and Amber Houchen for ATC report production services.

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Chapter 1
Introduction

The magnitude-6 South Napa earthquake occurred on August 24, 2014 with an epicenter located 8 km (5 miles) south southwest of the City of Napa. The cities of Napa and Vallejo, as well as the surrounding areas, were significantly impacted by the event. The earthquake struck at 3:20 in the morning, which was the primary reason for only one fatality and the low number of serious injuries. Had the earthquake struck 12 hours earlier, during a street festival in downtown Napa, the number of fatalities could have easily been in the hundreds due to falling debris from masonry buildings and nonstructural components.

The earthquake was recorded by eight strong-motion recording instruments at which horizontal ground motions were significant (exceeded 0.1g), and three of those instruments were located within the City of Napa. Availability of recorded ground motions and documentation of the impact of the earthquake provides an excellent opportunity to calibrate and evaluate existing earthquake hazard reduction methodologies. This is also an opportunity to expand existing knowledge and databases on the performance of buildings and other structures, including seismically retrofitted unreinforced masonry (URM) buildings.

1.1 Project Goal

The goal of the project was to assess and document the performance of a population of buildings impacted by the earthquake and develop a series of recommendations to further improve mitigation. The building stock in Napa consisted of mostly older buildings, many of which had previously been seismically retrofitted. This project focused on the performance of seismic retrofitting, particularly for URM buildings, and the performance of nonstructural components, since they were responsible for the vast majority of the damage and injuries. The performance of buildings designed in accordance with recent building codes was also investigated. It is envisioned that data from this project will eventually be used to assist in the validation of two new assessment methodologies documented in recently released FEMA documents, FEMA P-58, *Seismic Performance Assessment of Buildings* (FEMA, 2012b) and the third edition of FEMA P-154, *Rapid*

Visual Screening of Buildings for Potential Seismic Hazards: A Handbook, (FEMA, 2015).

1.2 Investigation Process

It is common to focus postearthquake reconnaissance on observations of damage. A major focus of this project was performance, both bad and good. In order to make it possible to correlate the relationships between ground shaking severity and the performance of building, this project collected information on all buildings located in the vicinity of a strong-motion recording instrument with the purpose of relating damage observations back to an input ground motion.

Postearthquake information was also collected and documented by the following organizations: The Earthquake Engineering Research Institute (EERI), the Geotechnical Extreme Events Reconnaissance (GEER) Association, the Pacific Earthquake Engineering Research (PEER) Center, the Post-Disaster Performance Evaluation Program (PDPOC) of the Structural Engineers Association of California (SEAOC), and the Technical Committee on Lifeline Earthquake Engineering (TCLEE) of the American Society of Civil Engineers (ASCE).

1.2.1 Survey Process

Most of the buildings discussed in this report were evaluated using the methodology developed in the ATC-38 report, *Database on the Performance of Structures near Strong-Motion Recordings: 1994 Northridge, California, Earthquake*, (ATC, 2000). This methodology is also the basis for the forms used by the Post-Disaster Performance Evaluation Program (PDPOC) of the Structural Engineers Association of California (SEAOC) in their data collection efforts.

The ATC-38 methodology was developed following the 1994 Northridge earthquake to systematically collect and analyze data from buildings located in the vicinity of strong-motion recording instruments. The methodology calls for designating one or more specific strong-motion instrument sites and investigating every building within a 1,000 foot radius of that instrument using a six-page postearthquake building performance assessment form. The forms were created to collect data including the structure size, age, location, structural framing system and other important structural characteristics, nonstructural systems and performance, fatalities and injuries, and estimated time to restore the facility to pre-earthquake usability. Information on postearthquake damage evaluations and placarding is also collected.

To collect data on buildings impacted by the 2014 South Napa earthquake, the forms developed and used for the Northridge earthquake were adopted for use with minor modifications to account for new knowledge and the specific issues related to this event. Modifications included the addition of the "minor" damage state, collection of general classification information for nonstructural damage, expanding opportunities for sketching building plans and elevations, describing irregularities in terms consistent with FEMA P-154, and expanding the scope of nonstructural systems and components examined. More information on the modification and a sample data collection form used in this project are provided in Appendix A.

1.2.2 Scope of Investigation

This investigation primarily focused on collection of data for all buildings within a 1,000 foot radius around the strong-motion recording instrument at Station N016 located on Main Street in Napa operated by the Northern California Seismic Network (NCSN) of the U.S. Geological Survey (USGS). This station was identified as the top priority for systematic data collection for the following reasons: (1) the level of ground shaking in the area was sufficient to cause damage to some buildings; (2) the area contains a large number of commercial and civic buildings, both historic and modern; and (3) there are several retrofitted unreinforced masonry buildings in the selected area. The source-to-site distance for Station N016 was 3.9 km, the peak ground acceleration (PGA) was 0.61g (north-south), 0.32g (east-west), and 0.24g (vertical) (GEER, 2015). It is recognized that although the location of the instrument adjacent to the Napa River may have contributed to the PGA values being higher than the other two instruments in Napa, the soils in Napa Valley are consistently alluvial in nature, resulting in fairly consistent ground motions within the study area.

The investigation was limited to a single site largely as a consequence of limited resources. However, reconnaissance work included collection of data for select buildings outside the 1,000 foot radius in order to more comprehensively examine the nature and scope of building performance in the earthquake. Specifically, the study investigated the performance of residential construction, manufactured housing, modern commercial buildings, healthcare facilities, and schools. In addition, given their economic importance to the region, wineries were also examined.

This investigation was limited to buildings and did not include infrastructure. There were many lifelines issues, including loss of power, but loss of water supply had the largest impact on building performance. The earthquake ruptured over 120 water mains, and at least one of these breaks impacted firefighting at one mobile home park, where a fire in one unit resulted in the loss of five units due to a delay in providing water to the scene. For information on the performance of lifelines in the South Napa earthquake, refer to *PEER Preliminary Notes and Observations on the August 24, 2014 South Napa Earthquake* (PEER, 2014) and *South Napa M 6.0 Earthquake of August 24, 2014* report (ASCE, 2014).

1.2.3 Site Investigations

Site investigations within the 1,000 foot radius were conducted on September 5, 8 and 9, 2014, approximately two weeks after the earthquake. Each investigation was conducted with a team of not fewer than two engineers, one of whom was a registered Structural Engineer in the State of California. On average, each team spent between approximately 1 and 1.5 hours investigating and documenting each building, using modified ATC-38 forms. The level of detail for each building investigation varied based on a number of factors including access to the interior of the building (buildings posted UNSAFE generally could not be entered), security (entry was not permitted or was limited in some buildings), and the availability and willingness of a building representative to facilitate and supplement observations. For a subset of buildings, drawings were made available for viewing by either the building owner or the City of Napa Building Division.

The database containing the information collected will be published by the Applied Technology Council.

Selected buildings outside the 1,000 radius were also studied. In most cases, building data were collected using ATC-38 forms. In some cases, such as healthcare facilities, schools, residential construction, manufactured housing, and wineries, information was collected throughout the affected area, during a period of several weeks after the earthquake, and without completing the ATC-38 forms. As with others, investigations varied in their level of detail and included a combination of observations, conversations with building owners and tenants, and review of drawings.

Photos in the report not individually acknowledged as originating from outside sources were provided by the Principal and Contributing Authors, Project Working Group members, and participating ATC staff.

1.3 Report Organization

This report describes the performance of structural and nonstructural systems in the 2014 South Napa earthquake. Chapter 2 describes the ground motion and regional seismicity. Chapter 3 provides a summary of the building survey data based on the information collected using the ATC-38 forms on the 68 buildings within the 1,000 foot radius. Chapter 4 provides a summary of the performance of selected buildings within and outside of the 1,000 ft radius and recommendations. Chapter 5 provides a detailed summary of the performance of healthcare facilities. Chapter 6 provides an overview of the performance of school facilities within the cities of Napa and Vallejo. Chapter 7 provides an overview of the performance of residential construction, including observations of surface rupture and afterslip. Chapter 8 provides a background on seismic requirements for manufactured housing and the performance of units in eleven mobile home parks near the epicenter. Chapter 9 provides an overview of the performance of wine industry facilities affected. Chapter 10 provides an overview of the performance of nonstructural elements including glazing, cladding, interior partitions, mechanical, electrical, and plumbing equipment, piping systems, contents, and solar arrays. Chapter 11 summarizes the response for postearthquake safety evaluation of buildings and observed placards. Chapter 12 summarizes the available resources and observed barricading of unsafe areas. Chapter 13 summarizes the recommendations.

Appendices provide additional information on the subject. Appendix A presents the survey forms and instructions used and Appendix B provides two Recovery Advisories developed in response to the damage sustained in the 2014 South Napa earthquake: the first is on cripple wall foundations and a summary is provided here because it was under development when this report was published; the second is on masonry chimneys and is provided in its entirety.

Chapter 2
Seismicity and Ground Motion Data

This chapter presents information regarding seismicity and ground motion in the 2014 South Napa earthquake. Much of the information in this chapter was taken from two publications: (1) National Science Foundation-supported GEER Report 037, G*eotechnical Engineering Reconnaissance of the August 24, 2014 M6 South Napa Earthquake*, (GEER, 2015); and (2) *Key Recovery Factors for the August 24, 2014, South Napa Earthquake*, (USGS, 2014). The reader is referred to both of those reports for a more detailed description.

2.1 West Napa Fault and Surrounding Seismicity

The region impacted by the South Napa earthquake is encompassed by the Bay Area San Andreas Fault System, which forms the boundary between the Pacific and North American tectonic plates. It is depicted by the U.S. Geological Survey to have a high probability of strong shaking in the future, as shown in Figure 2-1. Specifically, the area has a 63% probability of experiencing a magnitude 6.7 or greater earthquake in the next 30 years. These data are further broken down by each specific fault, and the Rogers Creek/Hayward fault system has a 31% chance of causing such an earthquake and the Green Valley/Concord fault system has a 3% chance of causing such an earthquake. The USGS map does not list the West Napa fault as contributing to this probability.

The August 24, 2014 South Napa earthquake occurred on the West Napa fault, which lies within a 70 km (44 miles) wide set of faults that make up the San Andreas fault system. The West Napa fault lies between the larger Rogers Creek and Green Valley faults and begins at American Canyon at the south end of Napa Valley where it meets San Pablo Bay and extends to the north-northwest along the west side of Napa Valley (Figure 2-2).

Napa Valley is a typical California Coastal Range valley situated between low lying mountain ranges and filled with 160 meters of older Pleistocene alluvial deposits overlaid by 10 meters of more recent Holocene alluvial deposits, and is equivalent to Site Class D soils.

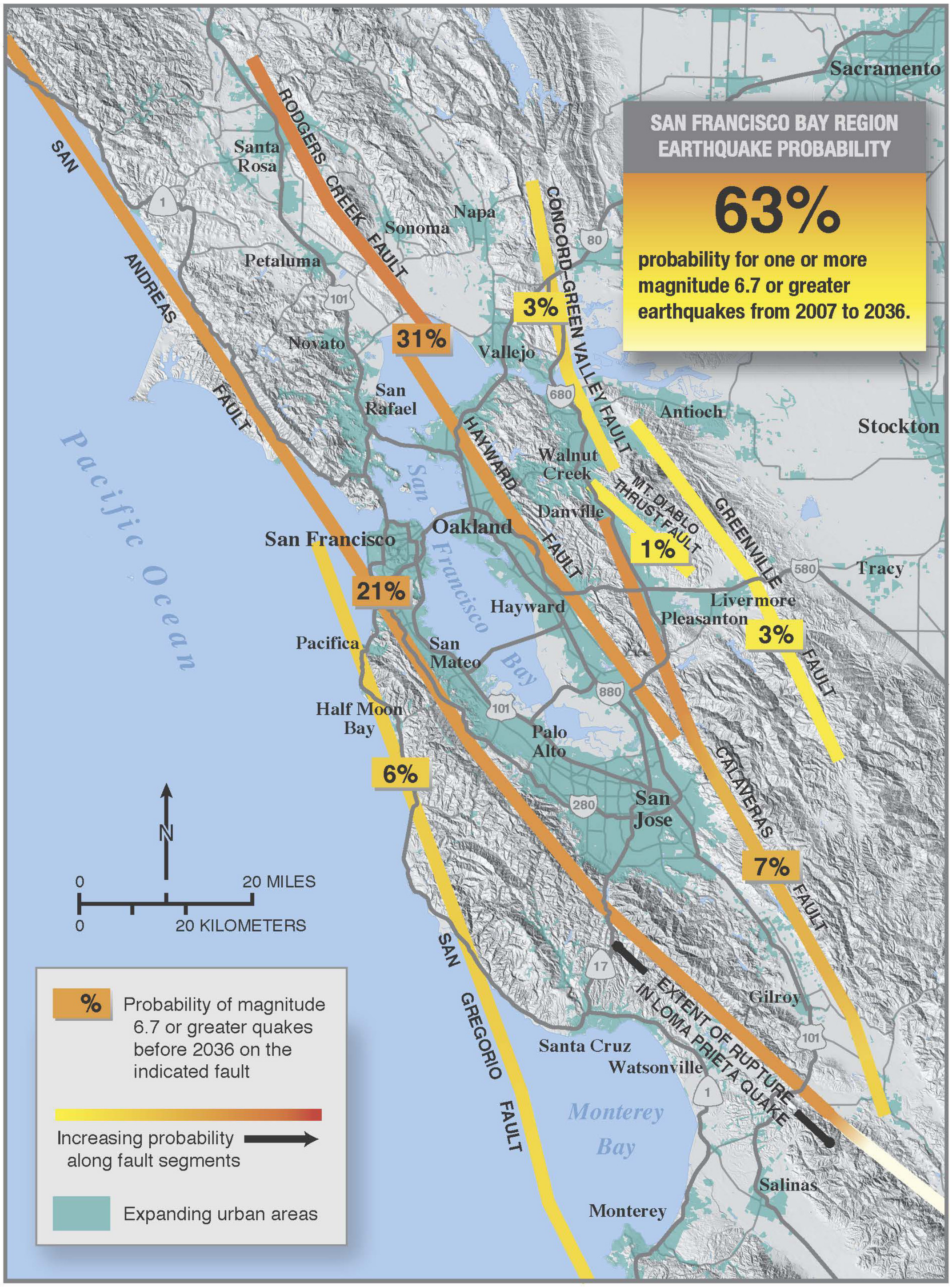


Figure 2-1 2008 earthquake probabilities from the USGS website. Available at http://earthquake.usgs.gov/regional/nca/ucerf/, last accessed March 5, 2015.

The most recent earthquake prior to 2014 was the magnitude-5.0 Yountville earthquake, which took place roughly 15km northwest of Napa on an unnamed fault west of the West Napa fault in September 2000.

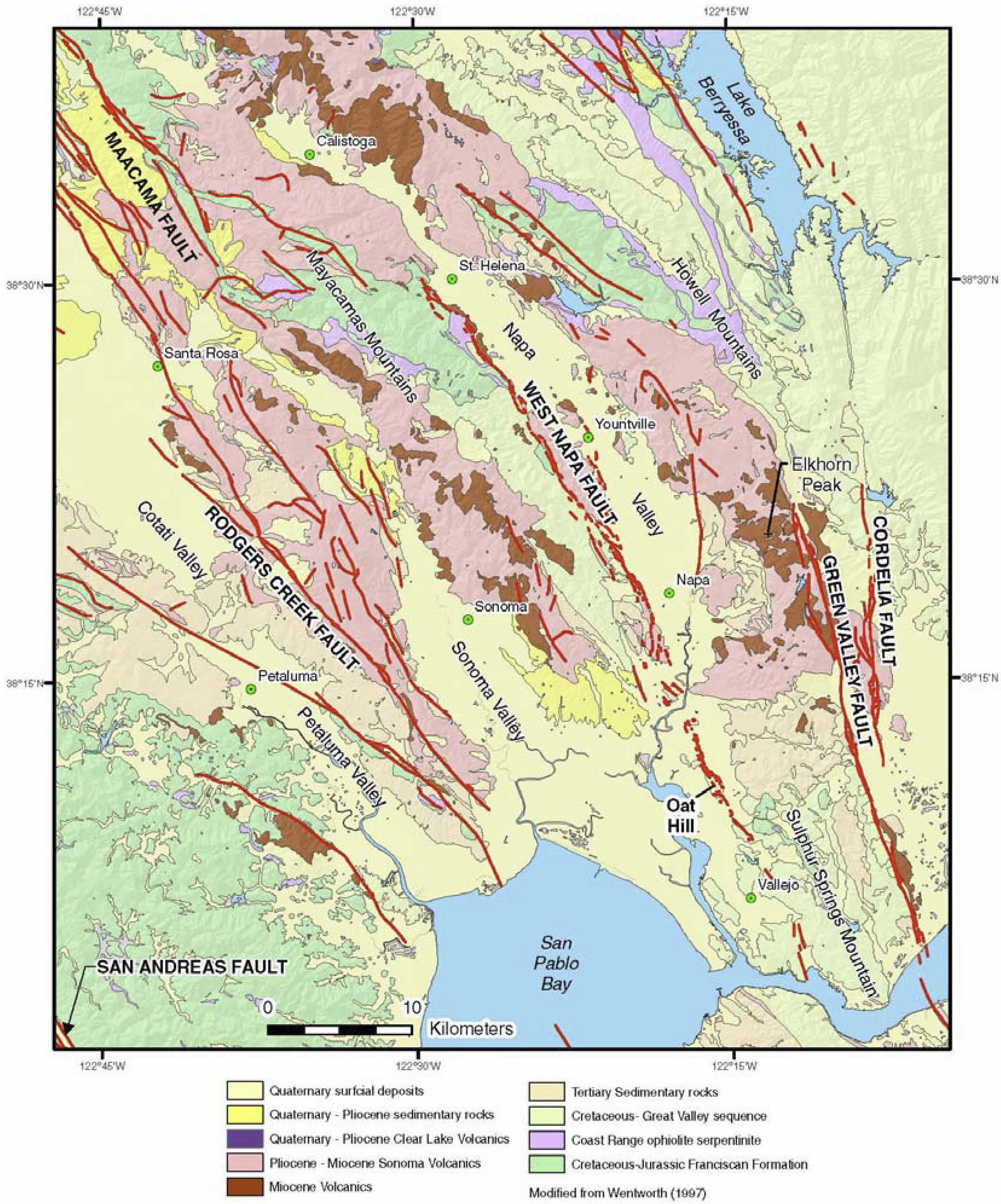


Figure 2-2 Geologic map showing fault lines (USGS, 2008).

2.2 The South Napa Earthquake

With a magnitude of 6.0, the South Napa earthquake is the largest earthquake to strike the San Francisco Bay Area since 1989, when the region was shaken by the magnitude-6.9 Loma Prieta earthquake. It was also the first earthquake to produce significant surface rupture in Northern California since the 1906 San Andreas event.

The epicenter was located approximately 8 km south-southwest of Napa. The earthquake was recorded by a network of seismographs in the Bay Area, which located the hypocenter at the south end of Napa Valley at a depth of 10 km. The primary direction of the earthquake ground motion was to the north, meaning that downtown Napa experienced the strongest ground motion.

The West Napa fault is actually a system of several active fault strands, all running north-northwest along the west side of Napa Valley. The earthquake resulted in significant fault rupturing that extended from Cuttings Wharf south of Napa roughly 14 km to the north through Browns Valley and ending at Alston Park in the northwest corner of Napa. Faulting also extended 1 to 2 km southeast in American Canyon (Figure 2-3).

A ShakeMap was developed immediately after the earthquake utilizing ground motion station information (Figure 2-4).

The ShakeMap indicates that the entire Napa Valley sustained shaking of Modified Mercalli Intensity (MMI) VI to VII (strong to very strong), with the southern portion of the Valley and the City of Napa shaking at MMI VII to VIII (very strong to severe) intensity. The maximum intensity reading was MMI IX (violent) at Napa Fire Station No. 3.

Three strong motion recording sites are located within the City of Napa with their attributes listed in Table 2-1 and the ground motion recordings in Figures 2-5 through 2-7.

The GEER report compared these recorded ground motions to the code-based design spectra. The results are shown in Figures 2-8a through 2-8c. This comparison for the sites show that the pseudo-spectral accelerations recorded at Napa College and Napa Fire Station No. 3 exceeded the Maximum Considered Earthquake (MCE) design spectra at a period around 1.5 seconds near the fault. This effect, referred to as a “bulge” by some, is related to the near-fault velocity pulses discussed in the GEER report and how the velocity pulse interacted with the Napa Valley soils. The GEER report recommends further investigation to study the damage observations related to the recorded ground motions compared to the design spectra.

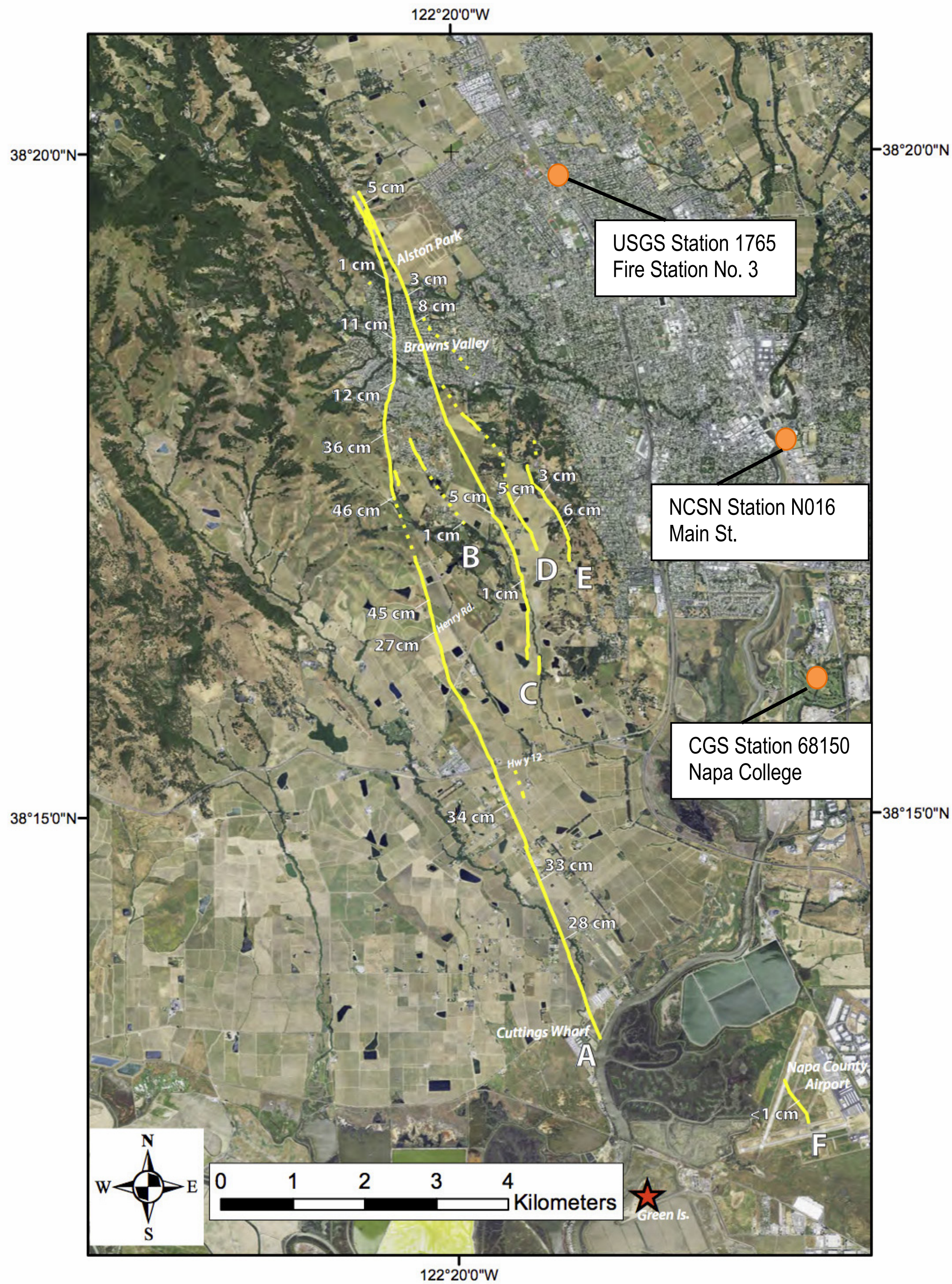
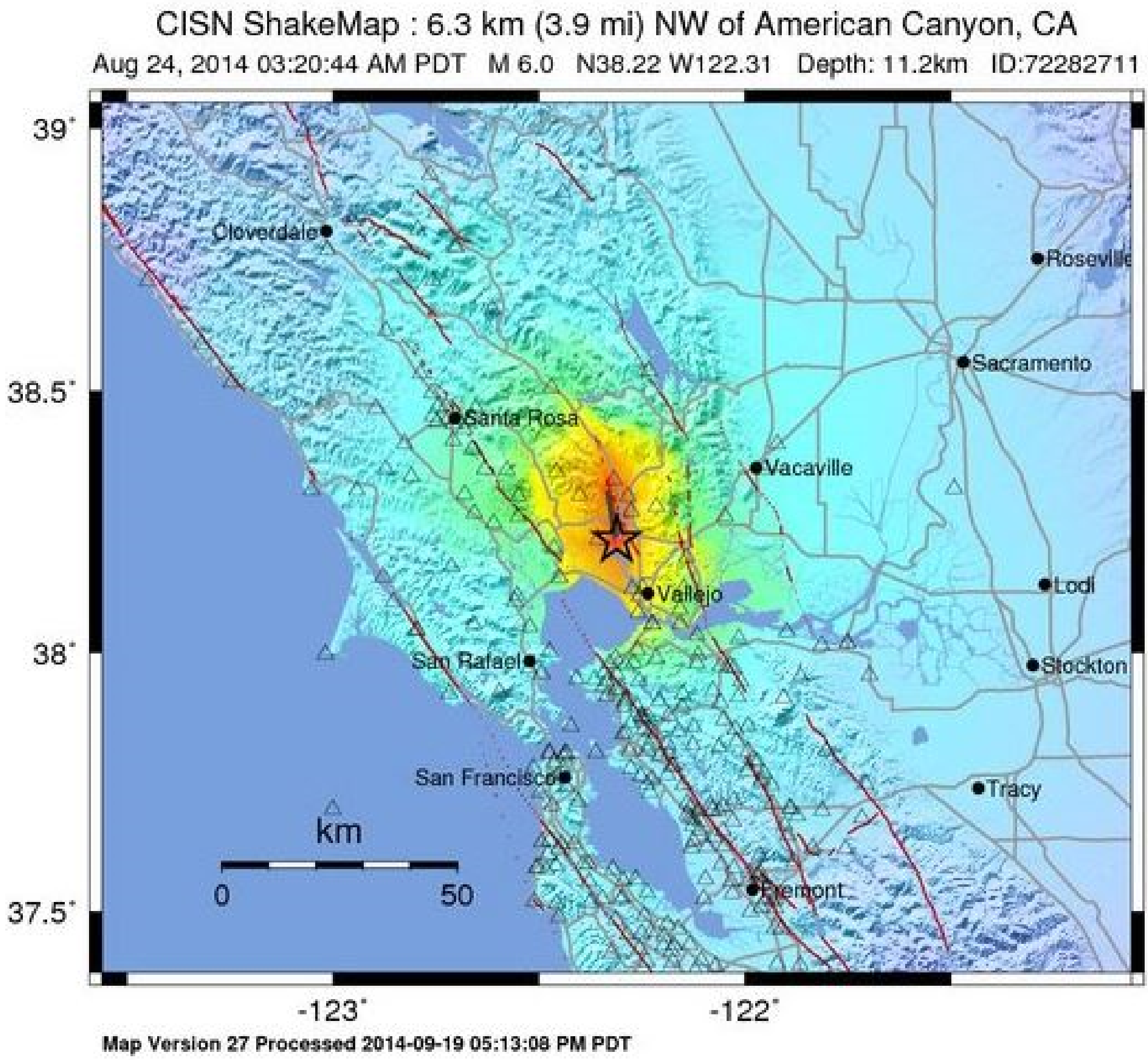


Figure 2-3 Surface faulting (yellow lines) produced by the August 24, 2014 South Napa earthquake (USGS, 2014). Numbers show maximum measured right-lateral offset at selected sites, rounded to the nearest cm; includes both coseismic, as well as measured afterslip as of November 17, 2014. Red star shows the location of earthquake epicenter, orange circles show the location of three ground-motion recording instrument stations.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.1	0.5	2.4	6.7	13	24	44	83	>156
PEAK VEL.(cm/s)	<0.07	0.4	1.9	5.8	11	22	43	83	>160
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Wald, et al.; 1999

Figure 2-4 USGS ShakeMap. Available at http://earthquake.usgs.gov/earthquakes/eventpage/nc72282711#impact_shakemap, last accessed March 15, 2015.

Table 2-1 Strong Motion Recording Sites within the City of Napa

Station Name	Station Owner	Station ID	Distance to Fault
Napa College	California Geological Survey (CGS)	68150	4.1 km
Main Street	U.S. Geological Survey Northern California Seismic Network (NCSN)	N016	3.9 km
Fire Station No. 3	U.S. Geological Survey National Strong Motion Project (NSMP)	1765	2.6 km

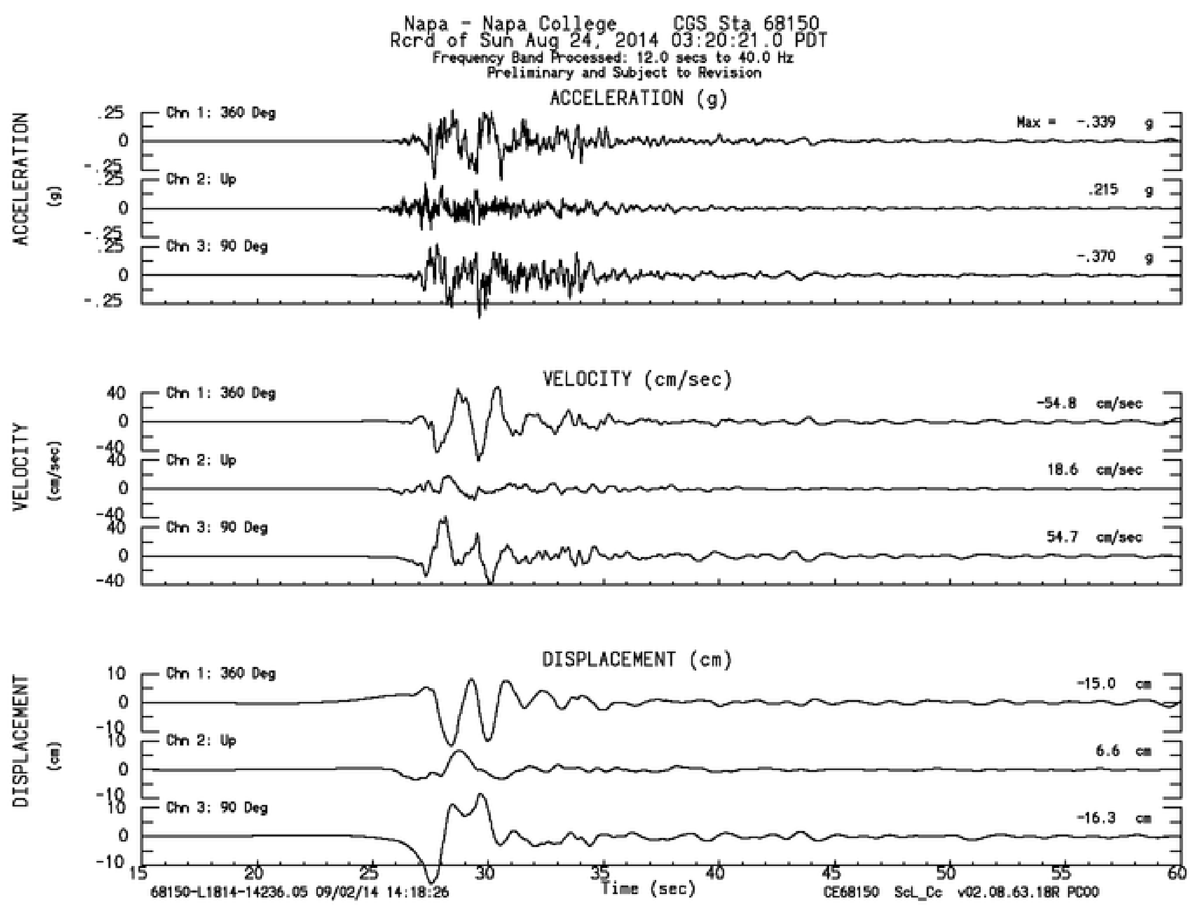


Figure 2-5 Ground Motion recordings from Napa College, CGS Station 68150 (from http://strongmotioncenter.org/, last accessed January 12, 2015).

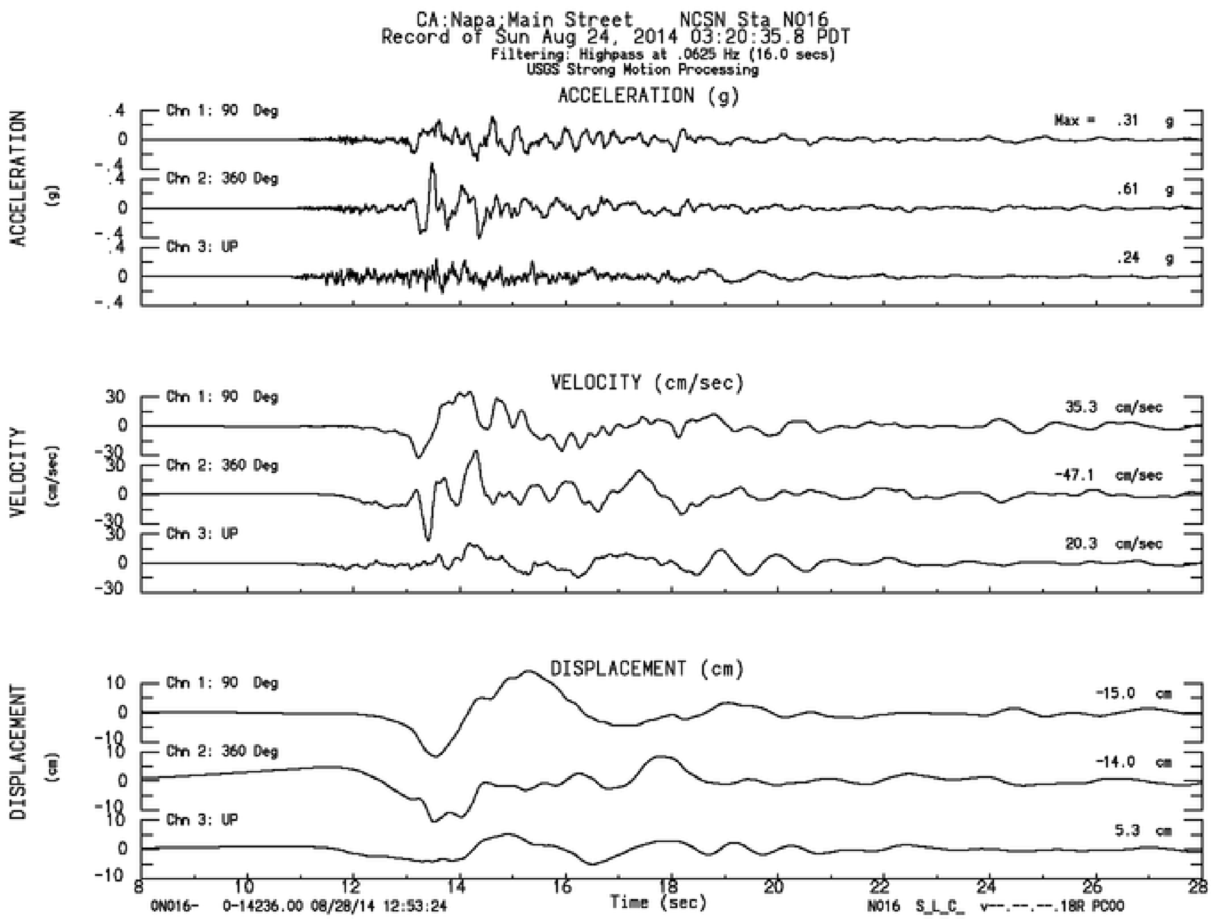


Figure 2-6 Ground Motion recordings from Main Street, USGS NCSN Station N016 (from http://strongmotioncenter.org/, last accessed January 12, 2015).

The Main Street recording depicted by RotD100 is comparable to or exceeds the design basis earthquake (DBE) for building systems using ASCE/SEI 7-10, *Minimum Design of Buildings and Other Structures*, (ASCE, 2010) with periods of vibration less than 0.7 seconds at this site, assuming Site Class D for soils. Note that this ground motion had a strong motion duration of only six seconds.

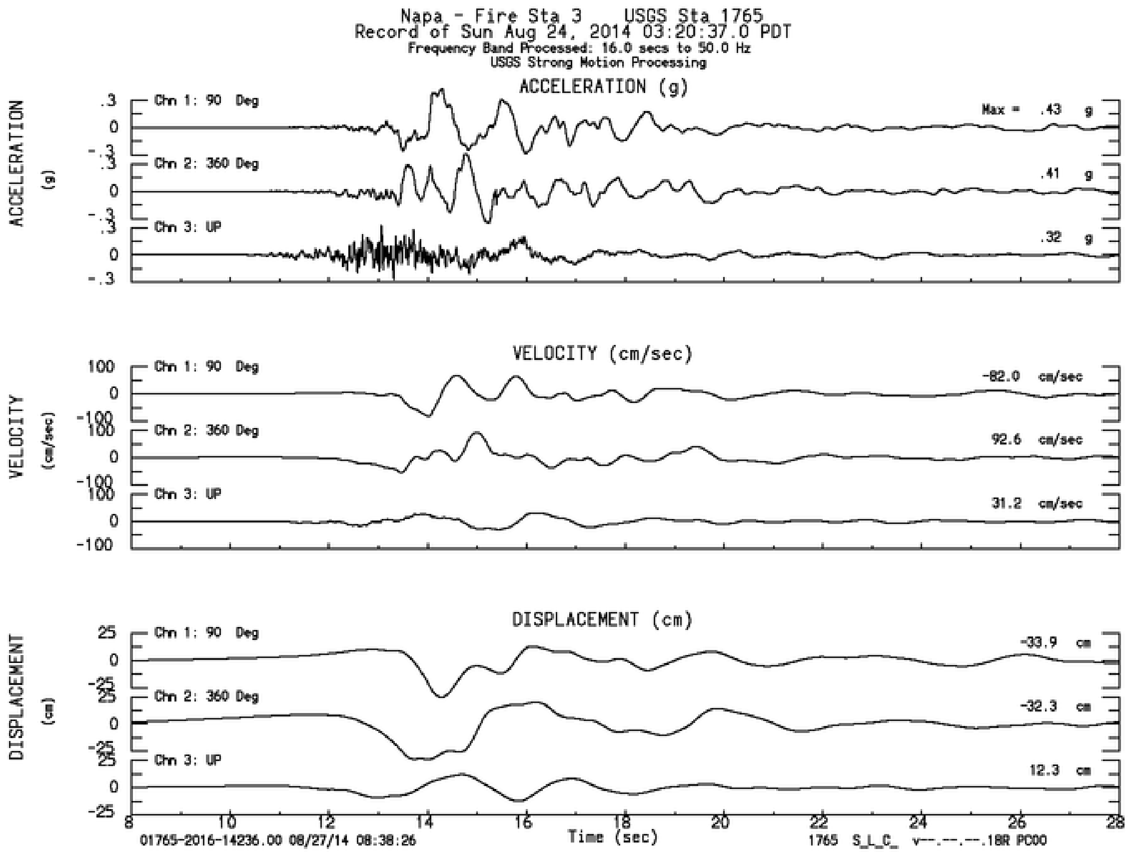
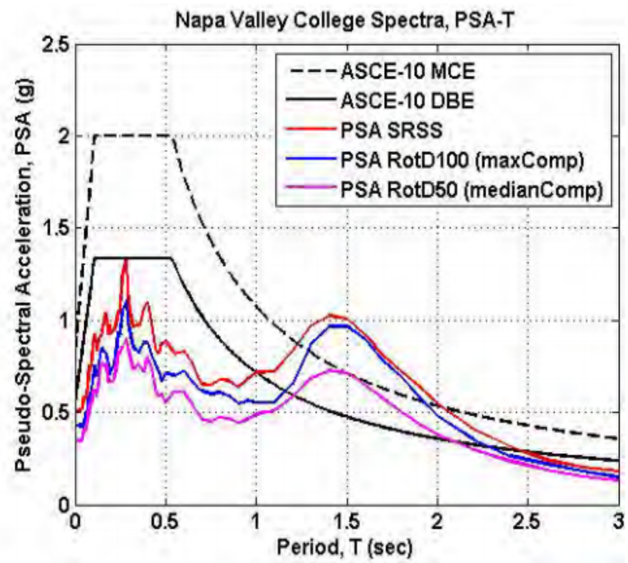


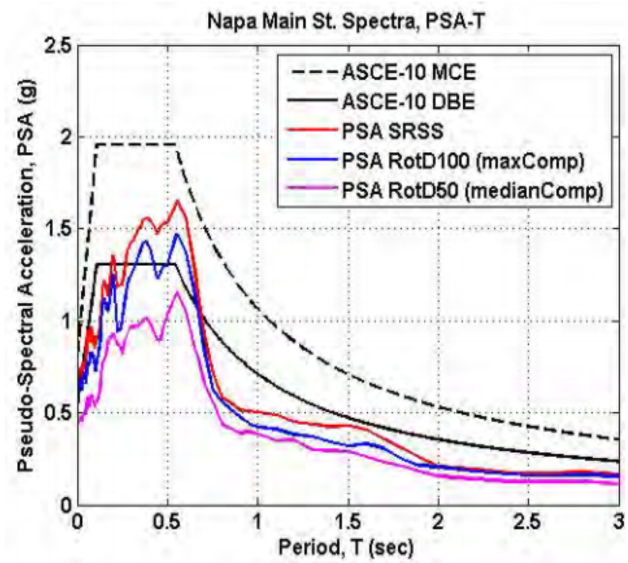
Figure 2-7 Ground Motion recordings from Fire Station No. 3 USGS NSMP 1765 (from http://strongmotioncenter.org/, last accessed January 12, 2015).

2.3 Impact of the South Napa Earthquake

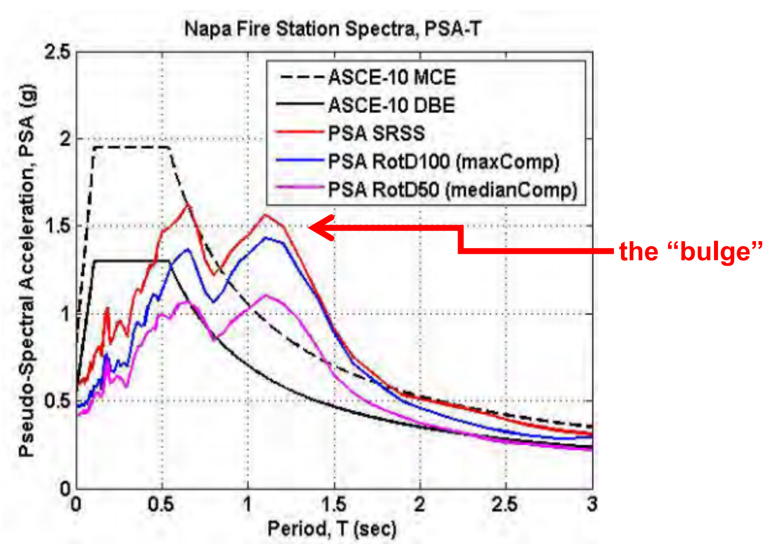
The City of Napa and the surrounding area constitutes a small but significant urban area containing about 77,000 people. This magnitude-6.0 earthquake is considered a moderately strong earthquake. In such an earthquake, while significant losses due to property damage are expected, the risk of serious casualties is believed to be low. The USGS Prompt Assessment of Global Earthquakes for Response (PAGER) system estimated that economic losses directly attributable to the earthquake damage would likely (63% chance) be in the range of $100 million to $10 billion. The PAGER system also estimated that there would be a very low likelihood of casualties. While the actual estimates for the economic damage are still being determined, the actual losses suffered appear to be generally in line with the performance expectations.



(a)



(b)



(c)

Figure 2-8 Comparison of code-based versus resultant pseudo-spectral acceleration (PSA) spectra on Site Class D for: (a) Napa Valley College; (b) Main Street; and (c) Fire Station No. 3. Red arrow indicates the bulge. From GEER (2015).

For non-essential structures, which make up the vast majority of the building stock, the performance expectations of the building codes for relatively modern structures subject to a moderately strong earthquake are repairable or no structural damage, and potentially significant but repairable nonstructural damage. Buildings designed prior to the introduction of modern seismic design requirements can sustain intense structural damage and serious nonstructural damage. However, except for known hazardous building types, such as unreinforced masonry structures that have not been retrofitted, the probability of life-threatening damage or collapse is believed to be low.

This event also served as a test of the earthquake early warning system presently being developed by the USGS and the State of California. The system generated a warning within five seconds, and provided a ten-second warning to the test site at Berkeley prior to the arrival of earthquake ground motions. No warning would have been possible within 20 miles of the epicenter due to the delay inherent within the system.

2.4 West Napa Fault Rupture and Afterslip

The earthquake produced more than 14 km of surface rupture from the Napa River at Cuttings Wharf in the south, through Browns Valley to Alston Park, within the City of Napa, as shown in Figure 2-3. This amount of fault slip is considered unusual for a magnitude-6.0 earthquake.

After the earthquake, parts of the fault were observed to continue to slip. This is a phenomenon called afterslip that has been previously described for many earthquakes, including several cases in California. Some examples include the 1979 Imperial Valley, 1987 Superstition Hills, and 2004 Parkfield earthquakes. Afterslip occurs quickly at first, then slows down and is thought to eventually stop long after the earthquake.

Figure 2-9 shows the same location 11 hours after the earthquake on August 24, 2014 and on January 12, 2015, over four months after the earthquake. According to the USGS, 11 hours after the earthquake (Figure 2-9a), the slip was observed to be several inches, and 135 days later on January 12, 2015 (Figure 2-9b), the slip was measured as 15 inches. Even though the creep rate several months after the earthquake is barely perceptible, the West Napa Fault Zone main strand at this location (and in the south, near Highway 12 crossing) is still creeping faster than the San Andreas fault's creeping section. However, the creep rate is slowing down and is expected to drop down to lower than the San Andreas creep rate in the coming months.

(a) (b)

Figure 2-9 Observation of afterslip at Leaning Oak Drive following the South Napa earthquake: (a) 11 hours after the earthquake (photo from Alex Morelan); (b) 4 months after the earthquake (photo from Kenneth Hudnut, USGS).

In this event, a portion of the fault system experiencing afterslip runs through a residential neighborhood, and is impacting a number of residential structures. The afterslip is primarily lateral.

Figure 2-10 shows all of the fault traces from the West Napa fault and their respective afterslip hazard.

- The yellow fault trace means a moderate level of afterslip hazard; likely to experience less than 15 cm, but more than 5 cm, of afterslip during the three years after the earthquake.
- The green fault trace means a low level of afterslip hazard; very unlikely to experience more than 5 cm of afterslip during the three years after the earthquake.

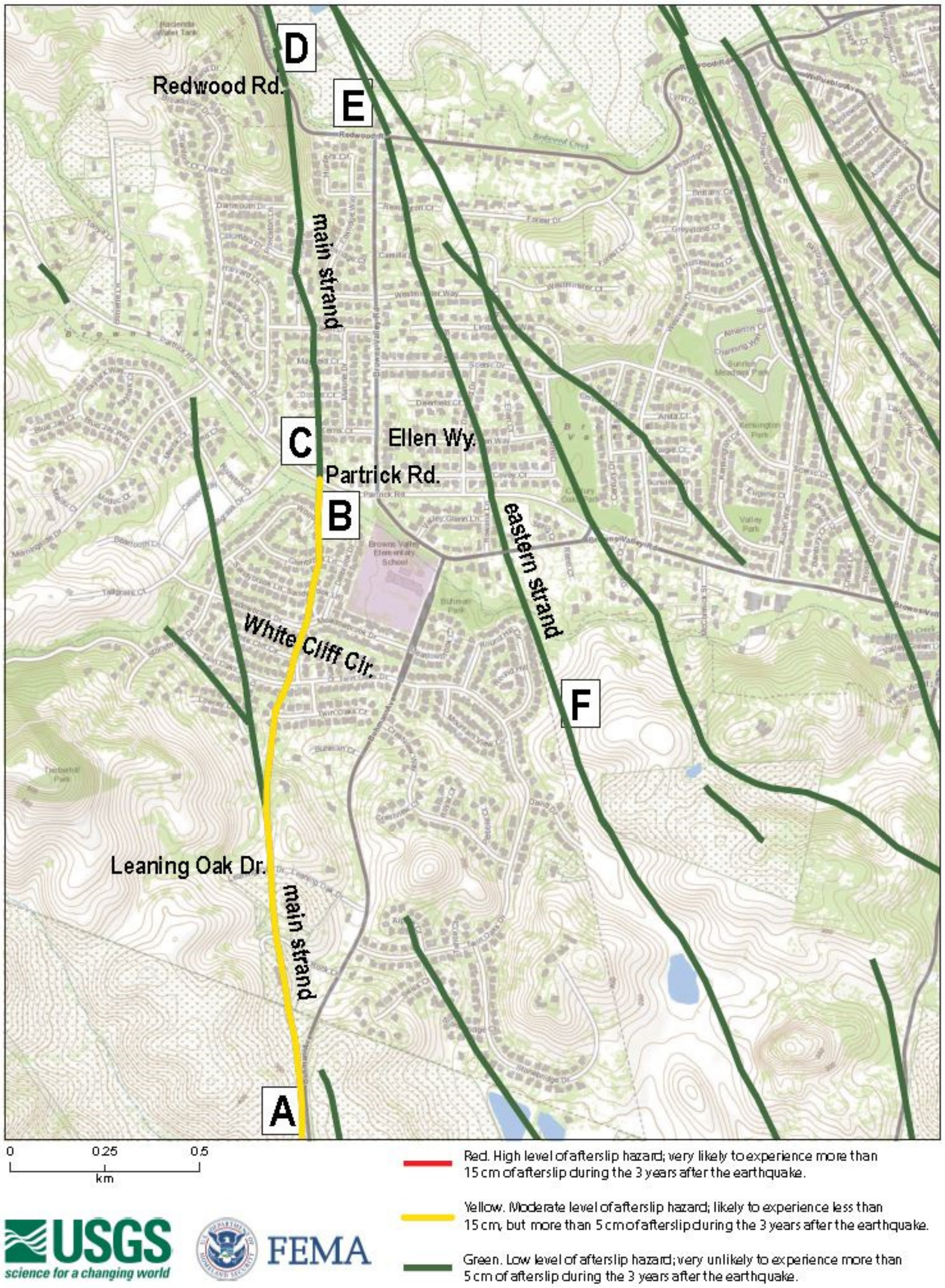


Figure 2-10 Afterslip map (USGS, 2014). Note that there are no red lines in the figure.

According to the USGS, for all levels of afterslip hazard, the afterslip amount that is measured 90 days after the earthquake may double within 10 years after the earthquake, although lower levels of afterslip are also possible. The afterslip and associated hazard decreases exponentially with time, posing a moderate hazard. The southern part of the main strand of the West Napa fault within Browns Valley, from south of Leaning Oak Drive up to Partrick Road (A to B on Figure 2-10), is shown in yellow on the map and poses an ongoing hazard to structures. However, the amount of afterslip is not expected to be so great as to pose a severe hazard.

The northern part of the main strand of the West Napa fault within Browns Valley, north of Partrick Road (C to D on Figure 2-10), has experienced no significant afterslip; low afterslip hazard exists on this part of the main strand of the West Napa fault system. The newly named "eastern strand" of the West Napa fault system (E to F on Figure 2-10) has experienced no afterslip; low afterslip hazard exists on this part of the eastern strand. Other fault strands shown in green on the figure also have low afterslip hazard.

2.5 Summary

The South Napa earthquake was a moderately strong event. The damage observed following the earthquake is consistent with the performance expectations for modern and older structures. The near-fault velocity pulses observed in the ground motion records might have had a greater impact on the built environment, had the building stock in Napa included taller, more flexible structures. The interaction between the velocity pulse and the Napa Valley soils is currently not completely understood.

The impact of afterslip on the Browns Valley residential neighborhood has caused considerable damage and is an ongoing issue. The afterslip phenomenon and the issues it poses for structures straddling a fault trace were not familiar topics to engineers or building owners, especially homeowners. In addition, mitigation strategies for this hazard are not well known or highly developed. Even if the mitigation approaches were more widely known, the homeowners would probably not be able to afford the measures since most do not have earthquake insurance, and without insurance or some other form of support, most would not be able to afford the effective mitigation measure.

2.6 Recommendations

The South Napa earthquake highlighted several areas for further study, including:

1. Further investigation of the damage observations in relation to recorded ground motions.
2. Comparison of response spectra from instrumental records to the design spectra for periods of vibration relevant to each building.
3. Comparison of estimates of nonlinear response of buildings to the ground motions with actual response, especially considering the influence of the velocity pulses.

4. Outreach to the community to increase awareness of the issue of afterslip.

5. Development of cost-effective methods to estimate afterslip potential and mitigate the effects of afterslip. For areas where afterslip potential is high and could impact a conventional foundation, the best way to mitigate this hazard would be to replace the existing foundation with a reinforced concrete mat or raft slab foundation, which has enough reinforcing steel to support itself even if the ground is still slipping beneath. Also, measures to protect gas lines and other vulnerable piping from damage caused by ground movement should be developed. However, given that most foundation repairs have already been made and given the cost of replacing a building foundation with a new system, this mitigation measure is likely not cost effective at this time for existing buildings. It should, however be considered for new construction within the moderate hazard areas.

Chapter 3
Performance of Buildings Near Station N016

A major focus of this project was the collection of data for all buildings within a 1,000 foot radius of Station N016 located on Main Street in Napa to be able to tie the damage observations back to an input ground motion. All surveyed buildings within a 1,000 foot radius around Station N016 are included in the data set. The database containing the information collected will be published by the Applied Technology Council (ATC). Buildings included in this report but outside of the 1,000 foot radius are not included in the data presented in this chapter.

This chapter summarizes the trends in the surveyed parameters judged to be most significant, and includes a general discussion of the damage patterns and trends that were observed. Buildings of special interest are discussed in greater detail in Chapter 4. Surveys were conducted over three days, ranging from twelve to sixteen days after the earthquake. Information recorded by the surveyors is based on their observation at that time.

3.1 Survey Area

A total of 68 buildings were surveyed within 1,000 foot of Station N016 located in downtown Napa. Figure 3-1 shows the extent of the area surveyed. Interior and exterior surveys were conducted for 50 of the buildings, with the remaining 18 receiving only exterior surveys.

3.2 Survey Method

Data in this chapter are based on the information and observations collected and recorded on forms originally developed for ATC-38 report, *Database on the Performance of Structures near Strong-Motion Recordings: 1994 Northridge, California, Earthquake*, (ATC, 2000) and modified for this project. Appendix A provides a summary of modifications implemented to account for new knowledge and specific issues related to this event and provides the forms and instructions used by investigators on the field.

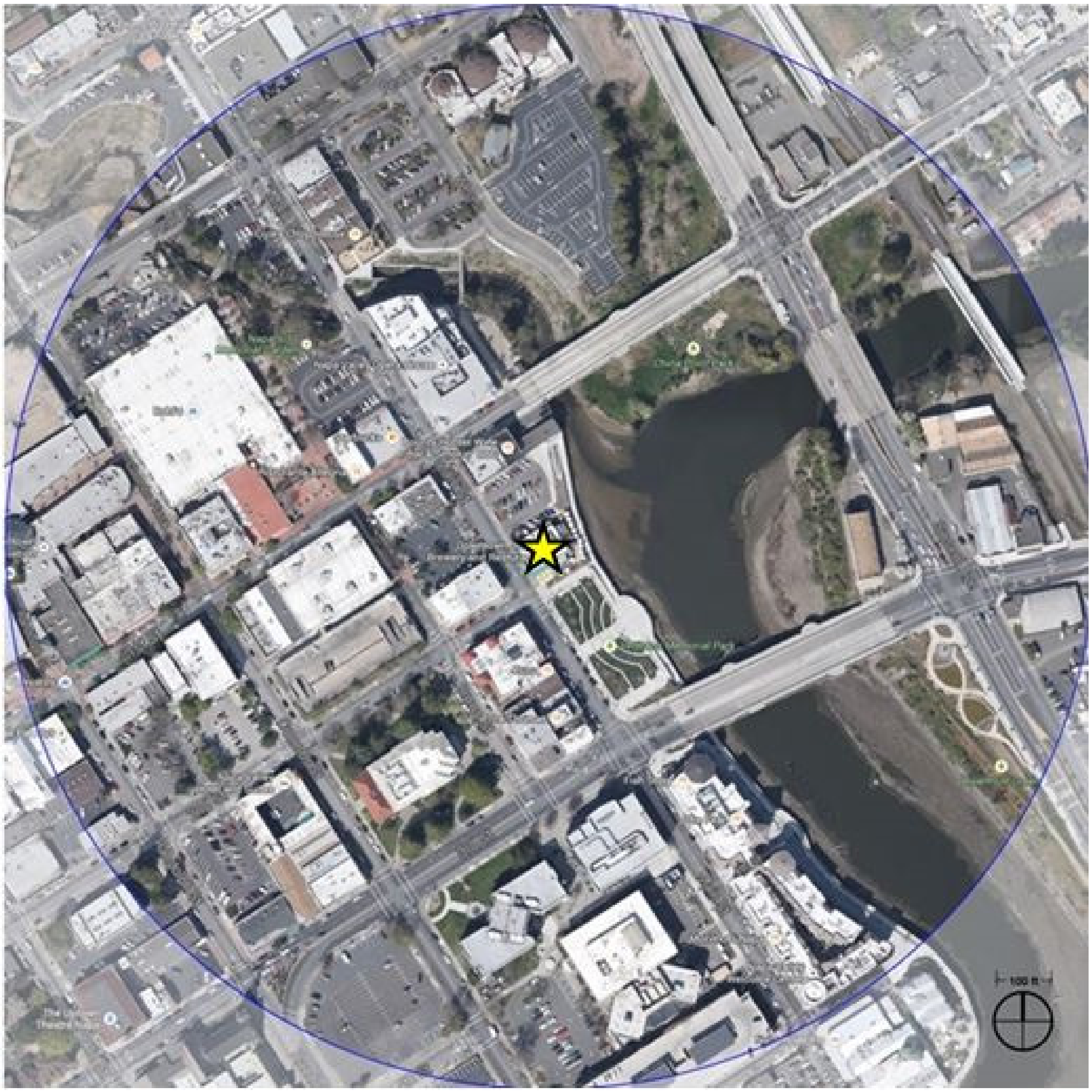


Figure 3-1 Aerial photo showing the surveyed area. The yellow star indicates the location of Station N016. The blue circle indicates the extent of the 1,000 foot radius. (Image source: Google Maps).

The form is six pages long and comprises the following blocks of information:

- Building site information
- Building construction data
- Model building type
- Performance modifiers
- Sketch of building
- Nonstructural elements

- General damage
- Nonstructural damage
- Injuries or fatalities
- Functionality
- Geotechnical failures
- Additional comments
- Detailed damage description

The remainder of this chapter summarizes the data collected where information is relevant.

3.3 Building Site Information

The information collected on this portion of the form pertains to site investigation date, building address, and contact information. In addition, existing posting placard information and general damage classification information is also collected.

The placard information is based on the methodology presented in *ATC-20-1 Field Manual: Postearthquake Safety Evaluation of Buildings* (ATC, 2005). Postearthquake safety evaluations in the City of Napa were managed by the Building Division, under the direction of the Chief Building Official. The City of Napa presented status reports on the process of evaluations on the City website on August 25, 2014 and September 5, 2014 (City of Napa, 2014a and 2014b). It was first reported that 70 buildings within the city were posted as UNSAFE and the number of buildings with RESTRICTED USE placards was approaching 200. On September 5, 2014, it was reported that 125 buildings within the city were posted UNSAFE and over 1,000 buildings were posted RESTRICTED USE. At the time of the field investigation on September 9, 14 buildings within the 1,000 foot survey area were posted UNSAFE and 15 were posted RESTRICTED USE (out of a total 68 buildings).

3.4 Building Construction Data

Dates of construction in the surveyed area vary widely, from 1856 to 2014. A significant number of historical buildings were located within the survey area, including 10 on the National Register of Historical Places. Approximate dates of construction were provided on 81% of the building surveys. Based on these data, the median age is 1930. Nineteenth century buildings make up 16% of the set, and 62% were constructed before 1950. Figure 3-2 shows the distribution of buildings by original construction date.

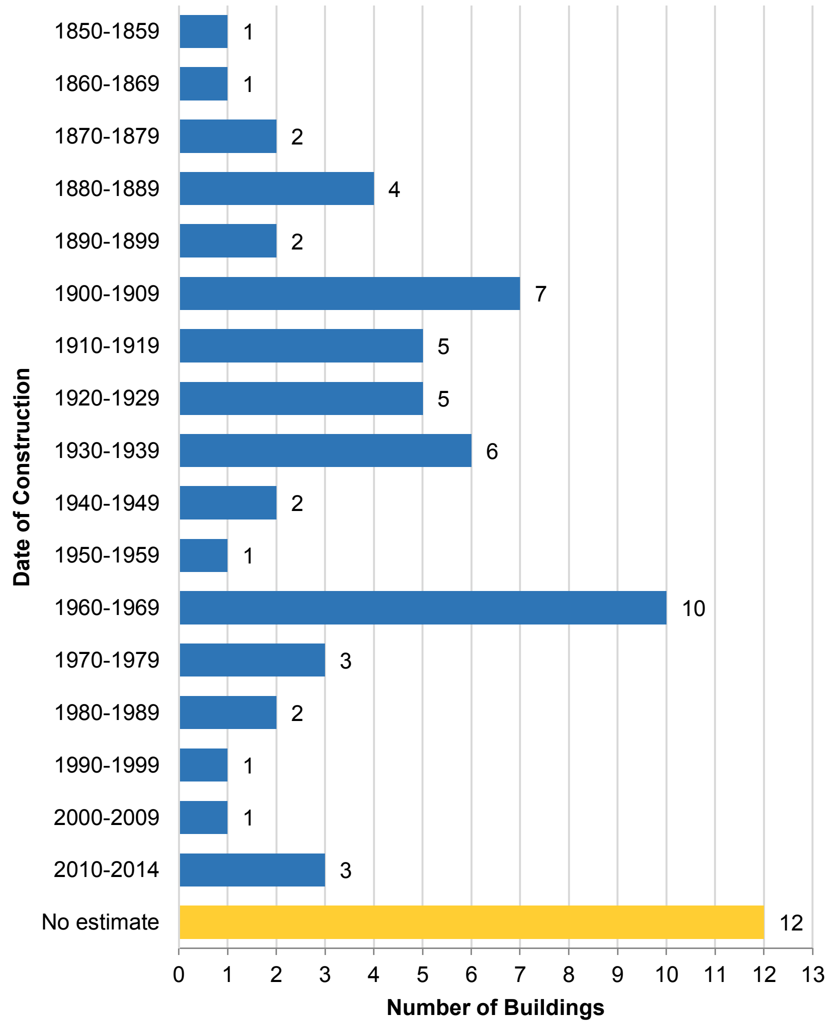


Figure 3-2 Distribution (by count) of buildings by date of original construction.

Buildings ranged in height from one to three stories, with the majority of buildings (57%) having one story.

The survey area is located in a commercial district of downtown Napa. Accordingly, the majority of buildings (59%) include retail or restaurant space. The next two most common occupancy types are offices (16%) and government (9%). The surveyed buildings also include three parking garages, two warehouses, two theaters, and one hotel. Figure 3-3 shows the distribution of buildings by occupancy type.

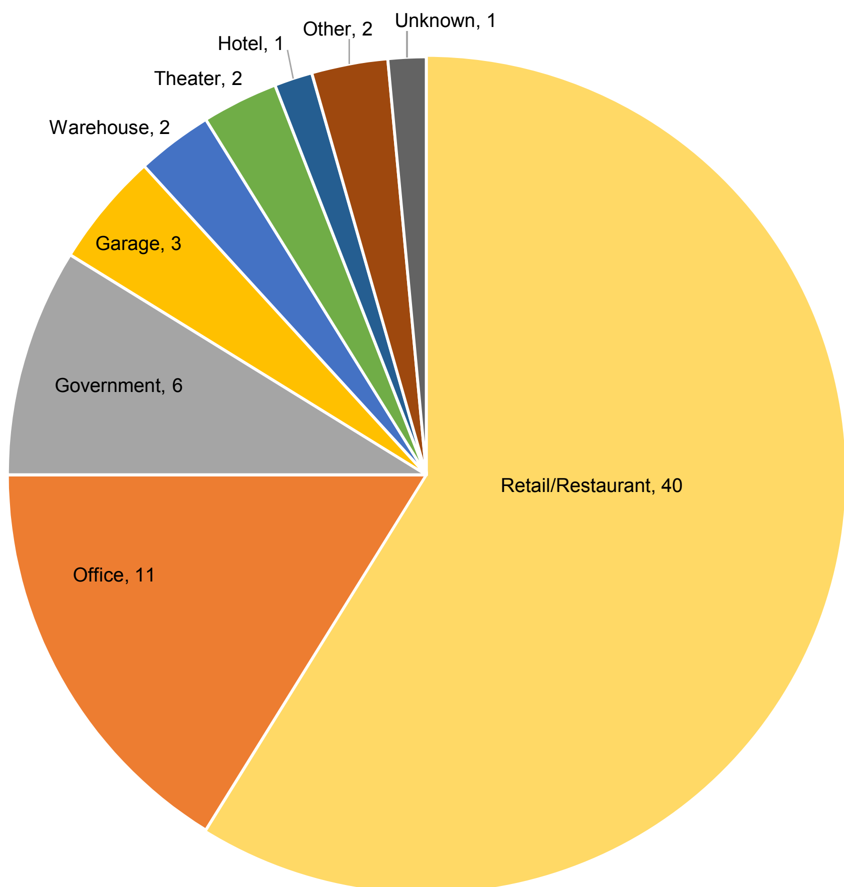


Figure 3-3 Distribution (by count) of buildings by occupancy type (of a total of 68 buildings).

The Napa River runs through the survey area. Many of the buildings in the surveyed area are located immediately adjacent to the river. The FEMA Flood Insurance Rate Map (FIRM), Panel 516 of 650, for Napa shows that while most of the study area lies within Zone X (higher elevation than the 0.2-percent-annual-chance flood), only one of the buildings was within Zone AE, but it was adequately elevated above the Base Flood Elevation of 18 feet. With the exception of three buildings, all of the buildings were observed to be on level sites.

3.5 Model Building Type

In the survey area, masonry bearing wall systems make up more than half of the inventory, though a wide range of building types are represented in smaller numbers. Information about building type was provided on 93% of the building surveys and was not reported on the remaining 7%. Unreinforced masonry (URM) construction is the most common original structural system (41% of the buildings) in the surveyed area. The second most prevalent type is reinforced masonry with 22% of the buildings. Nontilt-up concrete shear wall buildings are the third most common with 7%. Other building types present include: wood-frame (light-frame and long-span), precast/tilt-up concrete shear wall, concrete moment frame, steel frame (moment and braced), and light-gage steel. Figure 3-4 shows the breakdown of the surveyed buildings by their original structural system.

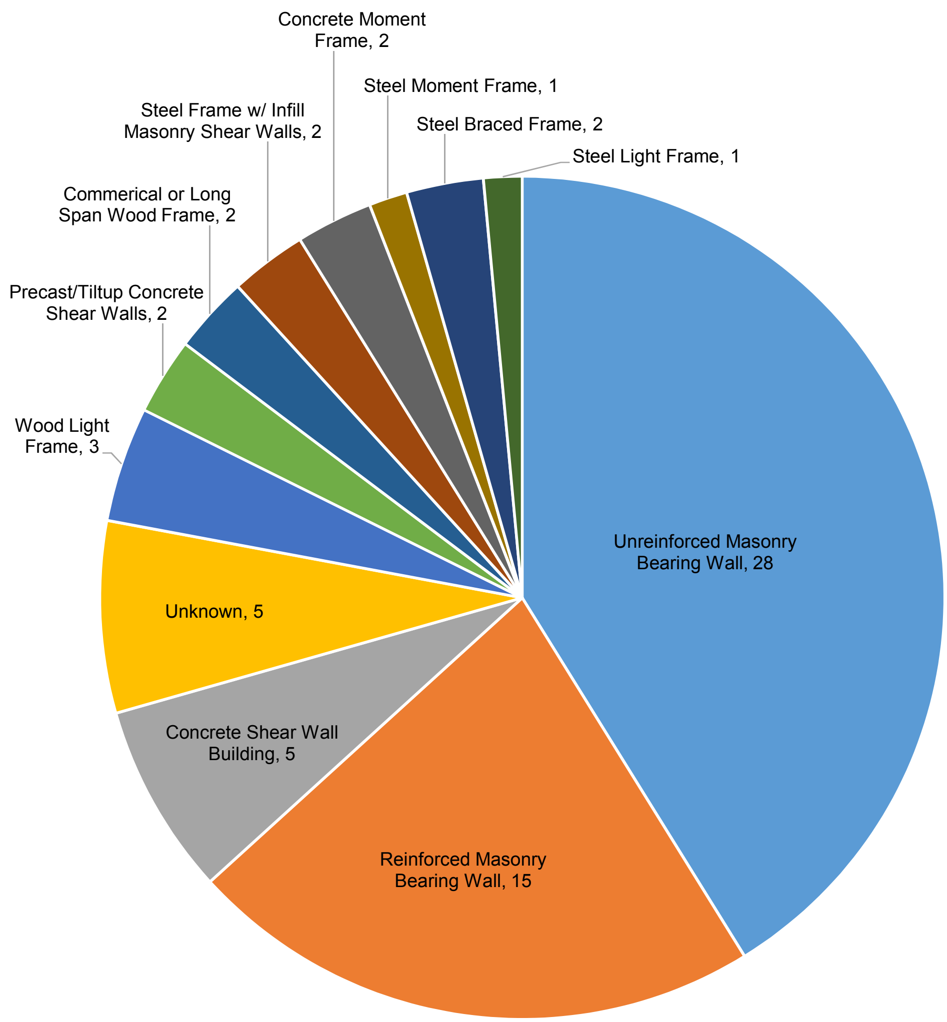


Figure 3-4 Distribution (by count) of buildings by original structural system (of a total of 68 buildings).

A major focus of this investigation was the performance of seismically retrofitted unreinforced masonry buildings; of the 28 URM buildings, 71% (20 buildings) have been retrofitted. Figure 3-5 shows the retrofit status of the unreinforced masonry buildings in the survey.

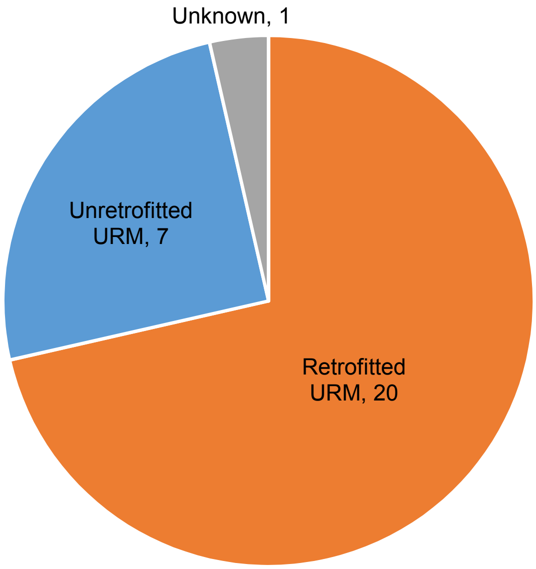


Figure 3-5 Distribution (by count) of unreinforced masonry building retrofits (of a total 28 URM buildings).

3.6 Performance Modifiers

More than half of the buildings (57%) were identified as having inadequate separation between adjacent buildings to preclude pounding. Storefront windows are common in the area, and approximately half of the buildings have open front plans (46%). Plan irregularities were identified in 21% of the buildings and structural deterioration was observed in 7%.

3.7 Nonstructural Elements

Parapets were observed on half (50%) of the buildings. Chimneys were uncommon in the area, observed in only 6% of the buildings.

3.8 General Damage

Information about damage to each building was collected for the structural system. Two independent damage rating systems were used in the survey:

- **General Damage Classification:** Damage is classified based on the extent of repairs required. The scale ranges from no visible damage ("none"), to "insignificant," "minor," and "moderate" to extensive damage for which repair may not be economically feasible and could require building demolition ("heavy") and "collapse." Surveyors were permitted to classify the damage as "unknown" if thorough observation to classify the damage was not safe or feasible. Appendix A provides detailed descriptions for each level.

- **ATC-13 Damage State:** Damage is classified based on percent damage (damaged value divided by replacement value) and is based on ATC-13, *Earthquake Damage Evaluation Data for California* (ATC, 1985). Appendix A provides detailed descriptions for each level.

3.8.1 General Damage Classification

Distribution of building damage by General Damage Classification is shown in Figure 3-6. Overall, nonstructural damage was more common than structural damage: 80% of buildings were observed to have some degree of nonstructural damage, while 54% were observed to have some degree of structural damage. However, the most severe damage classification "Heavy" was only observed for structural systems; classifications of nonstructural damage were limited to "insignificant" (40%), "minor" (19%), and "moderate" (10%). However, it is noted that UNSAFE placards limited access to observation of nonstructural damage, so these buildings with greater levels of structural damage may well have also had greater amounts of nonstructural damage, but because this could not be verified, they have been categorized under "Unknown."

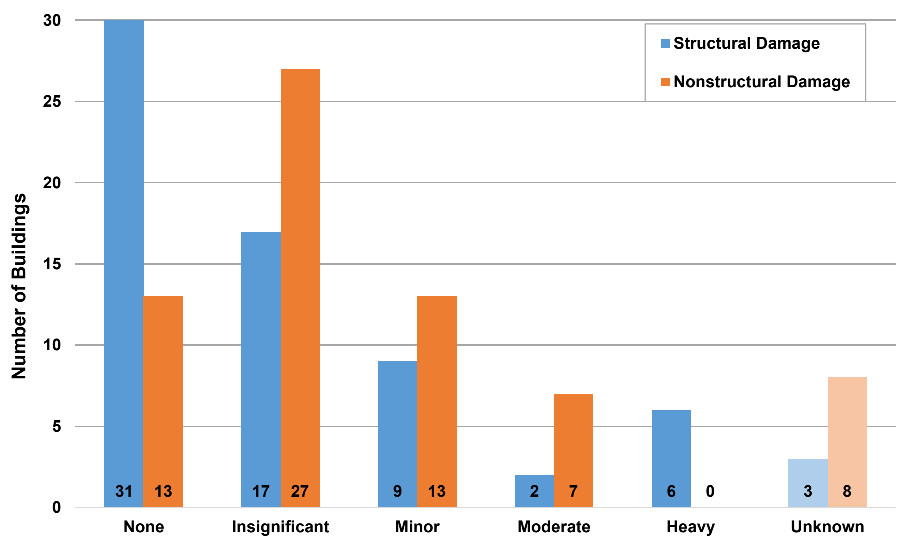


Figure 3-6 Distribution of building damage according to general damage classification.

3.8.2 ATC-13 Damage State

Distribution of building damage by ATC-13 Damage State is shown in Figure 3-7. The data for these assessments show a similar pattern for nonstructural and structural components as that observed in Section 3.8.1. More buildings were assessed to have some degree of nonstructural damage than structural damage, although nonstructural damage was clustered in the less severe states: "slight," "light," and "moderate." Three buildings were assessed as having "heavy" or "major" structural damage, compared to only one building having this severity of nonstructural damage.

This classification system also included assessments for equipment and contents damage. Observation of equipment damage was the lowest of the four categories, with only 19% of buildings. Contents damage was common, with 56% of buildings affected. Two buildings had contents damage classified as "Heavy."

No buildings were classified under any category as "Destroyed" (100% damage).

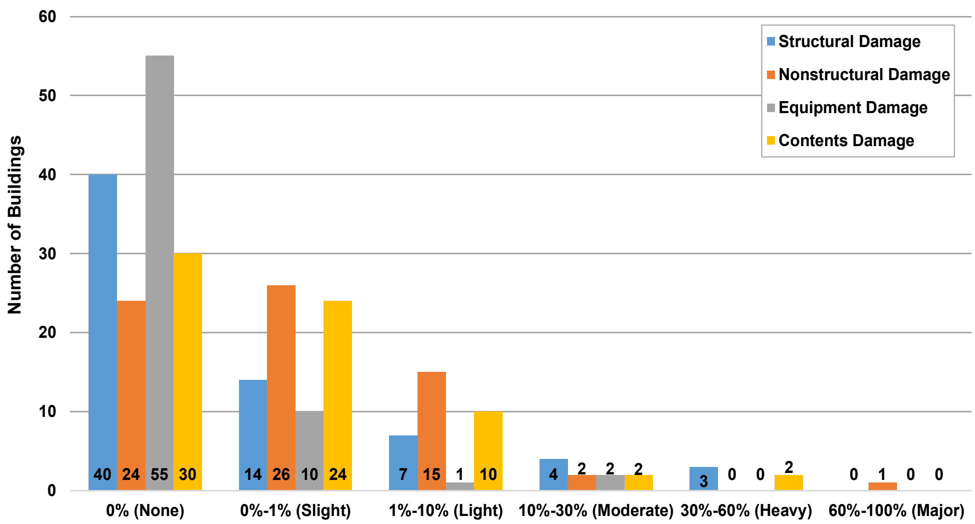


Figure 3-7 Distribution of building damage according to ATC-13 damage states.

3.9 Structural Performance

More than half of the buildings (37) within the 1,000 foot radius of Station N016 sustained some structural damage but the General Damage Classification was "insignificant" or higher for only half of these. The majority of the damage was reported as minor cracking in concrete and masonry buildings.

For the 68 buildings in the study area, no damage to structural components was reported for 31 buildings (46%), and insignificant damage was reported for 17 buildings (25%). A total of 9 buildings (13%) sustained minor structural damage and 2 buildings (3%) sustained moderate damage; with 6 buildings (9%) sustaining heavy damage. The level of structural damage in three buildings could not be determined.

Of the 39 non-URM buildings with reported damage states, 36 sustained either insignificant damage or no damage. One building suffered heavy damage when a URM wall from an adjacent structure collapsed into structure and a portion of one building (Napa County Courthouse, discussed in greater detail in Section 4.3.7) sustained damage to its older portions. A single-story concrete structure built circa-1930 suffered insignificant damage, with cracking observed at the base and top of some concrete columns.

Unreinforced masonry buildings (28) make up a large percentage of the surveyed buildings. Of the seven unretrofitted URM buildings, five were posted UNSAFE:

- One building was heavily damaged: The building is a two-story structure with a setback and sustained significant loss of masonry, with portions of the second story walls falling both into the single-story portion of the building and out into an adjacent alley.
- One building was moderately damaged and sustained serious damage to brick and stone URM walls, including a portion of an exterior wall that fell on a parked automobile.
- One building suffered insignificant damage, but was initially posted UNSAFE. It was subsequently reposted INSPECTED.
- Two remaining buildings were not accessible for inspection; the nature and severity of the structural damage to these buildings could not be determined:

A single-story unretrofitted URM building was posted RESTRICTED USE, but was not accessible for inspection and the nature of the damage is unknown. A single-story unretrofitted URM commercial building suffered insignificant damage; it appears to have been unoccupied at the time of the earthquake.

Among the 20 retrofitted URM buildings, 10 buildings sustained no structural damage or the damage was deemed insignificant; six buildings suffered minor damage, one building moderate damage, and three were heavily damaged. No buildings collapsed. Among the buildings sustaining minor damage, two showed minor cracking of URM walls, and one building that was retrofitted with a combination of steel braced and moment frames sustained localized yielding and buckling in some of the steel elements. One of the two heavily damaged buildings is a two-story commercial structure that suffered extensive cracking to the URM walls, including X-cracking of slender URM piers. Damage to decorative terra-cotta elements was also significant. The other heavily damaged building, also a two-story structure, was reported to have sustained a wall anchorage failure.

A number of different approaches were used to retrofit URMs, and partial retrofits of URM buildings were less successful in limiting damage compared to those that received more comprehensive upgrades. Stone masonry walls and parapets were more likely to sustain damage compared to those of brick masonry. Overall, based on the performance of the URM buildings within 1,000 feet of Station N016, it was observed that the hazard mitigation efforts in Napa were successful in reducing damage and the risk to life safety due to URM construction: Only 20% of the retrofitted URM buildings were posted UNSAFE (one additional building was posted UNSAFE due to damage to an adjacent structure), compared to over 70% of the unretrofitted buildings.

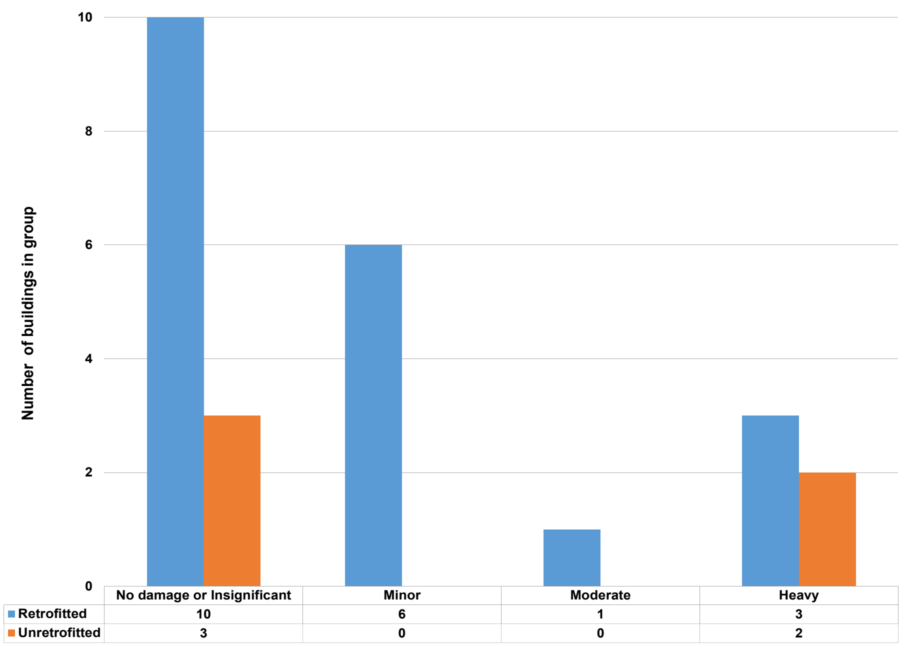


Figure 3-8 Damage comparison between unretrofitted and retrofitted URM buildings.

3.10 Nonstructural Damage

Information about damage to each building was collected for nonstructural components, equipment, and contents. Figure 3-9 shows the distribution of buildings for three major kinds of nonstructural damage: partitions, lights and ceiling, and contents.

Although a majority of the buildings sustained damage in each of these three categories, the observable damage was mostly assessed as "insignificant."

Damage to contents was the most common of the three, with 24% experiencing a significant level of loss ("minor", "moderate," or "heavy"). Damage to lights and ceilings was the next most common, with 16%. Damage to partitions was sustained by the fewest buildings (8%).

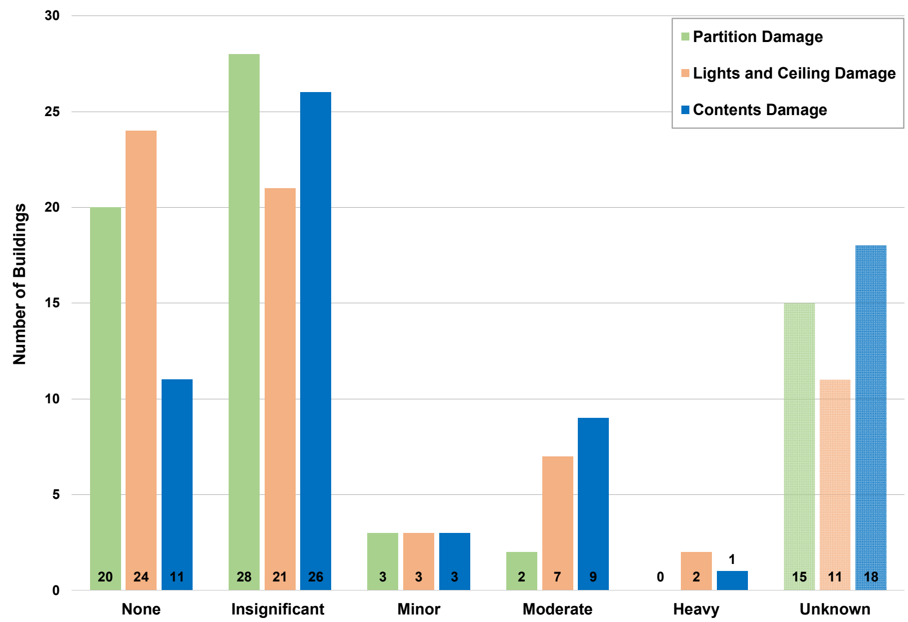


Figure 3-9 Number of buildings where select types of nonstructural damage was observed.

The performance of parapets is summarized in Figure 3-10. Of the subset of buildings with parapets, 21% had observed damage.

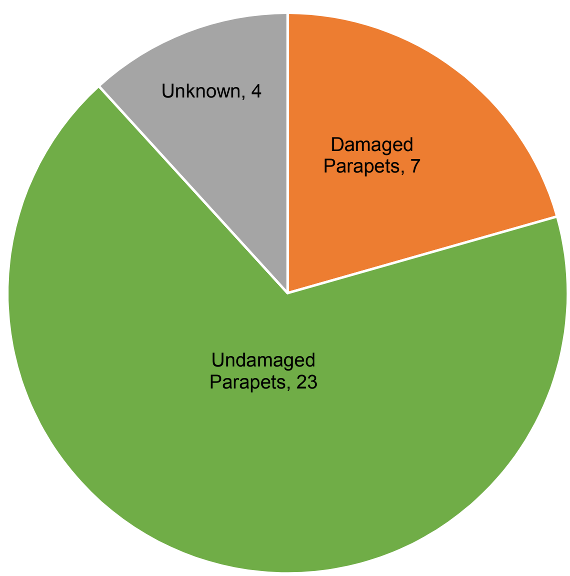


Figure 3-10 Distribution of parapet performance.

For the 68 buildings in the study area, no damage to the nonstructural components was reported for 13 buildings (19%), and insignificant damage was reported for 27 buildings (40%). A total of 13 buildings (19%) sustained minor nonstructural damage and 7 buildings (10%) sustained moderate damage. The level of nonstructural damage in 8 buildings could not be determined.

Of the 27 buildings with insignificant damage, 11 were URM structures. Eleven buildings sustained some damage to glazing, and four of these were estimated to have lost 25% or more of their glazing area. The partitions and ceilings in the majority of the buildings sustained either no damage or insignificant damage.

In the 13 buildings with minor nonstructural damage, damage to ceilings and lights was most common. Three buildings sustained minor, seven buildings sustained moderate, and two buildings sustained heavy ceiling damage. The majority of these buildings sustained no or insignificant partition damage. Two buildings sustained damage to 50% or more of their exterior glazing. Of the seven buildings with moderate nonstructural damage, four sustained some damage to partitions, and five had damage to ceilings.

A total of 32 buildings (47%) had fire sprinkler systems. Damage was reported in five buildings. In one case, a number of short pipe hangers failed in an unbraced sprinkler system, but there was no loss of water. The sprinkler system developed a leak in one building, causing damage to an ornamental plaster ceiling. Sprinkler failures in three of the buildings resulted in flooding and significant damage to floor coverings and partitions.

3.11 Injuries or Fatalities

No injuries or fatalities were reported in the buildings surveyed.

3.12 Functionality

In this survey, full functionality is defined as all space in the building being usable in its pre-earthquake function. In most cases, loss of functionality resulted from damage to contents, nonstructural components and systems, or structural members. Given the commercial character of the area, full functionality was often synonymous with business operation. In a few cases, adjacency of the building to a damaged building delayed full functionality until the building was shielded from falling hazards.

The surveys were conducted between 12 and 16 days following the earthquake, so responses of “immediate,” “1-3 days,” and “within 1 week” correspond to buildings that were fully functional at the time of the survey. In these cases, the length of time is based on conversations with occupants of the building and direct observation of the amount of usable space. Responses of “within one month” and “within 6 months” correspond to those building not fully functional at the time of the survey. For these buildings, the time to return to full functionality is a projection based on conversations with the occupants and the surveyors’ observations of the damage extent and realistic repair time. For roughly a third of the buildings (34%), adequate information to record or estimate the length of time was not available and as a result, no information regarding return time to full functionality was provided for these buildings. It is likely that many of these buildings without adequate information were not functional after the earthquake and that the surveyor chose not to provide as estimate because they did not have enough information.

For buildings where information was provided, less than one half of the buildings (44%) remained fully functional through the earthquake. Within one week of the earthquake, many buildings were able to make the necessary minor repairs, raising the percent of fully functional buildings to 89%. Based on projections by the surveyors, the number of fully functional building levels off with 91% building within one month and 98% of building full functional within 6 months. Figure 3-11 shows the duration to return to full functionality for buildings with information and estimates provided.

One older county office building had to enact an asbestos abatement program after fire sprinkler damage revealed the presence of asbestos.

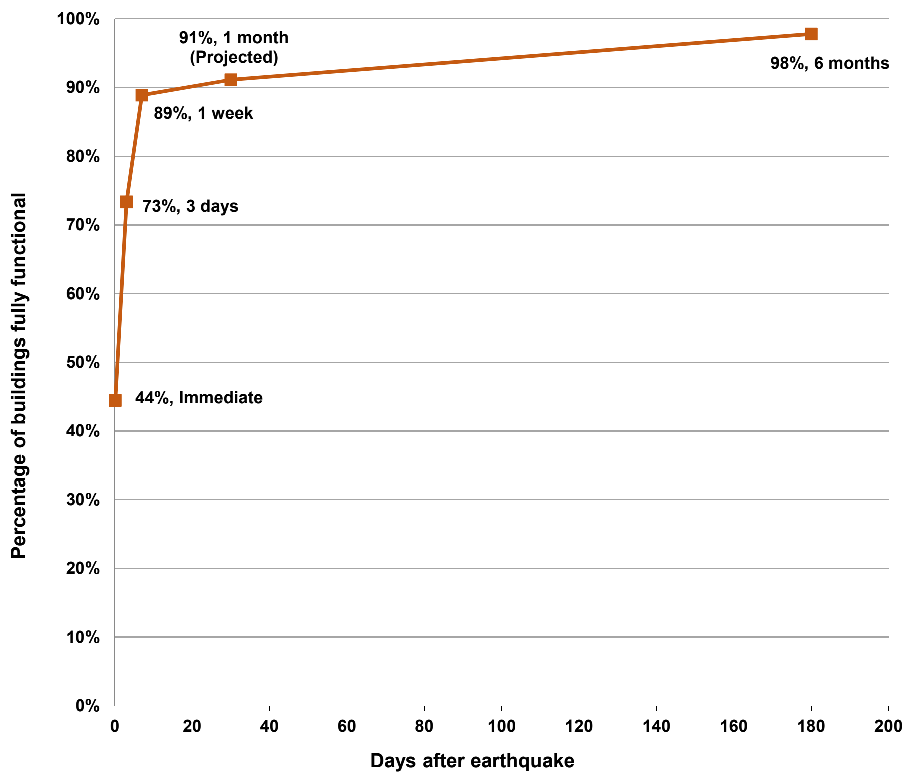


Figure 3-11 Building functionality: duration (in days) after earthquake to resume full functionality.

Chapter 4
Performance of Selected Buildings

4.1 Introduction

In this chapter, case studies documenting the performance of buildings are presented. The buildings described in this chapter were chosen to illustrate representative examples of seismic performance and include structures that sustained significant damage, as well as some that sustained little or no damage. The buildings are organized into the following groups of structures:

- Newer construction: Any building designed and constructed using the 1998 or later edition of the *California Building Standards Code* (California Building Standards Commission, 1998)
- Older construction, excluding unreinforced masonry (URM) construction
- Unretrofitted URM construction
- Retrofitted URM construction with significant damage
- Retrofitted URM construction with limited damage

Table 4-1 provides a summary of the buildings discussed in this chapter. The chapter concludes with a summary of building performance observations and recommendations for improvements and further study.

Table 4-1 Summary of Case Studies

Structure Group	Section	Number of Case Studies	Number of Buildings within 1,000 ft Survey Area
Newer Construction	4.2	3	1
Older Construction, not URM	4.3	7	4
URM Construction, not Retrofitted	4.4	2	2
URM Construction, Retrofitted	4.5 and 4.6	10	10

Seventeen of the 22 buildings described in this chapter were located within the 1,000 foot radius around Station N016. Observations and performance of all buildings were recorded using the modified Postearthquake Building Performance Assessment Forms provided in Appendix A during field visits conducted over several days, two weeks following the earthquake. Each subsection describing a building summarizes information based on the form. The location of each building is designated with an alphanumeric code in the section titles. Buildings coded A-N were located within the 1,000 foot radius around Station N016, and are shown on Figure 4-1. Buildings coded Z were located outside of the 1,000 foot radius and their approximate locations are shown in Figure 4-2. It is noted that more detailed investigations were conducted for some buildings, and information gained is provided in the text.

Figure 4-1 Key map providing alphanumeric designations for buildings within the 1,000 foot radius around Station N016 (image source: Google Maps).

Figure 4-2 Key map providing alphanumeric designations for buildings investigated but located outside of the 1,000 foot radius around Station N016 (image source: Google Earth). Building Z4 is not shown.

4.2 Newer Construction

For the purposes of this report, newer construction is defined as any building designed and constructed using a 1998 or later edition of the *California Building Standards Code*, which was based on the 1997 *Uniform Building Code* (ICBO, 1997). The most current edition of the *California Building Standard Code* is based on the 2012 *International Building Code* (ICC, 2012a). Generally, newer construction sustained little or no structural damage. However, nonstructural damage resulted in the closure of several buildings, some of which will be closed for months.

4.2.1 Three-Story Commercial Building, Main Street (Building L1)

- **Structural System, Height, Year Built:** The building is a three-story tall steel concentric-braced frame structure with a wood-framed roof. An overview of the building is shown in Figure 4-3. The construction date is estimated as 2002.

Figure 4-3 Exterior view of building.

- **Occupancy Type:** Retail and offices on the ground floor, offices on the upper floors.
- **Posting Placard:** At the time of the field investigation, the building was posted RESTRICTED USE. The City of Napa website reported damage at the north wall of the third floor, and a change in elevation of the floor in an office on the second floor. There was no access to the office at the time of the investigation and no observable distress was noted in the vicinity.
- **Structural Performance:** No significant damage was observed. A brace on the north side of the building buckled elastically out-of-plane, causing some damage to the light-frame furring wall (Figure 4-4).
- **Nonstructural Performance:** In general, very little nonstructural damage to the building was observed. There was almost no drywall damage observed on the first floor, and no ceiling damage reported. As noted above, there was damage to the light-frame furring at the third floor north wall, in the vicinity of one brace. Along the north wall, damage to the exterior wall occurred when the out-of-plane wall anchors of the adjacent building struck the drywall (Figure 4-5). No damage to equipment or piping, including the fire sprinkler system, was reported.
- **Time until Full Occupancy:** The building remained operational.

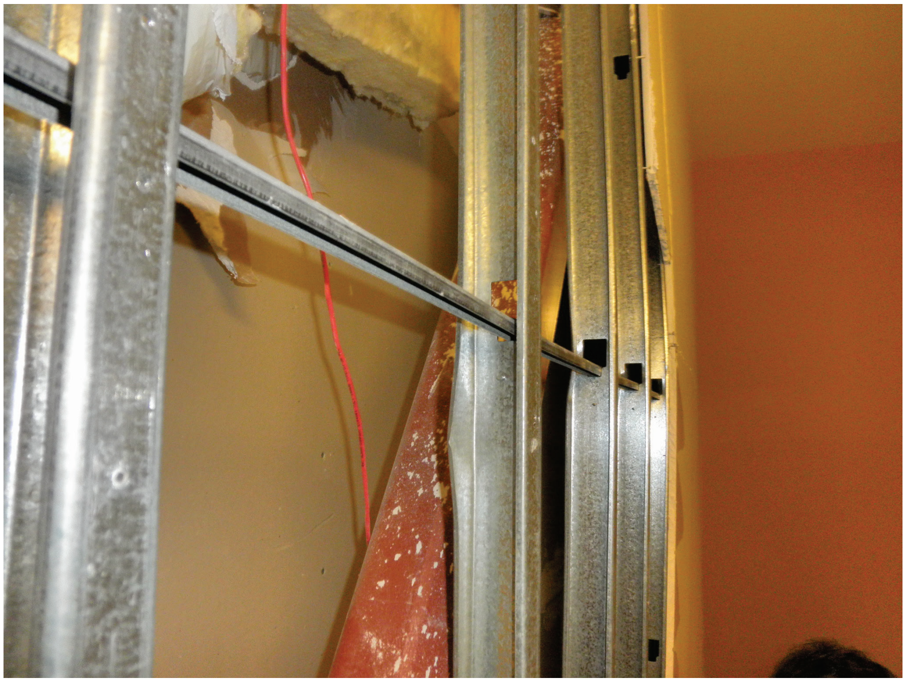


Figure 4-4 Damage to steel studs due to out-of-plane buckling of brace.

Figure 4-5 Out-of-plane wall anchor of adjacent building penetrated drywall.

4.2.2 Three-Story Office Building, Franklin Street (Building Z1)

- **Structural System, Height, Year Built:** The structural system consists of a special welded steel moment frame with reduced beam sections (RBS). The building is three stories tall with basement parking, constructed in 2008. The front elevation is shown in Figure 4-6. The building is constructed with metal deck and concrete fill at the floors; the roof is constructed with untopped metal deck. Typical cladding consists of balloon-framed metal studs with gypsum fire sheathing and Portland cement plaster with stone veneer on foam substrate and solid stone veneer near the base.

Figure 4-6 Exterior view of building.

- **Occupancy Type:** Office.
- **Posting Placard:** The building was posted UNSAFE immediately after the earthquake, but at the time of the field investigation, the building was posted INSPECTED.
- **Structural Performance:** There was no reported structural damage. One RBS connection was exposed for examination approximately six weeks after the earthquake. No damage was observed at the connection.
- **Nonstructural Performance:** The south exterior wall of the building, which contains large sections of solid wall, suffered the most significant damage (Figures 4-7 and 4-8). The wall, constructed with metal studs sheathed on the exterior with 5/8 inch gypsum fire-rated wallboard and approximately 1 1/4 inch of cement plaster (Figure 4-9), does not appear to have been detailed to accommodate story drift through a sliding or

yielding mechanism. Consequently, the wall suffered several types of damage due to in-plane displacement, including failure of connections at the floors causing separation and bowing of the wall (Figure 4-10 and 4-11), distressed studs, and plaster and gypsum wallboard cracking (Figure 4-12). In addition, the following damage was reported and/or observed: (1) Some adhered cast-stone foam-backed veneer and stone veneer became dislodged (Figure 4-12); (2) a water heater at the second floor, which was reportedly strapped, became dislodged, broke a water pipe and caused water to flow into an electrical room below where metering equipment suffered water damage; and (3) the glass in an interior room partition was broken.

Figure 4-7 Damaged studs in exterior wall.

- **Time until Full Occupancy:** The building was closed for several days following the earthquake and partially reopened within one week after the event. Portions of the building were vacated for three months to conduct detailed damage investigations of the south wall and to conduct related repairs.
- **Other Notes:** This building is located outside of the 1,000 foot radius around Station N016. It was reported that the south wall was required to have a three-hour fire rating because of the proximity to an existing adjacent building, and that this heavier than normal construction may have contributed to the damage.

Figure 4-8 Bowed wall separated from floor (photo on right from Lauren Biscombe).

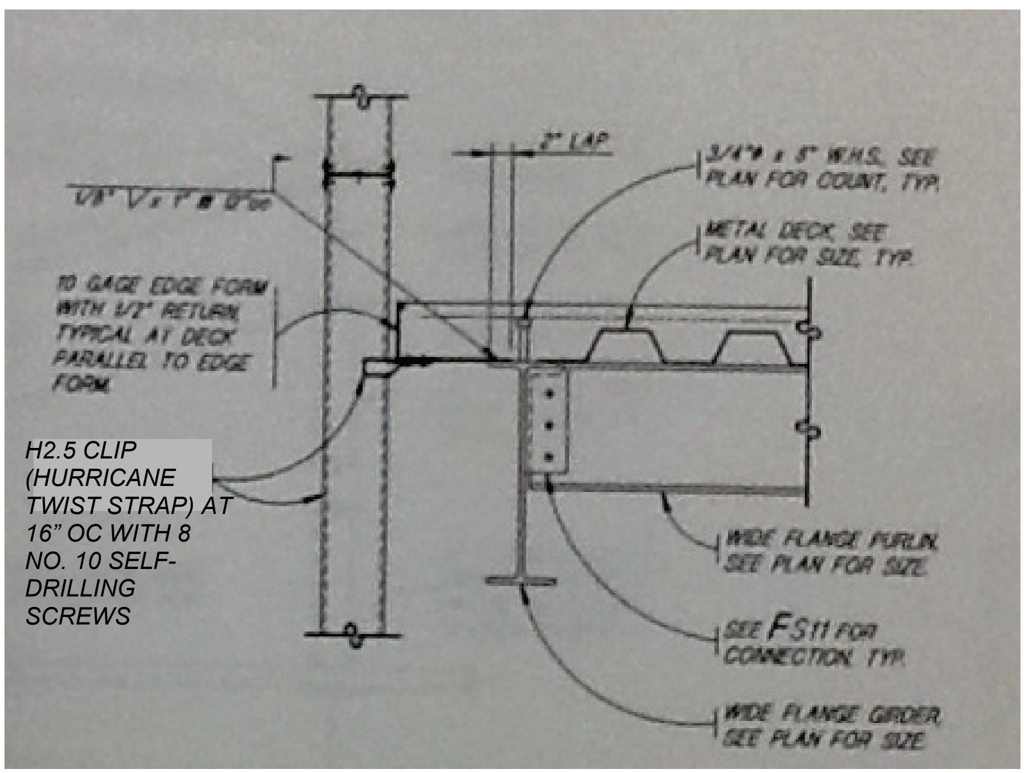


Figure 4-9 Detail from original drawings showing connection of exterior wall to each floor.

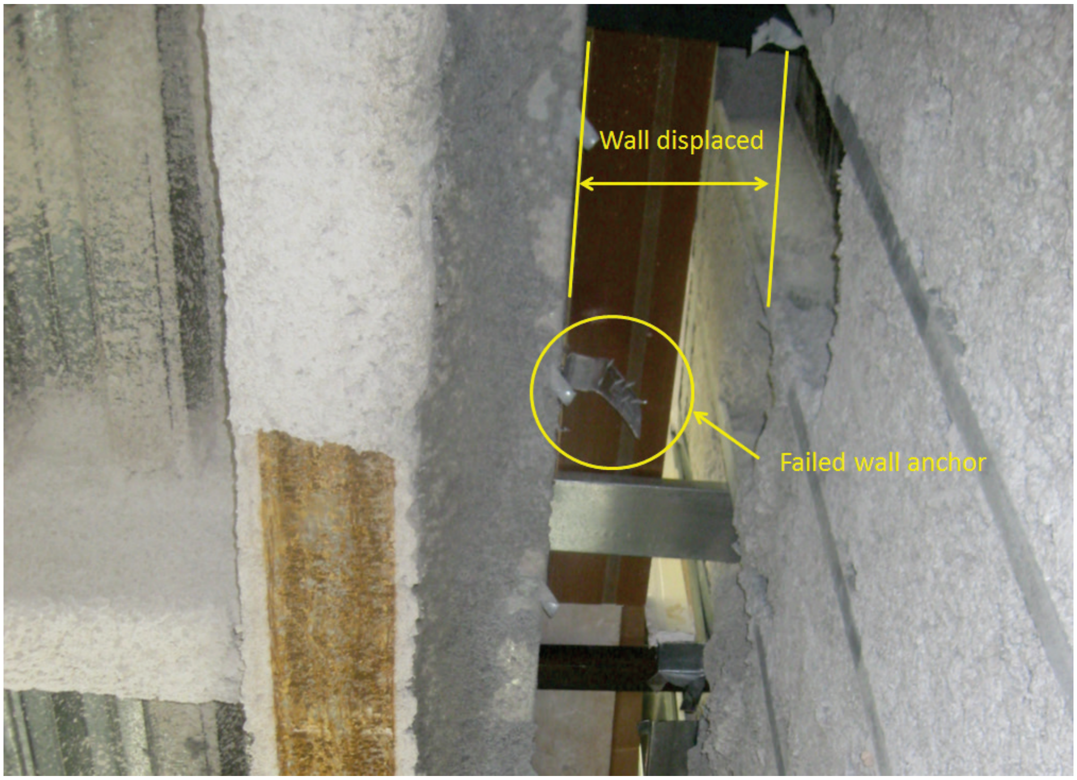


Figure 4-10 Failed wall anchor at bowed wall.

Figure 4-11 Exterior wall, cracked plaster.

Figure 4-12 Dislodged adhered veneer.

4.2.3 Five-Story Hotel, 1st Street (Building Z5)

- **Structural System, Height, Year Built:** This building is a five-story hotel, built in 2009. The front elevation is shown in Figure 4-13. It is rectangular in plan with an east-west longitudinal length of 254 feet and a north-south transverse length of 131 feet. A projection at the northeast corner is 26 feet by 55 feet. There is street frontage on the south and east sides, with car access through a tunnel under the northeast projection to a rear, north alley. There is a seismic joint between a building to the north and the northeast projection of the structure. The first floor is at street grade.

  Concrete bearing walls and columns and concrete masonry unit (CMU) walls at the first story support a post-tensioned concrete podium at the second floor. Interior walls bear on mat foundations; perimeter walls and columns bear on spread footings and perimeter grade beams. A number of the interior bearing walls continue up to the roof level. The third floor through roof level diaphragms are concrete fill on metal deck supported by the cold-formed stud bearing walls, light steel W-beams, concrete walls, and CMU walls. Steel tube columns in the stud walls support the ends of the W-beams. Lateral loads are resisted by the concrete and CMU shear walls. Above the podium level, shear walls are generally configured eccentrically to the floor plate and two street façades.

  The structural drawings indicate that the structural design used the 2001 *California Building Standards Code* (California Building Standards Commission, 2001), which would be equivalent to the 1997 *Uniform*

Figure 4-13 Exterior view of building.

Building Code (ICBO, 1997a). The building is clad with stone at the lower stories and stucco at the upper stories. The cladding is supported by concrete at the base and steel studs at upper stories.

- **Occupancy Type:** Hotel and restaurant at the first story; hotel at the upper four stories.
- **Posting Placard:** The building was posted RESTRICTED USE following the earthquake due to nonstructural falling hazards posed by loose stone veneer. The posting remained at the time of the field investigation.
- **Structural Performance:** Cracking of a coupling beam over the door in the concrete tower around one stairwell was observed (Figure 4-14). Repairs included epoxy injection in cracks.
- **Nonstructural Performance:** Substantial cracking at the stucco cladding and damage to the stone cladding were sustained (Figures 4-15 and 4-16). It is estimated that less than 1% of the veneer became

Figure 4-14 Cracking in spandrel in concrete stair core.

dislodged during the earthquake. After the earthquake, stone cladding was sounded with hammers and stones identified as loose were removed as part of repairs (Figure 4-17). The architectural drawings show control joint locations in the stucco and mortar joint locations for the stone. The architectural drawings also show anchored veneer, and the specifications require the anchors to accommodate vertical and horizontal movement between the stone and structural backing. However, based on field observations of damaged areas, it appears that veneer above the base was adhered, and there were no installed details that would explicitly allow for story drift in the stone cladding or stucco.

The most significant damage was caused in the restaurant area by a single sprinkler head that impacted an adjacent beam and was triggered. This released water onto both wood and marble flooring (Figure 4-18) for several hours, causing enough damage that required the floor to be completely replaced.

The seismic switch for the elevator was triggered, but the elevator was later brought back into use. Some ceiling tiles fell down. An emergency generator fell off its supports and severed a gas line, but the seismic shutoff to the gas line was triggered, preventing loss of gas and potential fire. There were cracks in the gypsum board partition finishes. Marble flooring tile was loosened. Mirrors above sinks fell and then damaged the finishes upon which they fell. Forty mirrors and sinks were identified for repair or replacement.

Figure 4-15 Adhered veneer damaged during the earthquake.

Figure 4-16 Photo showing failed anchored veneer.

Figure 4-17 Stone masonry veneer, showing where loose units have been removed.

Figure 4-18 Sprinkler impacted HVAC components and discharged.

- **Time until Full Occupancy:** Following the earthquake, hotel guests were transferred to other hotels in the area. The building was insured for earthquake damage and business interruption costs. Repairs took several months and the hotel was partially reopened (three out of the four floors with rooms) on December 15, 2014, approximately four months after the earthquake. Independent retail businesses on the first story of the hotel remained open. It is important to note that building closure was entirely related to nonstructural damage.

- **Other Notes:** This building is located outside of the 1,000 foot radius around Station N016.

4.3 Older Construction Excluding URM

This section describes the performance of buildings other than unreinforced masonry bearing wall structures constructed prior to the adoption of the 1998 edition of the *California Building Standards Code*. Damage observed in these buildings generally highlighted known vulnerabilities.

4.3.1 Three-Story Commercial Building, 1st Street (Building I1)

- **Structural system, height, year built:** The building is a three-story tall steel frame structure with URM infill. An overview of the building is shown in Figure 4-19. The building was constructed in 1914.

Figure 4-19 Exterior view of building.

- **Occupancy Type:** Retail on ground floor, offices and public meeting rooms on upper floors.

- **Posting Placard:** At the time of the field investigation, the building was posted RESTRICTED USE due to falling debris.

- **Structural Performance:** No significant structural damage was reported, this speaks very well for a 100-yr old structural system.

- **Nonstructural Performance:** A few bricks fell from the façade, penetrating the cold-formed steel canopy over the sidewalk (Figure 4-20). About 30% of the windows in the first floor storefront were damaged. Damage to lath and plaster wall finishes was widespread. There was minor damage to suspended ceilings in some areas (Figure 4-21). A support leg of a commercial convection oven failed (Figure 4-22). Unanchored and marginally anchored rooftop equipment displaced but did not fail. No damage to the fire sprinkler system was reported.
- **Time until Full Occupancy:** Portions of the building were inaccessible to the public following the earthquake due to fallen debris. At the time of the field investigation, some of the retail establishments on the ground floor were open. The use of the upper floor spaces was restricted due to nonstructural damage.

Figure 4-20 Damage to canopy due to falling brick.

Figure 4-21 Minor damage to suspended ceilings.

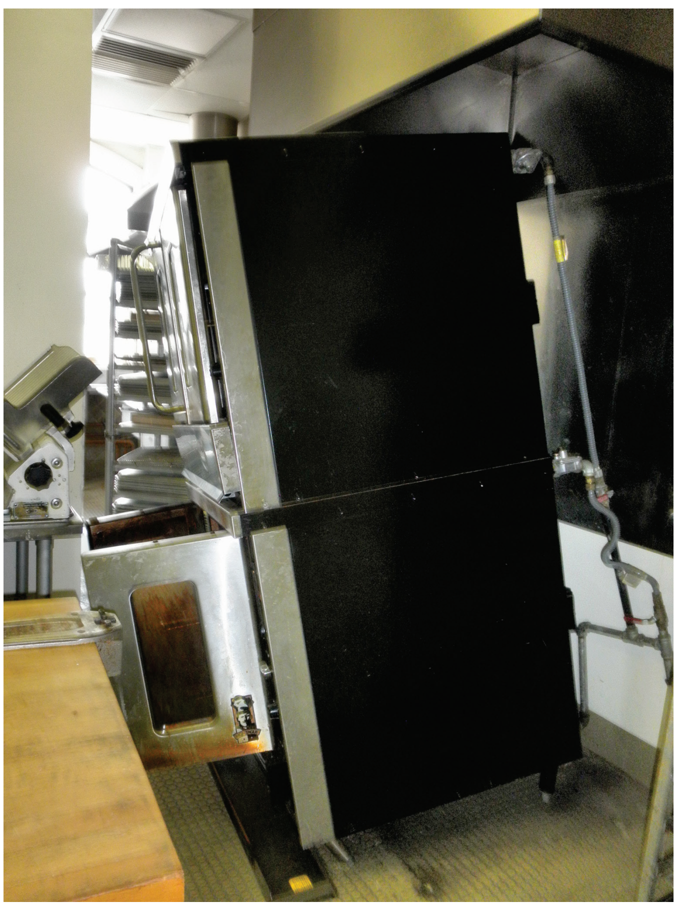


Figure 4-22 Convection oven with failed support leg.

4.3.2 Historic Post Office Building (Building Z3)

- **Structural System, Height, Year Built:** The structural system consists of a steel frame with brick masonry infill. The extent of steel framing is unknown but steel corner columns were exposed by masonry damage, and the column-free interior suggests the presence of steel roof trusses. The building was dedicated in 1933. The building is one-story tall with a partial mezzanine and full basement. The front, north elevation of the building is shown in Figure 4-23.

Figure 4-23 Exterior view of building.

- **Occupancy Type:** United States post office.

- **Posting Placard:** At the time of the field investigation, the building was posted UNSAFE.

- **Structural Performance:** Masonry piers sustained heavy damage on all elevations, but worse on front and rear sides. Corner piers sustained major inclined cracking two inches or more in width, rear elevation piers had cracking in excess of 1/2 inch (Figure 4-24). Some other piers shifted laterally at the sill line (Figure 4-25). Short segments of exposed corner columns indicate that they may be bowed or no longer plumb. No damage was reported in the basement.

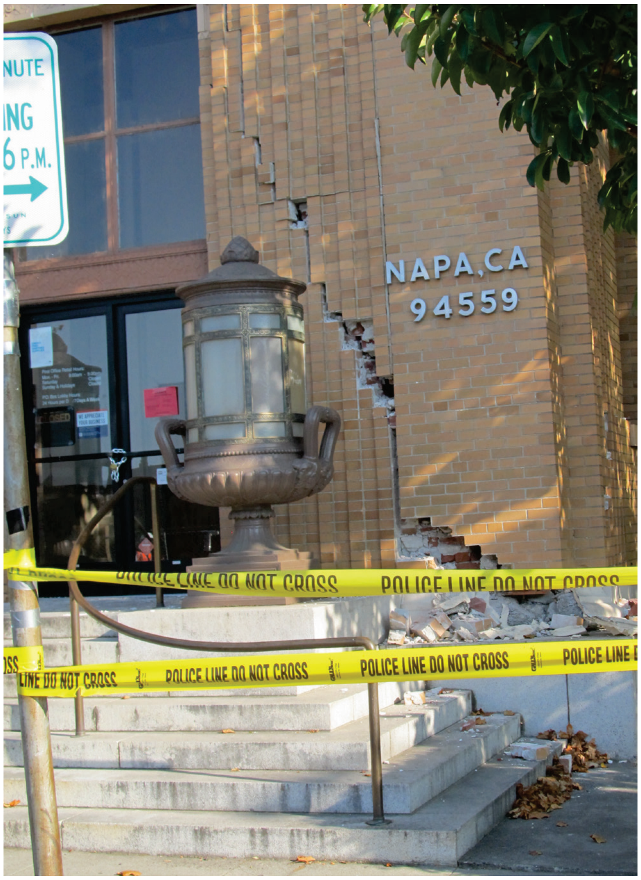


Figure 4-24 Damage to corner pier.

Figure 4-25 Lateral shifting at sill line.

- **Nonstructural Performance:** Significant cracking was observed to the exterior brick veneer of the building, which is mostly a true veneer with anchor ties. Interior finish on exterior wall and interior partitions are hollow clay tile with plaster. These were heavily damaged, especially near the front corner piers, with significant fallen clay tile units and debris. Plaster ceilings were damaged, especially in the vicinity of the corner piers. Heavy glazing damage was also observed (Figure 4-26).

Figure 4-26 Glazing damage.

- **Time until Full Occupancy:** At the time of the field investigation, the building was inaccessible to the public following the earthquake due to damage. The building remained closed six monhs after the earthquake.
- **Other Notes:** This building is located outside of the 1,000 foot radius around Station N016. It was reported that during the 2000 Yountville earthquake, numerous windows on the front elevation were broken.

4.3.3 One-Story Retail Building, Clay Street (Building Z6)

- **Structural System, Height, Year Built:** The store is a one-story, precast tilt-up concrete shear wall building with a flexible roof diaphragm constructed in 1973. The roof is constructed with plywood sheathing, 2x purlins and glulam beams supported by exterior concrete bearing walls and interior steel columns. The tilt-up panels are interconnected with cast-in-place pilasters. Three sides of the building have nearly solid concrete walls; the front has concrete walls on both sides of the storefront. There have been no significant modifications to the structural system since the time of construction.

- **Occupancy Type:** Retail store.
- **Posting Placard:** Building was inspected by a structural engineer on the day of the earthquake and evacuated until emergency shoring was installed. On August 26, 2014, following the installation of out-of-plane wall bracing (Figure 4-27) and shoring of damaged glulam beams, the building was posted RESTRICTED USE.

Figure 4-27 Shoring of wall with damaged pilasters.

- **Structural Performance:** Five glulam beam-to-pilaster connections on the north wall of the building suffered damage at the pilaster connection (Figure 4-28). The top of the pilaster spalled or cracked around the anchor bolts. One mezzanine support beam experienced some damage at its connection to the rear wall.
- **Nonstructural Performance:** One unbraced sprinkler line broke (Figure 4-29), and water ran for several hours. The lightly strapped hot water heater overturned breaking a water line but not the gas line, which had a flexible connection. There was damage to refrigeration equipment. The suspended acoustic tile ceiling was severely damaged, including substantial loss of tiles and failures in the grid, requiring replacement of

Figure 4-28 Damaged pilaster-to-roof beam connection.

the entire ceiling (Figure 4-30). Some light fixtures were dislodged at their connections to the struts supporting them. Some unanchored store fixtures shifted or overturned. At one aisle, all of the fixtures along one side overturned into the aisle; there was substantial loss of contents from shelves, and virtually all the product was lost (Figure 4-31).

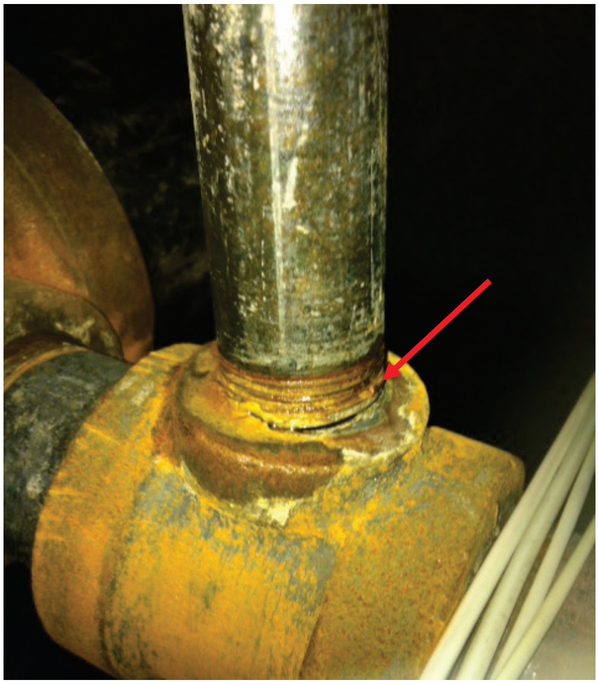


Figure 4-29 Cracked sprinkler pipe.

Figure 4-30 Suspended ceiling damage.

Figure 4-31 Overturned store fixture, loss of contents.

- **Time until Full Occupancy:** At the time of the field investigation, it was reported that the building would be out of service for at least six months. Later it was reported that the store may be moved to a new location.

- **Other Notes:** This building is located outside of the 1,000 foot radius around Station N016. The total cost of repairs is estimated to be approximately $1.5 million (approximately $500,000 attributed to structural repairs and shoring).

4.3.4 Napa County Administration Offices (Building F2)

- **Structural System, Height, Year Built:** This three-story tall building has a welded steel-braced frame structure with a basement. The building has plan irregularity (re-entrant corners). The construction date is estimated as 1970s.
- **Occupancy Type:** Office.
- **Posting Placard:** The building was not posted.
- **Structural Performance:** A small number of weld fractures in the steel braced frames were reported, but there were no additional reports of significant structural damage.
- **Nonstructural Performance:** Several of the large curb-mounted rooftop HVAC units and other rooftop equipment shifted as a result of anchorage failures. Movement of the equipment caused domestic water pipe breaks (Figure 4-32) in the penthouse resulting in water damage to the third floor ceilings. Most of the piping in the penthouse was unbraced. As a result of water damage, ceiling finishes in the hearing room collapsed. Some of the water damage to the ceilings was associated with failure of brazed/soldered piping connections to coils in the ductwork (Figure 4-33). The building does not contain a fire sprinkler system. Electrical components in the penthouse suffered water damage.

  There was extensive ceiling damage in some rooms of the third floor (Figure 4-34), with progressively less ceiling damage at each floor lower. At the third floor, about 20% of the grid was damaged, including some light fixtures that detached from the grid, although safety wires kept them from falling. In some areas, ducts disconnected from independently supported in-line HVAC components (Figure 4-33).

  Damage to gypsum board partitions spanning floor-to-floor was generally minor. Some modular partitions on the third floor came out of the top track and fell over, and were subsequently removed (Figure 4-35). A reported failure mode was the partitions coming off their top tracks. Minor cracking of ceramic tile on the floor slabs was observed, as was minor damage to exterior insulation finishing system (EIFS) cladding.

Figure 4-32 Evidence of pipe motion in penthouse partition wall.

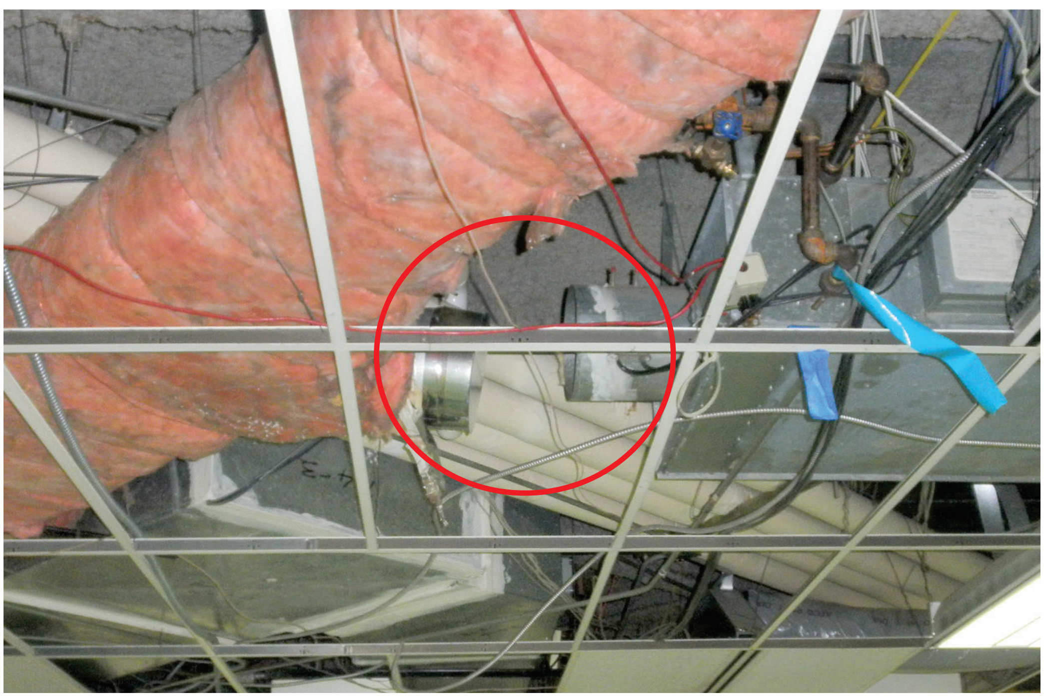


Figure 4-33 Ceiling and duct damage. The duct separated from the in-line HVAC unit to the right.

Figure 4-34 Ceiling removed due to water damage, third floor.

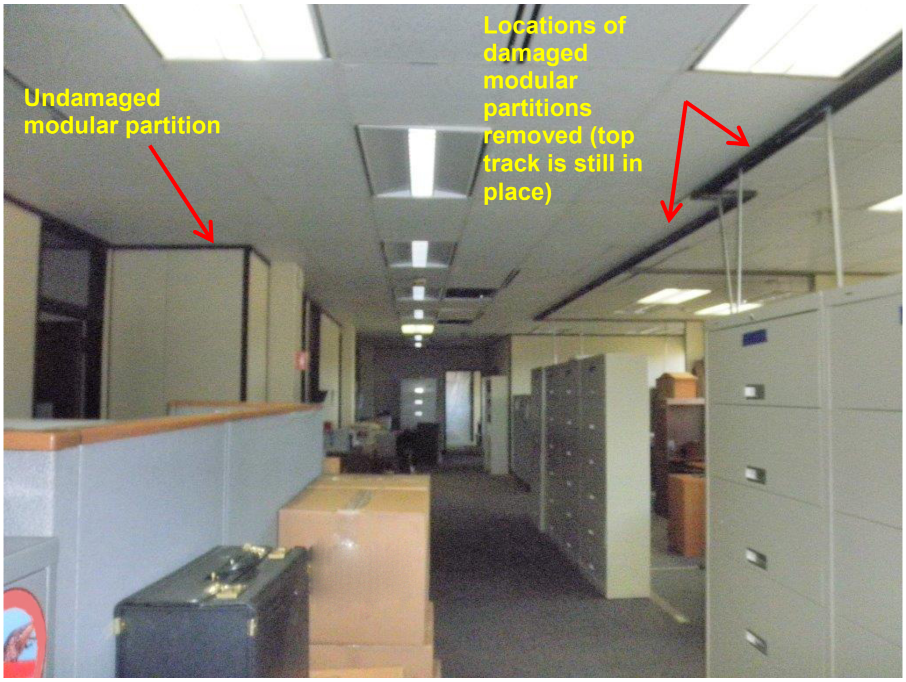


Figure 4-35 Third floor of building after the earthquake. Modular partitions similar to those at the left were damaged and removed.

- **Time until Full Occupancy:** The elevators were nonfunctional for two days. Portions of the building were unavailable following the earthquake due to water damage. At the time of the field investigation, two weeks after the earthquake, the third floor was still unoccupied.

4.3.5 Napa County Hall of Justice (Building F3)

- **Structural System, Height, Year Built:** The building consists of two wings with reinforced concrete frames with CMU shear walls (Figures 4-36 and 4-37). The building is three stories tall with a basement. One wing was constructed circa 1974, and has a concrete waffle slab floor system and precast concrete window units on the exterior. The penthouse of the 1974 wing is a steel frame structure with tension rod bracing. The second wing was built circa 1989, and has concrete flat slab floor systems.

Figure 4-36 Exterior view, 1974 wing.

- **Occupancy Type:** Correctional facility and offices.

- **Posting Placard:** At the time of the field investigation, the building was posted RESTRICTED USE. The City of Napa website reported visible damage in the court tunnel, and impairment of the domestic water system.

- **Structural Performance:** Minor X-cracking in the first floor CMU shear walls was observed, along with minor spalling of the CMU face shells at the ends of the walls (Figure 4-38). Pounding damage was observed between the 1974 and 1989 wings (Figure 4-39). In the 1974 wing, some damage was noted in the vicinity of the precast concrete panels containing the windows (Figure 4-40). These units are integrated into the CMU walls, without any allowance for isolating the panel from the effects of story drift. The tension rods in the penthouse reportedly failed, but had been repaired or replaced at the time of site visit.

Figure 4-37 Exterior view, 1989 wing.

Figure 4-38 CMU face shell spalling, typical at base of walls.

Figure 4-39 Floor finish damage due to pounding at the seismic joint indicated by the arrow.

Figure 4-40 CMU damage at precast concrete panels containing the window.

- **Nonstructural Performance:** Significant nonstructural damage was noted in the 1974 wing penthouse. Nearly all vibration-isolated components in the penthouse failed (Figure 4-41). Many failures were associated with splitting of the unreinforced concrete housekeeping slabs

(Figure 4-42), resulting in anchorage failures. The gas-fired emergency generator, which suffered vibration isolator failures, fell from its mounts and displaced laterally, but continued to function. The unbraced batteries for the generator slid but remained functional. A motor control center displaced several feet, but the conduits entering the unit at the top kept it from toppling. It was reported that four of the ten duct drops in the penthouse failed, but the nature of the failure could not be confirmed. Unbraced vibration isolated axial flow fans in-line with the ducts broke free from the ducts but did not fall. A mushroom fan on the roof toppled. Pumps failed their isolator mounts and were restrained by the pipe drops, and one pump pounded against an adjacent partition wall. Ducts and conduit crossing the seismic joint between the wings were not provided with flexible connections and some failed. Damage to suspended ceilings was noted (Figure 4-43), but no general failures of the grid were observed. On the third floor, some of the damage to the ceilings was associated with failure of brazed/soldered piping connections to coils in the ductwork. Damage to gypsum wallboard partitions due to story drift was generally minor. Some disruption to the hydraulic elevators due to buckled gypsum wallboard in the shafts was reported.

Figure 4-41 Anchor failure, rooftop cooling tower 1974 wing.

Figure 4-42 Housekeeping slab failure, chiller in 1974 wing penthouse.

Figure 4-43 Ceiling damage at seismic joint.

The nonstructural components in the 1989 wing performed significantly better than those in the 1974 wing. No problems with piping connections to in-line coils were reported. No sprinkler issues were reported.

- **Time until Full Occupancy:** Building remained operational.

4.3.6 Two-Story Retail Building, Napa Center (Building Z2)

- **Structural system, height, year built:** The store is a two-story building of mixed construction. Figure 4-44 shows an overview of the building. The roof is sheathed with plywood supported by 2x wood framing, glulam beams, and steel columns. The second floor includes gypsum concrete fill on plywood supported by manufactured trusses, glulam beams, and steel columns. The lateral system includes pre-Northridge moment frames, chevron-braced frames, and plywood shear walls. Tenant improvements were constructed in 1987. The date of building construction is not known, but estimated as mid-1980s.

Figure 4-44 Exterior view of building.

- **Occupancy Type:** Retail.
- **Posting Placard:** The building was posted RESTRICTED USE following the earthquake. A visible placard was not observed during the field visit.
- **Structural Performance:** There was no reported structural damage reported or observed.
- **Nonstructural Performance:** Water was released from five lines suspended from the roof for several hours (Figure 4-45). The sprinkler piping has threaded connections and damage appears to have been related to swaying and interaction with adjacent HVAC components (Figure 4-46). The elevator and escalator pits were flooded, and both systems required repair. Contents were damaged as a result of water;

some merchandise racks were overturned. Considerable amounts of gypsum wallboard were water damaged and required removal. The suspended acoustic tile ceiling was severely damaged, including damage to the grid ("free" ends came off ledger angle supports, "fixed" ends pulled out of the wall, and splices failed). A distance of up to approximately 70 feet was measured between parallel fixed- and free-ends of the ceiling assembly. The ceiling contained splay wire bracing, but no compression posts. Suspended lights were installed with independent wire supports. Partitions were braced by the ceilings and may have contributed to the ceiling damage at some locations. Anchored air-handling units on the roof reportedly slid off their supports.

Figure 4-45 Water pressure from broken sprinkler line created a hole in the exterior cladding (photo by KCRA.com).

- **Time until Full Occupancy:** This building remained closed six monhs after the earthquake.
- **Other Notes:** This building is located outside of the 1,000 foot radius around Station N016.

Figure 4-46 Damaged sprinkler pipe. Damage believed to have been caused by interaction with adjacent air handler.

4.3.7 Napa County Courthouse, Brown Street (Building D1)

- **Structural System, Height, Year Built:** The courthouse is comprised of three different structures. The west end is a 1916 concrete shear wall building. The central portion was built in 1977 and is a reinforced CMU infill building with concrete fill on metal deck floors supported by open web joists. The east end is the original URM historic courthouse built in 1856 (Figure 4-47). The main level of the original courthouse is raised above grade; a ramp has been added for accessibility. The east façade has a series of setbacks in plan at the northeast and southeast corners, creating a series of short orthogonal walls. It is owned by Napa County, but houses State of California Court functions. The buildings form a rectangle in plan with the long direction of approximately 203 feet oriented east-west and the transverse direction of approximately 95 feet oriented north-south. There are two stories above ground in the 1856 and 1916 portions and three stories in the 1977 portion. The building is set back from the street on the north, east, and south sides and abuts the sidewalk on the west side. At the time of field visits, it was surrounded with a fence set back a substantial distance from the north, east, and west façades.

  It was reported that in 1977, the URM building was partially retrofitted, and the central CMU structure was added. The CMU structure is seismically separated from the URM building.

Figure 4-47 East façade of historic URM courthouse. Note fallen bricks below the damaged portion at the top of the wall.

- **Occupancy Type:** Office and courthouse.
- **Posting Placard:** The historic courthouse structure was posted with an UNSAFE placard. During a site visit on September 9, 2014, the concrete structure on the west side was posted with a RESTRICTED USE placard with the following annotation: "Damage to the east side of building. OK to use this entrance to secure building. Must be evaluated by an engineer." Subsequently, the 1916 and 1977 portions of the building have been reopened.
- **Structural Performance:** No structural damage was reported in the west (1916) and central (1977) portions of the structure. The east (1856) URM structure suffered extensive diagonal cracking to spandrels on the north-south façade, especially at the roof level, with heavier damage concentrated in masonry above the second floor and in the corner piers (Figure 4-48). There was partial collapse of the four-wythe brick attic wall at the southeast corner just below the roof. It was observed that roof-to-wall ties were no longer connected to the walls. Some minor damage was visible where the URM structure meets the CMU structure.
- **Nonstructural Performance:** Significant damage to ceilings and plaster wall finishes in the URM structure, as well as water damage, were reported. The building was not accessible for field investigations, thus details are not known.

Figure 4-48 Close-up of damage at top of the wall.

- **Time until Full Occupancy:** At the time this report was being developed, the URM structure remained closed. The county court website (Napa County Court, 2015) notes: “After considerable remediation, the court has resumed partial operations in the newer portion of the Historic Courthouse… However, due to significant seismic damage to the historic portion of the building, including the front entrance, Departments A, B and N, as well as the Civil and Family Law Clerk’s Office, will continue to be closed indefinitely while it undergoes structural testing and analysis, and ultimately repair and reconstruction.”
- **Other Notes:** The 1856 URM structure was damaged and closed following the 2000 Yountville earthquake.

4.4 Unretrofitted URM Construction

Unretrofitted URMs generally performed poorly, and would have caused many deaths and injuries had the earthquake occurred at a time when pedetstrians were walking on the streets. However, some unretrofitted URMs suffered only little or no damage as a result of immediate proximity to other newer or retrofitted buildings on each side, allowing them to effectively “lean” on their neighbors, the relatively short the duration of shaking, and in some cases, for reasons that have not been identified.

4.4.1 Two-Story Commercial Building, Brown Street (Building E1)

- **Structural System, Height, Year Built:** The building is a two-story URM building with a flexible roof diaphragm (Figure 4-49). It was constructed in 1904.

Figure 4-49 Failure of URM south wall (photo courtesy of Marko Schotanus).

- **Occupancy Type:** Office.
- **Posting Placard:** At the time of the field investigation, the building was posted UNSAFE. It is reported that due to damage to this building and an adjacent structure, four surrounding buildings (Buildings E4, E9, E10, and E11) were also posted UNSAFE due to the risk of collapsing masonry walls striking the building. The surrounding buildings were subsequently posted INSPECTED following the erection of wood barriers (Figure 4-50).
- **Structural Performance:** The masonry wall on the south side of the building separated from the structure and portions of it fell into the neighboring parking lot, striking an unoccupied, parked car (Figure 4-49). Stones fell off the back of the building into the alleyway that separates the building from several retrofitted URMs (Figure 4-51). Large parapet stones fell into the attic space, which was used for storage. A large X-crack was observed in the north masonry wall.

Figure 4-50 Wooden barrier constructed on the roof a building neighboring a masonry wall at risk of collapse.

Figure 4-51 Failure of stone masonry east wall. Red arrow indicates wood barrier protecting adjacent property from falling stones.

- **Nonstructural Performance:** Cracking of the plaster wall and ceiling finishes was observed. HVAC equipment was damaged by falling stone masonry (Figure 4-52).

Figure 4-52 HVAC equipment damaged by falling stones.

- **Time until Full Occupancy:** The building remained closed at the time this report was being developed.

4.4.2 One-Story Restaurant Building, 1st Street (Buildings B2/B2A)

- **Structural system, height, year built:** The building is a single-story structure that includes both CMU and unreinforced masonry walls, with a flexible roof diaphragm (Figure 4-53). The date of construction is unknown.

Figure 4-53 Front elevation of Buildings B2/B2A.

- **Occupancy Type:** Restaurant.

- **Posting Placard:** At the time of the field investigation, the building was posted UNSAFE due to the collapse of a URM wall on the east side of the buildings, and collapse of the ceiling in the dining area.
- **Structural Performance:** The building shares a common URM wall with the structure to the east (Building B3, located to the left of the roof drain in Figure 4-53, with the green awning). This wall collapsed into the dining area, striking about six tables (Figure 4-54). This is another example indicated that the time of the earthquake prevented increased numbers of injuries and fatalities. It was reported that a portion of the east URM wall above at the roof level that had not collapsed, being supported by a beam, was deemed a falling hazard. The parapet at the front of the building appeared to have displaced. No damage to the CMU walls was noted.

Figure 4-54 Collapsed URM wall and ceiling in dining area.

- **Nonstructural Performance:** The plaster ceiling in the dining area collapsed. Moderate cracking in interior wall finishes and exterior stucco was observed. No damage to equipment was noted.
- **Time until Full Occupancy:** The building was closed for repairs at the time this report was being developed.

4.5 Retrofitted URM Construction with Significant Damage

City of Napa passed an ordinance in 2006 that required property owners to retrofit unreinforced masonry buildings by the summer of 2009. The stated objective of the ordinance was "to reduce the risk of death or injury" (Napa Municipal Code, Chapter 15.110). Some retrofitted URM buildings suffered damage as a result of incomplete seismic upgrading measures. Portions of buildings where improvements were limited demonstrated vulnerabilities over portions of the same building that were more comprehensively retrofitted.

4.5.1 Three-Story Office Building, 2nd Street, Napa, CA (Building E5)

- **Structural System, Height, Year Built:** This three-story URM building was built in 1910 as a hotel and seismically retrofitted circa 1984 to 1986 to house offices (Figure 4-55). This work included an addition to the structure. The retrofit consisted of the addition of steel moment and braced frames, and wall-to-floor and roof diaphragm anchorage.
- **Occupancy Type:** Offices.
- **Posting Placard:** At the time of the field investigation, the building was posted both UNSAFE and RESTRICTED USE. The more heavily damaged portion of the building was posted UNSAFE.
- **Structural Performance:** The building sustained a collapse of the top floor URM walls at the northwest and southwest corners (Figures 4-55b, 4-56, and 4-57). The horizontal tube steel strongback was not effective at preventing the wall from collapsing out-of-plane. It was observed that the anchor bolts were straight and did not appear embedded through all brick wall wythes (Figure 4-58). The soffits of the roof cupola were several feet higher than the horizontal tube steel braces. The wall also reduced in thickness by at least one wythe above the horizontal braces. This resulted in a thinner, unbraced wall section, several feet high, spanning between the braces and soffits of the roof cupola. The collapse of the walls resulted in a loss of support to the wood-framed roof cupolas at the building corners, which were at incipient collapse following the earthquake. The other portions of the building sustained minor structural damage.

(a)

(b)

Figure 4-55 Photographs of building: (a) circa 1910 and (b) 2014.

- **Nonstructural Performance:** With the exception of areas in the vicinity of heavy structural damage, little nonstructural damage inside the building was reported. Several rooftop mechanical units slid off of their bases, most appeared to have little or no seismic anchorage.
- **Time until Full Occupancy:** The less heavily damaged portions of the building reopened within two weeks of the earthquake. The heavily damaged portion remained closed at the time this report was being finalized.

Figure 4-56 Collapse of URM walls supporting a corner cupola.

Figure 4-57 Collapse of the URM wall at a corner cupola.

Figure 4-58 Close-up of steel tube and masonry anchor at corner cupola.

4.5.2 Two-Story Building, 1st Street (Building I2)

- **Structural System, Height, Year Built:** This URM building is two-stories tall, built in 1901, with a partial seismic retrofit at the roof in 1975 and a comprehensive seismic retrofit and expansion in 2004 (Figure 4-59). The building is rectangular in plan, 79 feet in the north-south longitudinal direction and 34 feet in the east-west transverse direction. There is about six feet of separation from the buildings on the west and east sides. The building is set back from the street on the north, and there is a parking lot at the rear, south façade. The first floor is above grade by approximately two feet, and there is a crawl space and mechanical subbasement. Exterior walls are two-wythe stone masonry. The roof is supported by wood carpenter trusses; the floors by wood framing. The top of the front façade has a tall parapet or tower feature that is rectangular in plan. There is an exterior steel framed fire escape on the east side.

Figure 4-59 Front façade showing damage and UNSAFE placard before scaffolding was installed.

The 1975 retrofit is located above the roof and consists of steel tubes doweled to the inside face of the parapet that are coupled with horizontal bracing made of a pair of transverse tubes at third points along the longitudinal axis and diagonal tension bracing rod connecting the tubes.

The City had not yet passed a URM ordinance at the time of the 2004 retrofit. The federally-funded 2004 retrofit was designed using the requirements of the 1997 *Uniform Code for Building Conservation* (ICBO, 1997b). As a qualifying historic building, the provisions of the 2001 *California Historical Building Code* (ICC, 2001) were also used.

The 2004 retrofit provided enhancements to the 1975 retrofit including improved parapet ties using vertical drilled dowels down through the cap stone of the parapet, additional deeper horizontal drilled dowels connecting the steel top to the parapet, replacement of the tension rods with larger diameter rods, enhancement of the steel tube splices, and connection of the transverse tubes through the roof into the existing wood roof trusses. Although some of the 1975 rods were specified to be removed, they are still in place. A concrete diaphragm is shown several feet below the top of the front tower feature to connect the four parapet or walls of the tower together.

The 2004 retrofit also included tension and shear ties from the walls to the roof, second floor, and first floor; a plywood overlay over the roof sheathing; a steel collector/chord at the second floor along the perimeter walls; reinforced concrete wall backing on the inside face of the front (transverse) façade; a transverse concrete wall with a grade beam, and diaphragm-to-wall collectors in the middle of the longitudinal direction; ties between members at the crawl space post and beam construction; anchor pins connecting the interior and exterior stone masonry wythes; epoxy injection of existing cracks; repointing; and some stone repair. It is possible that some of the repair work may have been associated with damage from the 2000 Yountville earthquake.

Historic drawings indicate that there was a small wooden lean-to structure attached to the rear wall that was about eight feet in depth and rose up to the parapet level. It had been removed at some point prior to 1999. The 2004 retrofit included a small two-story wood-frame addition at the same rear façade that provided an elevator and second stairway. No damage to the wood-frame structure was observed.

- **Occupancy Type:** The building was originally constructed as the town library; it now houses offices and research library/archival space for the Napa County Historical Society. Offices are on the first story and offices and library space on the second story.
- **Posting Placard:** The building was initially given an UNSAFE placard. Following installation of scaffolding on the west, north, and south sides and more permanent fencing, removal of loose wall and ceiling plaster, and closure of the front portion of the building, staff working in the building were permitted to reoccupy the southern portion of the building. An INSPECTED placard was posted at the rear entrance, granting access to the southern portion of the building.
- **Structural Performance:** Damage included cracking of the stone perimeter façade pier near the roof level, typically through mortar joints, corner damage at all four corners with out-of-plumb movement of approximately 2 inches at the northwest and northeast corners and 4 to 6 inches at the southwest corner. At the northwest corner, the masonry just below the cornice had shifted outward up to 3/4 inches. There were small fallen stones on the east façade, light cracking above the roof at the

parapets at the southwest and southeast corners, and very significant damage to the front façade stone tower feature at the roof level that included fallen stone and shifting of the stone on mortar joints. See Figures 4-60 and 4-61 for damage at the front façade and tower feature.

Figure 4-60 Cracking on east side of the tower feature.

Figure 4-61 Top of concrete shear wall at roof level behind the tower feature.

The seismic retrofit work generally appeared effective in mitigating the original structural deficiencies, tying the structure together, and limiting damage to the masonry elements. Minimal movement was observed at the roof-to-parapet ties, and there were no obvious signs of diaphragm-to-wall damage or residual offsets. See Figure 4-62 for a parapet tie at a corner. The two notable exceptions were outward movement at all four corners of the building and damage at the front façade tower feature.

Figure 4-62 Steel tube bracing at parapet.

The steel tubes at the roof extended around the corners and held the top of the wall and parapets together as did the second floor diaphragm-to-wall ties. The outward wall movement occurred below the cornice level that was approximately between the roof and second story ceiling levels.

The concrete backing at the front façade stopped at the roof level and did not continue up the remaining height of the tower feature. The most significant damage began at about this level. The tower feature also did not have any parapet bracing or capstone ties like the other parapets. Reportedly, historic preservation concerns limited the extent of retrofit work that was implemented at the tower feature.

- **Nonstructural Performance:** Lath and plaster at the walls and ceiling cracked and pieces fell (see Figure 4-63), there was movement at the second story ceiling, and many books fell from bookshelves in the library (the shelves were bolted to the floor). During a site visit on September 9, 2014, the elevator was not functioning. The cab had descended to the first floor level, and the light was on in the cab, but the doors did not fully open. Power, water, and computer service remained in operation. The water heater was strapped and was not damaged. No windows were broken despite the extent of shaking in walls and damage to furring. The steel fire escape on the east side was physically anchored to both buildings without a seismic joint on one side. Some out-of-plane pullout of the connection at the adjacent building was observed.

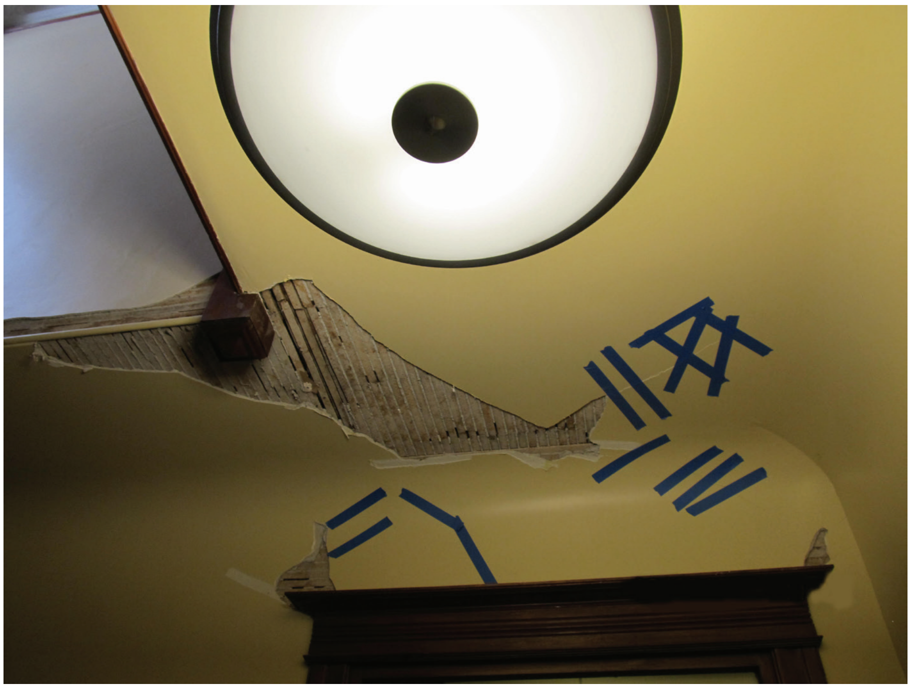


Figure 4-63 Lath and plaster ceiling damage and temporary taping at cracks and spalls.

- **Time until Full Occupancy:** The building was closed until the interim scaffolding, fencing, and repair work were completed and reposted with the INSPECTED placard at the rear entrance on September 23, 2014.
- **Other Notes:** The 2004 retrofit was partly funded by FEMA. As a result, California Office of Emergency Management is conducting a State Mitigation Assessment Review Team (SMART) assessment of the retrofit and damage, including review of retrofit costs and potential repair costs.

4.5.3 Two-Story URM Building, 1st Street (Building J4)

- **Structural System, Height, Year Built:** This URM building is two-stories tall, built in 1905, and seismically retrofitted in 2004 (Figure 4-64). Retrofit consists of steel moment frames on the open storefront façades on the south and east elevations and wall-to-diaphragm anchorage. The north and west sides of the building have URM walls only, and no out-of-plane strengthening was provided between floors.

Figure 4-64 Exterior view of building.

- **Occupancy Type:** Restaurant on the first floor, offices on the second floor.
- **Posting Placard:** At the time of the field investigation, the building was posted UNSAFE.
- **Structural Performance:** There was in-plane damage at the south façade (Figure 4-65) due to "bookend effect" (the two adjacent unretrofitted URM structures to the west had no lateral force-resisting systems). Out-of-plan punching shear failure of URM wall was observed at the second floor anchors of the south wall (Figure 4-66). The building suffered moderate to severe diagonal shear of URM at north and west walls. The floor-to-wall diaphragm connections for the north and west walls had only straight anchors to resist tension (not inclined at 22.5 degrees), and many were pulled out.
- **Nonstructural Performance:** Unknown. Building interior was not available for observation.
- **Time until Full Occupancy:** The building remained closed six months after the earthquake.
- **Other Notes:** The building suffered damage in the 2000 Yountville earthquake. Observed damage included cracking of spandrels along south façade and over the corner diagonal entry at southeast corner of the building (Figure 4-67).

Figure 4-65 Cracking in URM on south façade.

Figure 4-66 URM damage, west elevation (photo from Chris Jonas, ZFA Structural Engineers).

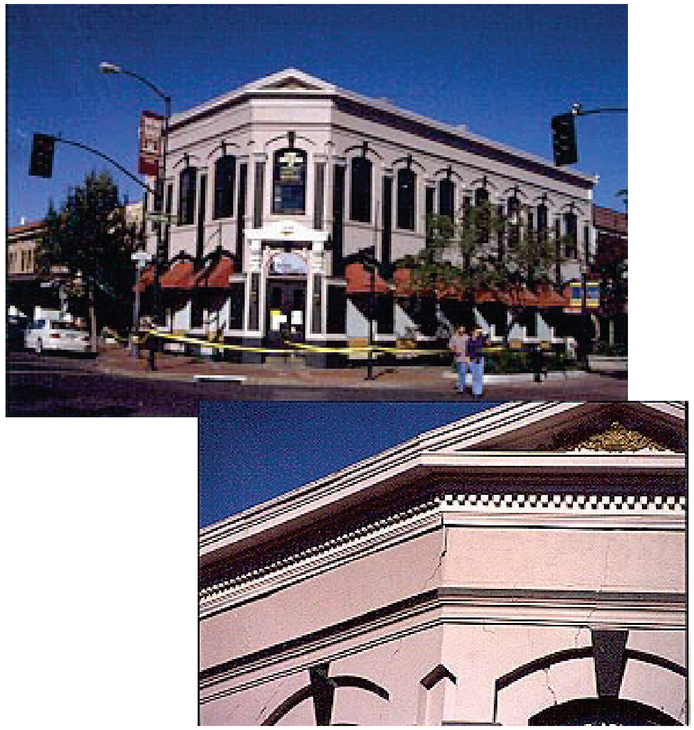


Figure 4-67 Photo of Building J4 and detail of damage after the 2000 Yountville earthquake (PEER, 2000).

4.6 Retrofitted URM Construction with Limited Damage

Most of the retrofitted buildings performed well. In some cases, limited sections of masonry became loose, and some were dislodged.

4.6.1 Three-Story Theater Building, Main Street (Building L2)

- **Structural System, Height, Year Built:** This three-story URM building was built in 1879 and seismically retrofitted over a period of years (Figure 4-68). This work included an addition to the structure. The retrofit consisted of the addition of steel moment frames along the front façade, parapet bracing (Figure 4-69), wall-to-floor and roof diaphragm anchorage, and steel strongbacks for the URM walls. The roof over the stage has been rebuilt with steel framing, and a CMU addition has been added to the rear of the structure.
- **Occupancy Type:** Theater.
- **Posting Placard:** The building was posted RESTRICTED USE. The City of Napa website indicates the fire alarm was on test, the elevator was not functioning, and the sprinkler system had been turned off.

Figure 4-68 Exterior view of building.

Figure 4-69 URM parapet bracing.

- **Structural Performance:** The building did not sustain significant structural damage. There is evidence of pounding with the structure to the south (Building L1). Some of the wall anchors on the south side of the building appear to have loosened, and these wall anchor bolts damaged the wall of the adjacent building (Building L1) (Figures 4-5 and 4-70).

Figure 4-70 Loosened wall anchors on south wall, looking down.

- **Nonstructural Performance:** A single fire sprinkler line failed at a threaded fitting above a walk-in cooler and flooded the first floor of the structure, damaging the hardwood floor. Minor cracking in the plaster wall and ceiling finishes was observed, and about 10% of the sprinkler head covers dislodged. The audio and lighting components suspended from the ceiling in the theater suffered no damage. In the theater, the crown molding made up of large fiberglass units was displaced in some areas. The door mechanism of the hydraulic passenger elevator malfunctioned; the freight elevator was undamaged. One compressor supported on a steel skid assembly suffered connection failures and shifted (Figure 4-71). Extensive loss of glassware and dishes in the first floor restaurant was reported, along with the loss of some stored wine.
- **Time until Full Occupancy:** The building was closed for a week to repair water damage.

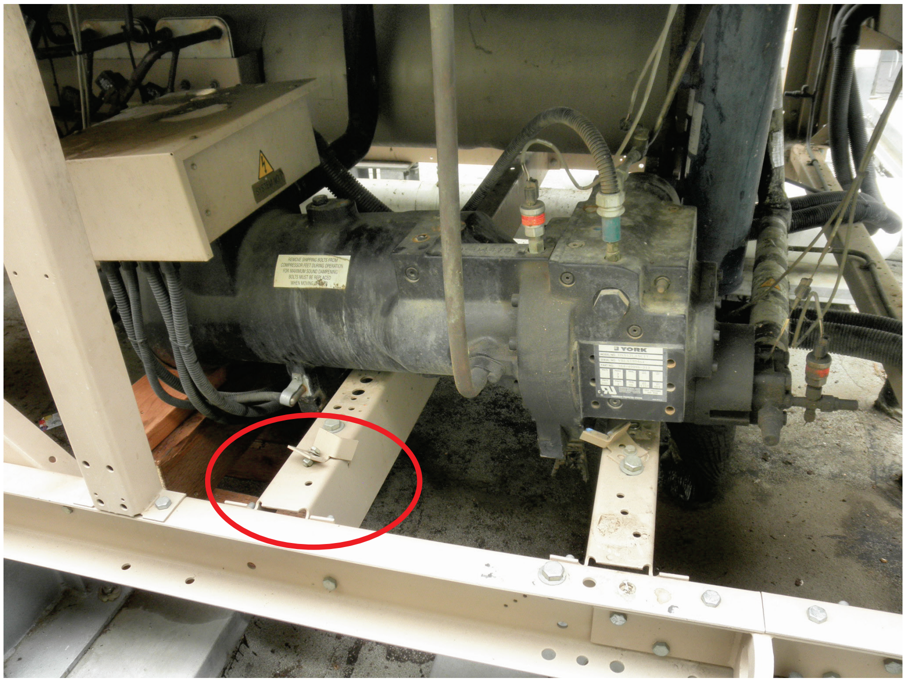


Figure 4-71 Connection failure at compressor.

4.6.2 One-Story Restaurant Building, Main Street (Building L4)

- **Structural System, Height, Year Built:** This single-story unreinforced stone masonry building, with mezzanines at the rear, was built in 1886. The building is split into three separate tenancies with solid demising stone masonry walls perpendicular to Main Street. The front façade (Main Street) is open. The rear façade is comprised of stone masonry piers and spandrels. The building was seismically retrofitted, including steel moment frames at the front façade, interior shotcrete overlays at the rear walls, out-of-plane wall anchorage to roof diaphragms and parapet bracing at the front façade.

- **Occupancy Type:** Restaurant, retail, and office.

- **Posting Placard:** At the time of the field investigation, the building was posted RESTRICTED USE prohibiting access from Main Street due to parapet damage, but permitted access via rear and side entrances. However, there was no warning sign on the interior of the door and, the exit was being actively used by staff leading into the cordoned area.

- **Structural Performance:** With the exception of the parapets (see Nonstructural Performance, below) the observed damage was generally minor (Figure 4-72). Minor to moderate cracking was observed at parts of the rear wall (Figure 4-73), specifically: at corners (stair stepped through bed joints) and inter-tenancy walls (both stair stepped through bed joints and through stone masonry units). It was unclear from visual observation as to how or whether these crack patterns related to the shotcrete overlays on the interior face of the wall.

Figure 4-72 Exterior wall on day of earthquake.

Figure 4-73 Diagonal cracks in stone wall.

- **Nonstructural Performance:** Stones were dislodged from the stone masonry parapets along the front and rear walls, with the majority of damage appearing to be on interior face of the parapet (Figures 4-74, 4-75, and 4-76). Some capstones were dislodged from the low-height parapet along the rear. Stone masonry chimneys at rear appeared essentially undamaged. It is unknown how or if these chimneys were retrofitted. Some glazing in the mezzanine broke, and there was minor loss of contents.

Figure 4-74 Exterior face of parapet wall.

Figure 4-75 Close up of dislodged stone on inside face of parapet.

Figure 4-76 URM parapet bracing, loss of stonework.

- **Time until Full Occupancy:** One tenant was able to reopen within one day of the earthquake. The other tenant was able to reopen within three days.

4.6.3 Two-Story Building, 3rd Street (Building M1)

- **Structural System, Height, Year Built:** This URM building is two-stories tall, built in 1887 and was seismically retrofitted in 2006-2007 (Figure 4-77). The building is rectangular in plan, 120 feet in the north-south longitudinal direction and 40 feet in the east-west transverse direction. The building stands alone with no adjacent structures. The building had been vacant for a number of years prior to the earthquake, and was surrounded by a fence. Exterior walls are two-wythe stone masonry. The ground story at the front façade has large window openings. There are limited openings on the east and north façades and no openings on the west façade. Buildings adjacent to the west façade were removed in 1999. The wood sheathed roof is supported by wood bowstring trusses at 20 feet on center with rafters and ceiling joists spanning north-south between the trusses. The second floor has steel W24 transverse girders supported by the masonry walls with sawn lumber joists spanning north-south to the girders that support wood floor sheathing. The first floor is slab-on-grade.

Figure 4-77 View from the southwest showing front façade and fencing.

The City of Napa had not yet passed a URM ordinance at the time of the retrofit. The retrofit was done using the requirements of the 1997 *Uniform Code for Building Conservation*. As a qualifying historic building, the provisions of the 2001 *California Historical Building Code* were also used.

The retrofit included parapet bracing of the north (rear) and south (front) gable façade, a concrete bond beam atop the east and west walls, tension and shear ties from the walls to the roof and to the second floor, a plywood overlay over the roof sheathing and over the second floor sheathing, a steel moment frame at the first story of the front façade where masonry piers are limited, concrete shear walls at third points along the longitudinal length of the building, supplemental or secondary vertical supports under the roof trusses that continue to the foundation, anchor pins connecting the interior and exterior stone masonry wythes, and repointing of deteriorated mortar. Some existing tension tie bearing plates were present that may be from original construction. Regrouting of damaged or missing grout under these bearing plates was specified as part of the retrofit. One of the new interior transverse shear walls is H-shaped in elevation with a new grade beam below the walls and a collector strengthening an existing roof truss at the top of the walls. The other concrete wall assembly is associated with a new stair and elevator; it is supported by a concrete mat. The fire sprinkler and electrical systems were upgraded during the retrofit.

- **Occupancy Type:** Vacant, with a cold shell. Partitions were removed with the retrofit as were finishes on the inside face of much of the stone walls. Ceilings were also not present.
- **Posting Placard:** At the time of the field investigation, the building was posted with a RESTRICTED USE placard. No specific information was given regarding which areas were restricted.
- **Structural Performance:** Damage includes stair-stepped cracking through mortar joints at piers and spandrels and loose stones over windows on the south façade, cracking and a fallen stone at the rear façade, and some light cracking at the east façade (Figures 4-78, 4-79, 4-80). The fallen stone at the rear is near the midspan of the wall where out-of-plane accelerations would be largest. No damage was observed on the solid west façade.

Figure 4-78 Loose stone cladding over window at front façade.

Figure 4-79 Diagonal cracking at inside face of second story pier at front façade.

Figure 4-80 Rear façade elevation. Red arrow points to the location where the stone fell at the center of the gable.

- **Nonstructural Performance:** Minimal nonstructural elements exist in the building. The fire sprinkler and electrical power remained operational.

- **Time until Full Occupancy:** Not applicable, as the building was vacant at the time of the earthquake.

- **Other Notes:** The 2006-2007 retrofit was partly funded by FEMA.

4.6.4 Two-Story Commercial Building, Main Street (Building A1)

- **Structural System, Height, Year Built:** This two-story unreinforced masonry building was constructed in 1890 and was seismically retrofitted circa 1985 (Figure 4-81). It appears that the adjacent building to the east shares a common URM wall with this structure. The retrofit consists of a steel moment frame at the first floor and steel braced frames at the second floor on the west elevation.

Figure 4-81 Exterior view of building.

- **Occupancy Type:** Restaurant and retail on the ground floor, offices on the second floor.

- **Posting Placard:** At the time of the field investigation, the building was posted RESTRICTED USE. The City of Napa website indicates that the wall adjacent to parking lot was leaning out.

- **Structural Performance:** Minor damage (cracking through the stucco) was observed in the URM walls on the west and south elevations (Figure 4-82). However, localized significant damage was observed at the south-east corner of the building, where the rear URM wall appears to be a common/party-wall with the adjacent building to the east. A relatively modern steel-framed exterior stair, which provides access to the second floor of adjacent building, is supported at the mid-height landing from the URM wall. The stair was rigidly attached to the URM wall, the adjacent building at the second floor, and at grade. The stair stringers appear to have acted as braces, inducing out-of-plane demands on the URM wall, which resulted in localized out-of-plane wall failure (Figure 4-83). The upper stair stringer also buckled.

Figure 4-82 Minor cracking in URM wall, north elevation.

Figure 4-83 Southeast corner, damage to URM wall due to interaction with steel stair.

- **Nonstructural Performance:** About 50% of the glazing on the ground floor and 10% of the glazing on the second floor were damaged (Figure 4-84). Minor cracking of the plaster wall and ceiling finishes was observed. No damage to the mechanical and electrical systems was reported.

Figure 4-84 Damage to glazing, west elevation.

- **Time until Full Occupancy:** Some of the tenants on the ground floor were able to remain open; others were closed until the glazing was repaired. The second floor offices appear to have been occupied soon after the earthquake.

4.6.5 Two-Story Restaurant Building, 1st Street (Building A2)

- **Structural System, Height, Year Built:** This two-story unreinforced masonry building was built in 1888 and has been seismically retrofitted (Figure 4-85). It appears that the adjacent building to the west shares a common URM wall with this structure. Retrofit comprised of adding lateral force-resisting elements in the transverse direction: a steel chevron braced frame at the 1st Street façade and a punched CMU wall at rear. The CMU wall was clad with a brick veneer. Parapet bracing and out-of-plane wall anchors were also observed; through-bolts with anchor plates were visible at roof level.

Figure 4-85 Exterior view of building.

- **Occupancy Type:** Restaurant on the ground floor, offices on the second floor.
- **Posting Placard:** This building was posted UNSAFE following the earthquake. At the time of the field investigation, the building was posted RESTRICTED USE.
- **Structural Performance:** Minor damage (hairline cracking) was observed at the interior CMU retrofit wall.
- **Nonstructural Performance:** The stonework parapet above the front entrance of the building dislodged (Figure 4-86). Anchorage failure at the parapet braces was observed, which may be due to poor adhesive installation of anchor bolts (Figure 4-87). One rooftop air conditioning unit failed at its base connection and slid slightly on the wood sleeper (Figure 4-88). The exterior steel framed stair at rear of building interacted with common/party-wall of the building to the west (Building A1) and caused out-of-plane wall failure (Figure 4-83).
- **Time until Full Occupancy:** The building was closed for at least five days, although the business was essentially ready to re-open after three days except for the parapet repair and removal of posting.
- **Other Notes:** $15,000 of contents damage to wine bottles and plates was reported.

Figure 4-86 Damage to stonework above main entrance.

Figure 4-87 Failed wall anchor, north elevation.

Figure 4-88 Failure of air handler unit anchor, screws pulled out of sleeper.

4.6.6 One-Story Restaurant Building, Main Street (Building A4)

- **Structural System, Height, Year Built:** Built in 1933, this single-story building is constructed with 6-inch thick unreinforced hollow terracotta perimeter bearing walls, and a wood-framed roof (Figure 4-89). There is a small wood-framed addition at the rear of the building. The original structure was seismically retrofitted circa 2007 ($600,000 construction cost, with $200,000 lost revenue during construction according to owner). The retrofit was extensive. Unreinforced masonry foundation stem walls were strengthened with reinforced concrete overlays and grade beams. Plywood sheathed wood-frame walls (typically 2x6 at 24 inches on center) were installed against the perimeter bearing terracotta walls to provide out-of-plane restraint and in-plane shear strength. The terracotta walls were anchored to the new wood-frame walls (6x6 studs at 4 feet on center) and strengthened roof diaphragm via threaded rod epoxy anchors. The relatively open storefront along Main Street was strengthened with an inverted moment frame, utilizing HSS14x10 posts cantilevered from a continuous grade beam.
- **Occupancy Type:** Restaurant.
- **Posting Placard:** At the time of the field investigation, the building was posted INSPECTED.
- **Structural Performance:** No damage was reported.

Figure 4-89 Building elevation from Main Street.

- **Nonstructural Performance:** The building contains beer tanks, kettles, and other equipment related to brewing operations. The only reported nonstructural damage was reported as: bottles and crockery falling, and an unanchored fermentation tank that slid 8 inches (Figure 4-90). The tank had flexible pipe connections which remained intact. Minor cracking of interior wall and ceiling finishes.

- **Time until Full Occupancy:** Aside from clean-up of broken bottles and dishes, the restaurant was open and serving food on the day of the earthquake.

- **Other Notes:** Station N016 is located in the crawlspace of the building, mounted to the top of the footing (Figure 4-91). Drawings of the seismic retrofit were available for review. The rear of the building is located immediately adjacent to the riverfront.

Figure 4-90 Unanchored tank shifted approximately 8 inches.

Figure 4-91 Photo of strong-motion recording instrument (Station N016) located in the crawlspace of Building A4.

4.6.7 One-Story Restaurant Building, Main Street (Building E7)

- **Structural System, Height, Year Built:** The building is a one-story unreinforced masonry building with a roof diaphragm constructed in 1890 (estimated) (Figure 4-92). The building was retrofitted in 2005.

Figure 4-92 East elevation.

- **Occupancy Type:** Restaurant.
- **Posting Placard:** The building was posted UNSAFE shortly after the earthquake due to damage to an adjacent building, but was INSPECTED and in use within one week. One of the two tenants in the building remained closed for unknown reasons which may have not been related to the earthquake.
- **Structural Performance:** No structural damage was observed.
- **Nonstructural Performance:** The building suffered loss of several large storefront windows, and damage to contents, such as glasses and plates. No damage to the building mechanical and plumbing systems was reported.
- **Time until Full Occupancy:** The building was closed for approximately a week.

4.7 Summary

Buildings constructed to recent codes (1997 *Uniform Building Code* or later editions of the *International Building Code*) generally performed well structurally. The vast majority of older, non-URM structures also performed well structurally, although known vulnerabilities, such as poor wall-to-roof connections, did result in significant damage and loss of use.

Nonstructural components and systems in most buildings constructed to recent codes were damaged to some extent, and in some cases buildings sustained significant damage resulting in loss of use for an extended period.

Serious damage to exterior curtain walls and ceiling systems occurred in some structures. Pressurized piping system failures, especially fire sprinkler systems, caused extensive and significant water damage even though the actual number of piping failures was comparatively small. The losses would have been much less if the water supply to the sprinklers had been shut off after the piping failures. All fire sprinkler systems are equipped with control valves; however, they are normally locked in the open position to prevent tampering or accidental closure. Due to the hour of the event and the time it took someone with the control valve key to arrive on scene, broken sprinkler systems ran for hours in several cases, greatly aggravating the water damage. In some situations, the fire department shut off the control valve, but this is not their top priority as they were often too busy responding to life threatening incidents. Under these circumstances, failure of even a single sprinkler head caused extensive and widespread damage to wall, floor, and ceiling finishes. In most cases, nonstructural components in buildings constructed before 1998 were installed prior to the widespread enforcement of seismic bracing requirements. Nonstructural damage to suspended ceilings, equipment, and piping systems is to be expected in buildings of this vintage, and was fairly common. These issues are discussed in greater detail in Chapter 10.

Among the retrofitted URM buildings, ten buildings suffered no structural damage, or the damage was deemed insignificant; six buildings suffered minor damage, one building moderate damage, and three were heavily damaged. None of the retrofitted buildings collapsed, and some exterior masonry loosened or fell from three of the damaged buildings. Based on the performance of the URM buildings in within 1,000 feet of Station N016, the URM hazard mitigation efforts in Napa were successful in reducing damage and protecting life safety. A number of different approaches had been used to retrofit URMs, and partial retrofits of URM buildings were less successful in limiting damage compared to those that received more comprehensive upgrades. Stone masonry walls and parapets seem more likely to sustain damage compared to those of brick masonry.

Damage to lath and plaster walls and ceilings was common in URM buildings. Damage to glazing was more severe in unretrofitted buildings and in those buildings that utilized flexible moment frames at the storefronts.

4.8 Recommendations

The performance of buildings in the South Napa earthquake suggests several areas for possible improvement and further study, including:

1. Unretrofitted URM structures continue to pose significant risks to the public. Every effort should be made to eliminate the risks posed by these buildings.
2. For ordinary building occupancies, building codes focus on preserving life safety by reducing the likelihood of building collapse in a very large earthquake. Many structural systems emphasize ductility, which can translate to structural damage in moderate events. The effects that this design approach has on the resiliency of communities subject to more frequent, smaller earthquakes should be explored.
3. In modern buildings, the single focus on collapse prevention produces structures that are vulnerable to significant losses due to nonstructural damage, even in moderate event like the South Napa earthquake. For example, the code limits for building drift are such that serious damage to drift-controlled components, such as cladding, glazing, and partitions, is inevitable in moderate earthquakes. Further study is needed to develop drift-controlled components that can tolerate the displacements permitted by code.
4. Even where the code specifies drift criteria for nonstructural systems, provisions for drift were often lacking in damaged buildings. Better coordination between the designer, contractor, and building officials is needed to ensure that proper seismic details are provided.
5. The reliability of nonstructural components in ordinary occupancies needs to be improved. Recommendations for this are found in Chapter 10.
6. Chapter 34 of the current *California Building Standards Code* contains requirements for earthquake evaluation and retrofit of select vulnerable buildings undergoing significant modifications. Enforcement of code triggers related to the scope of renovations should be encouraged for all vulnerable buildings to reduce the structural and nonstructural risks they pose.

The following are recommendations specific for improving URM building evaluation and retrofit methodologies:

1. There was a wide range of retrofit approaches observed for URM buildings. Detailed assessments of selected buildings should be conducted to determine if common URM evaluation and retrofit methodologies accurately predict observed pier and spandrel cracking mechanisms. Findings can be incorporated into future updates of evaluation and retrofit guidelines.

2. With the exception of FEMA 306 and 307, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings: Basic Procedures Manual and Technical Resources*, (FEMA, 1998a and b, respectively) and recent New Zealand work, common URM evaluation and retrofit guidelines focus on pier mechanisms, but provide little guidance for spandrel mechanisms. Spandrel damage was observed in a number of buildings, and in some cases led to the formation of two-story tall pier mechanisms. Study of the cases where this was observed will be of value to better understand and predict this behavior and whether it should be incorporated into future updates of evaluation and retrofit guidelines.

3. Poorly performing buildings often had retrofit schemes that were either incompatible with the properties of the buildings or were not sufficiently comprehensive. For example, flexible moment frames are less effective in protecting URM buildings than more rigid approaches. Tall URM walls may require intermediate supports for out-of-plane forces.

4. Vertical cracking at corners, out-of-plumb leaning, and horizontal out-of-plumb movement at bed joints were observed in a number of buildings. Strong steel tubes well developed around the corner (corner ties) appeared to mitigate the movement and damage to some extent. Investigation of this issue is recommended to determine whether prescriptive requirements should be added beyond the typical reduced diaphragm-to-wall spacing requirements.

5. There were a number of failures of adhesive-type anchors observed in retrofitted URM buildings. Failures could be due to a variety of factors, including detailing (straight versus bent anchors), problems with adhesion between the anchor and the substrate, deterioration of the adhesive material, or the effects of cracking in the masonry on anchor capacity. The cause of these failures should be determined and steps should be taken to improve the reliability of these anchors.

6. Further study is recommended to assess possible variations in interpretations of *State Historical Code* and the *International Existing Building Code* requirements. In some cases, the goal of minimizing the impact to the historic fabric at historic buildings may have led to a reduction in the extent and scope of seismic retrofitting and may have contributed to damage of elements with reduced mitigation and an increase in localized life-safety risks. A study is recommended to identify examples of these issues and to develop ways to better address both historic preservation and sufficient seismic safety.

Chapter 5

Performance of Healthcare Facilities

5.1 Introduction

Following the catastrophic collapse of hospital buildings in the 1971 San Fernando earthquake, the 1972 *Hospital Seismic Safety Act* identified hospitals in California as essential facilities and established seismic performance goals for California hospitals stating:

> "…hospitals, that house patients who have less than the capacity of normally healthy persons to protect themselves,… must be reasonably capable of providing services to the public after a disaster…"

Unlike most commercial buildings, hospital buildings are designed to remain operational after an earthquake. The California Office of Statewide Health Planning and Development (OSHPD) enforces the *Hospital Seismic Safety Act*. In addition to oversight of design and construction of hospitals, OSHPD performs postearthquake assessments of buildings under its jurisdiction. The assessments are performed by multidisciplinary teams of OSHPD staff, including structural engineers, inspectors, and fire/life safety officers, who examine the buildings and post them using procedures documented in ATC-20 family of documents. Damage assessment reports are compiled and are available to the public.

Following the South Napa earthquake, OSHPD activated its Emergency Operations Center (EOC) at 4:08 am, 48 minutes after the ground shaking. The EOC was staffed by 13 individuals. From the EOC, OSHPD dispatched ten investigation teams consisting of 22 total individuals to perform field operations. Based on proximity to strong shaking, the first teams were dispatched on August 24, 2014 to Queen of the Valley Hospital in Napa and Sutter Solano Medical Center in Vallejo. Building inspections continued through August 27, 2014. Investigation results were reported to back to the EOC on a regular basis.

The information provided in this chapter was largely provided by OSHPD. EOC inspection results are also publically available on OSHPD's website.

5.2 Performance of Hospitals

Six hospitals were located near the earthquake epicenter and were investigated by OSHPD following the earthquake. The locations of the hospitals are shown on the map in Figure 5-1, and more detail is provided in Table 5-1.

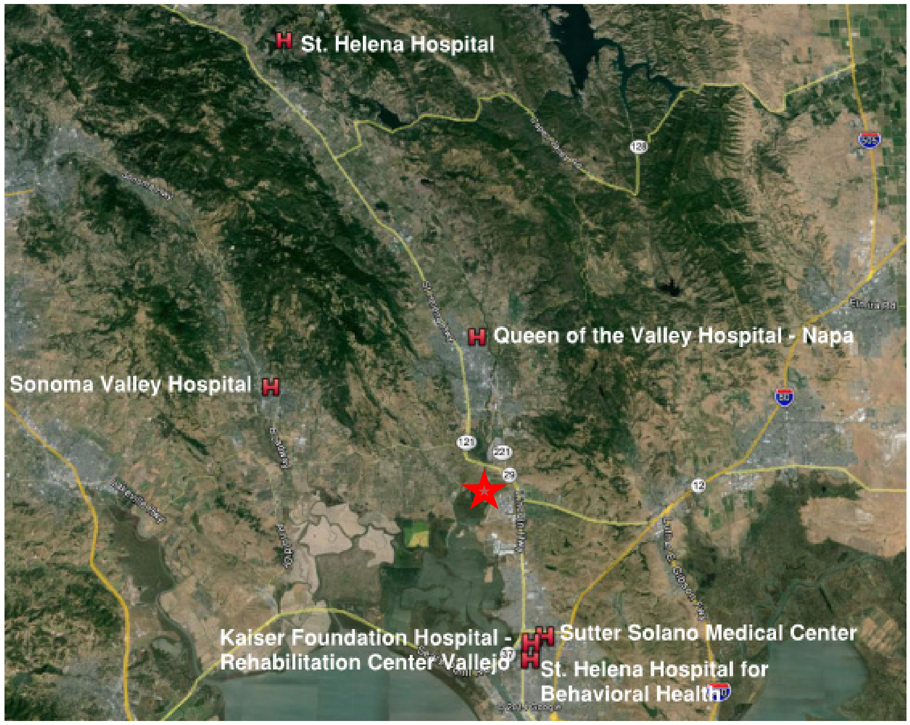


Figure 5-1 Map of hospitals (shown as red "H"s) investigated following the South Napa earthquake (epicenter is shown as the red star) (image source: Google Earth).

Table 5-1 List of Hospitals Investigated by OSHPD

OSHPD Facility ID	Hospital Name	Distance to Epicenter (miles)
13142	Kaiser Foundation Hospital - Rehabilitation Center Vallejo	7.1
12525	Sutter Solano Medical Center	7.2
10362	Queen of the Valley Hospital - Napa	7.7
11013	St. Helena Hospital Center For Behavioral Health	7.7
11064	Sonoma Valley Hospital	9.4
10366	St. Helena Hospital	>15

In general, hospitals performed well and remained operational following the earthquake. There was no significant structural damage reported, and no building was closed as a result of structural damage. One room at Queen of the Valley Hospital in Napa was posted RESTRICTED USE after inspectors found a crack in a precast beam that required further evaluation. Other observed structural damage was limited to minor cracking of concrete wall, beam, and slab-on-grade elements.

Nonstructural damage included damage to suspended acoustic tile ceilings, minor gypsum wallboard cracking, damage to a storefront glazing system, damage to exterior wall cladding, a small number of broken water pipes, movement of unanchored equipment and furnishings, and damage to expansion joint covers. Loss of power or a drop in power were reported at several hospitals. All emergency generators came on-line and provided electrical service to the facility. The Queen of the Valley Hospital ran on emergency generators for approximately three hours, while St. Helena Hospital experienced only a 35-minute power interruption during which emergency generators operated. Two hospitals had elevators out of service following the earthquake.

The following sections provide a summary of the reported performance of each hospital. Table 5-2 provides summary description of structural system used in the sections that follow.

Table 5-2 Structural System Descriptions

Structural System	Description
C1	Concrete moment frame
C2	Concrete shear wall
S1	Steel moment frame
S2	Steel braced frame
S4	Steel frame with concrete shear walls
PC2	Precast concrete frame with concrete shear walls
RM1	Reinforced masonry bearing wall
RM2	Reinforced masonry with rigid floor and roof diaphragms
W1	Wood light frame
W2	Wood light frame, commercial and industrial buildings

5.2.1 Queen of the Valley Hospital – Napa (OSHPD Facility ID 10362)

- **Structural System, Height, and Year Built:** The Queen of the Valley Hospital in Napa consists of 20 individual buildings. The oldest building was constructed in 1957, and the newest is still under construction. Buildings range from one- to three-stories in height, and construction types include steel moment frames, concrete shear walls, precast concrete frames, and reinforced masonry. A plan of the hospital is shown in Figure 5-2. Descriptions of the buildings, including number of stories, structural system type, year built, and structural performance category (SPC) class and nonstructural performance category (NPC) class designations, are presented in Table 5-3.

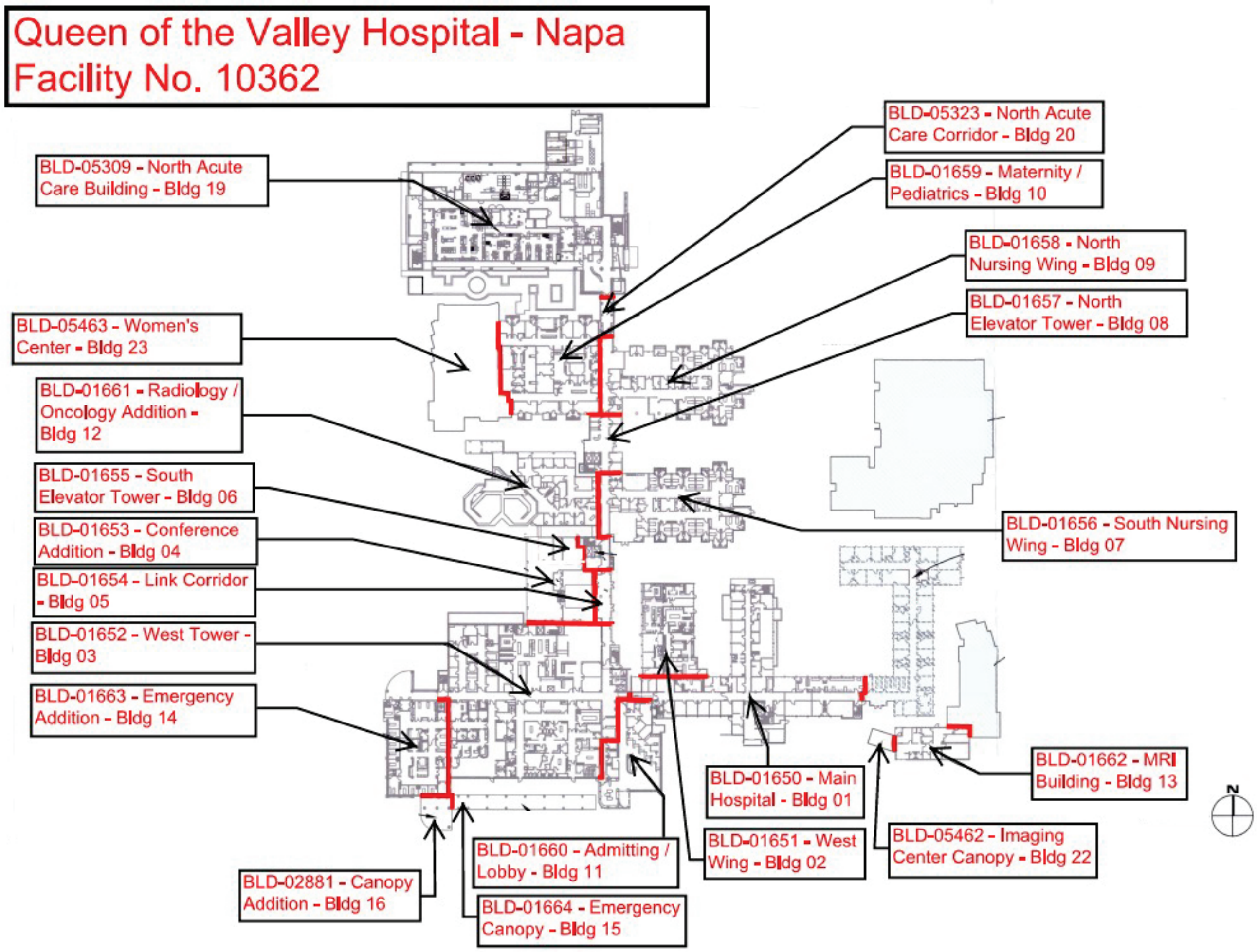


Figure 5-2 Queen of the Valley Hospital site plan indicating building numbers. Red lines indicate structural building separations.

- **Posting Placards:** Between August 24 and 26, 2014, OSHPD teams investigated all 20 of the buildings and identified minor structural damage and widespread minor nonstructural damage. Three of the buildings were posted RESTRICTED USE, while the rest were posted INSPECTED. The entire hospital stayed in operation during and after the earthquake, and all essential services were maintained.

Table 5-3 Queen of the Valley Hospital – Napa Building Descriptions and Posting

Building Number	Building Name	Stories	Structural System	Year Built	SPC Class	NPC Class	Posting
BLD-01650	Main Hospital	3	PC2	1957	1	2	RESTRICTED USE
BLD-01651	West Wing	2	RM1	1964	1	2	INSPECTED
BLD-01652	West Tower	2	S4	1973	2	2	INSPECTED
BLD-01653	Conference Addition	1	S1	1983	3	2	INSPECTED
BLD-01654	Link Corridor	2	S1	1983	3	1	INSPECTED
BLD-01655	South Elevator Tower	3	C2	1983	4	2	INSPECTED
BLD-01656	South Nursing Wing	3	S1	1983	3	2	RESTRICTED USE
BLD-01657	North Elevator Tower	3	C2	1983	4	2	INSPECTED
BLD-01658	North Nursing Wing	3	S1	1983	3	2	INSPECTED
BLD-01659	Maternity / Pediatrics	2	S1	1983	3	2	INSPECTED
BLD-01660	Admitting / Lobby	1	S1	1993	3	2	INSPECTED
BLD-01661	Radiology Oncology Addition	1	S2, C2	1995	4	2	INSPECTED
BLD-01662	MRI Building	1	W1	1992	5	2	INSPECTED
BLD-01663	Emergency Addition	1	S1	1998	5	4	INSPECTED
BLD-01664	Emergency Canopy	1	S1	1998	5	4	INSPECTED
BLD-02881	Canopy Addition	1	S1	1998	5	4	INSPECTED
BLD-05309	North Acute Care Building	3	S2	2014	5s	4	INSPECTED
BLD-05323	North Acute Care Corridor	3	Unknown	2014	5s	4	RESTRICTED USE
BLD-05462	Imaging Center Canopy	1	RM1	2007	5	4	INSPECTED
BLD-05463	Women's Center	1	S1	2007	5	4	INSPECTED

- **Structural Performance:** Some structural damage was observed. A crack in a precast beam (Figure 5-3) in the Main Hospital (BLD-01650), a three-story structure constructed in 1957, led investigators to post the the room below the beam RESTRICTED USE pending further analysis. Other minor concrete cracking was also identified in this building

(Figure 5-4). Minor cracking and spalling in a precast panel in the Conference Addition (BLD-01653) was noted (Figure 5-5); the building was posted INSPECTED.

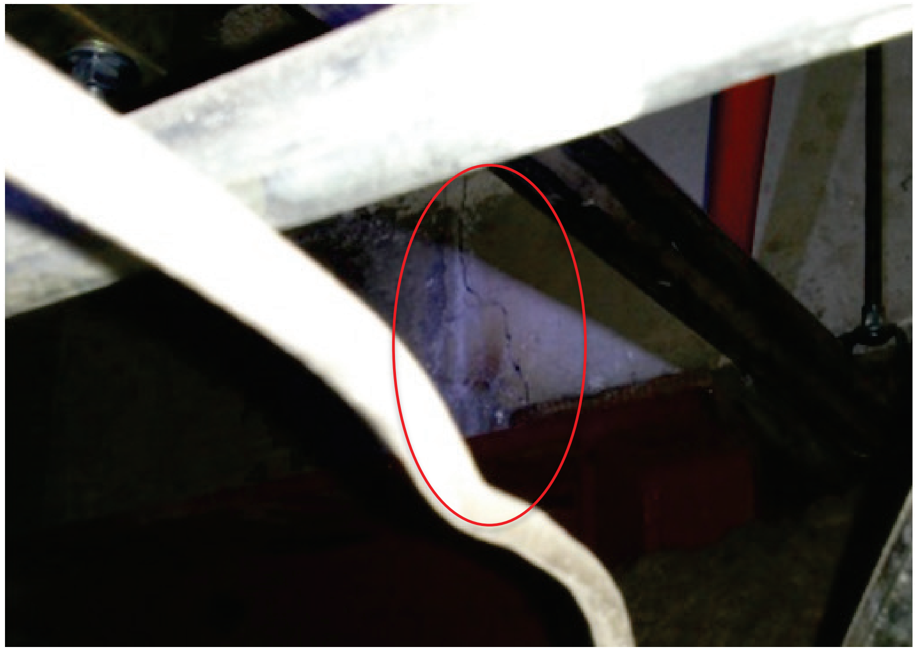


Figure 5-3 Damaged concrete beam at the third floor of BLD-01650.

Figure 5-4 Cracked concrete wall piers in BLD-01650.

Figure 5-5 Cracked precast concrete panel in BLD-01653.

- **Nonstructural Performance:** Nonstructural damage occurred throughout the hospital, in both the oldest and newest portions. Nonstructural damage included cosmetic cracking of gypsum board walls and ceilings (Figure 5-6), and acoustic tile ceiling damage (Figures 5-7, 5-8). It is reported that the seismic switches tripped in the elevators. In the old Main Hospital building, damage to the rollers and bent guide rails disabled the two of the elevators. Water damage due to leaking pipes was noted (Figure 5-9), but there were no reports of flooding. Damage to a storefront glazing system resulted in a RESTRICTED USE posting at one hospital entrance (Figures 5-10 through 5-11).

Figure 5-6 Cracked gypsum board wall in BLD-05309.

Figure 5-7 Third floor ceiling damage in BLD-05323 corridor link.

Figure 5-8 Third floor ceiling damage in BLD-01656.

Figure 5-9 Evidence of water leak in corridor of BLD-01650 (paint stretched to contain water).

Figure 5-10 South Nursing Wing storefront glazing damage leading to entrance posted RESTRICTED USE at BLD-01656.

Figure 5-11 South Nursing Wing storefront glazing damage leading to RESTRICTED USE posting at BLD-01656. Yellow line indicates amount of movement in the top track.

Damage was also observed at seismic separation joints (Figures 5-12 through 5-14). Some damage to rooftop equipment occurred, especially to electrical components on cantilevered supports (Figures 5-15 and 5-16). Dust released as a result of movement between diffusers and sprinkler escutcheons and the ceiling compromised infection control and prevented the use of two operating rooms. In some cases, spontaneous opening of ceiling access doors compromised the sterile environment. It was also reported that cracking in ceiling or walls affected the air balance and the ability to maintain positive pressure in some operating rooms. Power was lost at this facility and emergency generators were activated for approximately three hours; the system performed as originally designed. All essential services were maintained.

Figure 5-12 Movement at seismic joint between BLD-01652 and BLD-01650 Main Hospital.

Figure 5-13 Damage to seismic joint flashing between BLD-01654 and BLD-01655.

Figure 5-14 Damage to seismic joint cover at second floor of BLD-01659.

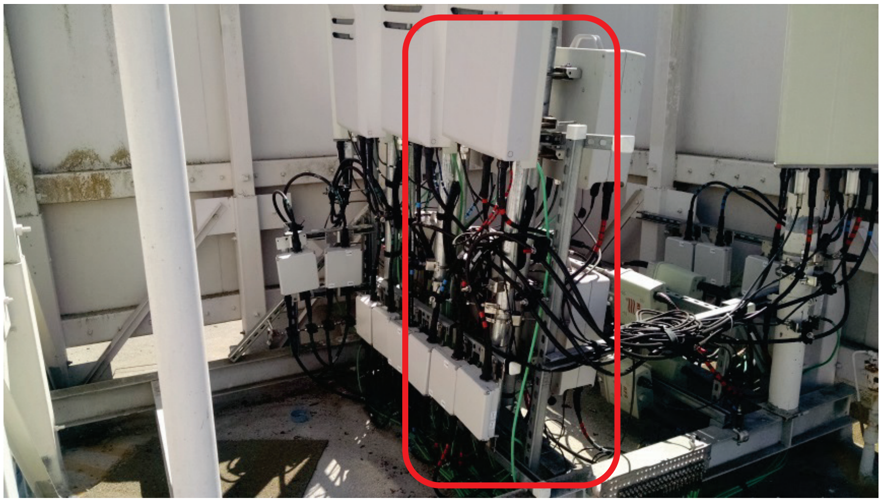


Figure 5-15 Telephone rack on cantilevered supporting frame at roof of BLD-01650 severely out of plumb.

Figure 5-16 Electrical panel on cantilevered supports damaged at roof of BLD-01656.

Suspended light fixtures were damaged when they struck one another (Figure 5-17). Content damage also occurred, including overturned office equipment, book cases, loss of contents off of shelving, and other items (Figures 5-18 to 5-20).

- **Time until Full Occupancy:** Although two operating rooms were temporarily out of service, no services were disrupted.

Figure 5-17 Damaged light fixture in BLD-05309.

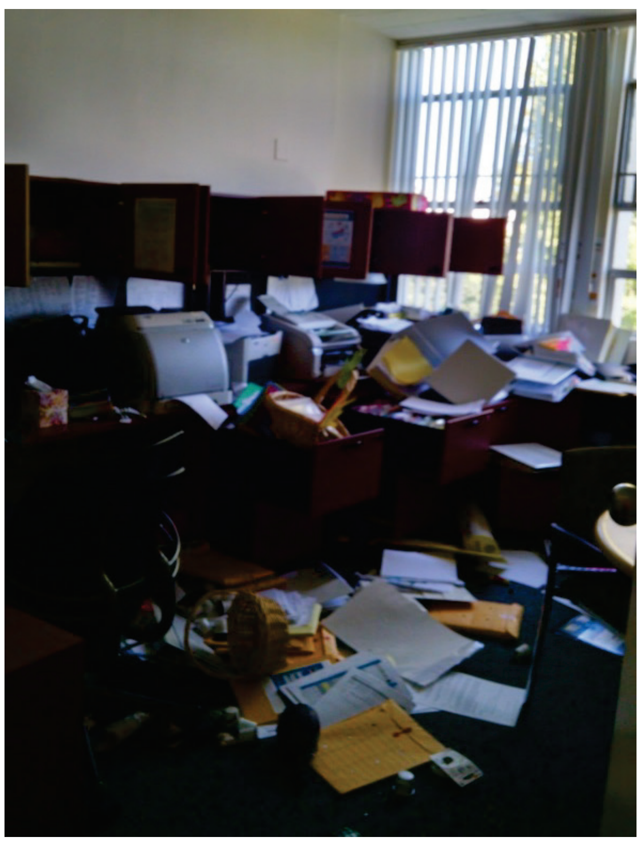


Figure 5-18 Contents damage in BLD-01650.

Figure 5-19 Contents damage in BLD-01650.

Figure 5-20 Unanchored vending machine in BLD-01656 shifted over 12 inches.

5.2.2 Sutter Solano Medical Center (OSHPD Facility ID 12525)

- **Structural System, Height, Year Built:** The Sutter Solano Medical Center in Vallejo consists of four individual buildings constructed between 1966 and 1989. Buildings range from one- to four-stories in height, and construction types include steel moment frames, steel braced frames, concrete shear walls, and reinforced masonry. A plan of the hospital is shown in Figure 5-21. Descriptions of the buildings are presented in Table 5-4.

- **Posting Placard:** All of the buildings at Sutter Solano Medical Center were posted INSPECTED.
- **Structural and Nonstructural Performance:** No structural or nonstructural damage was observed.
- **Time until Full Occupancy:** No services were disrupted.

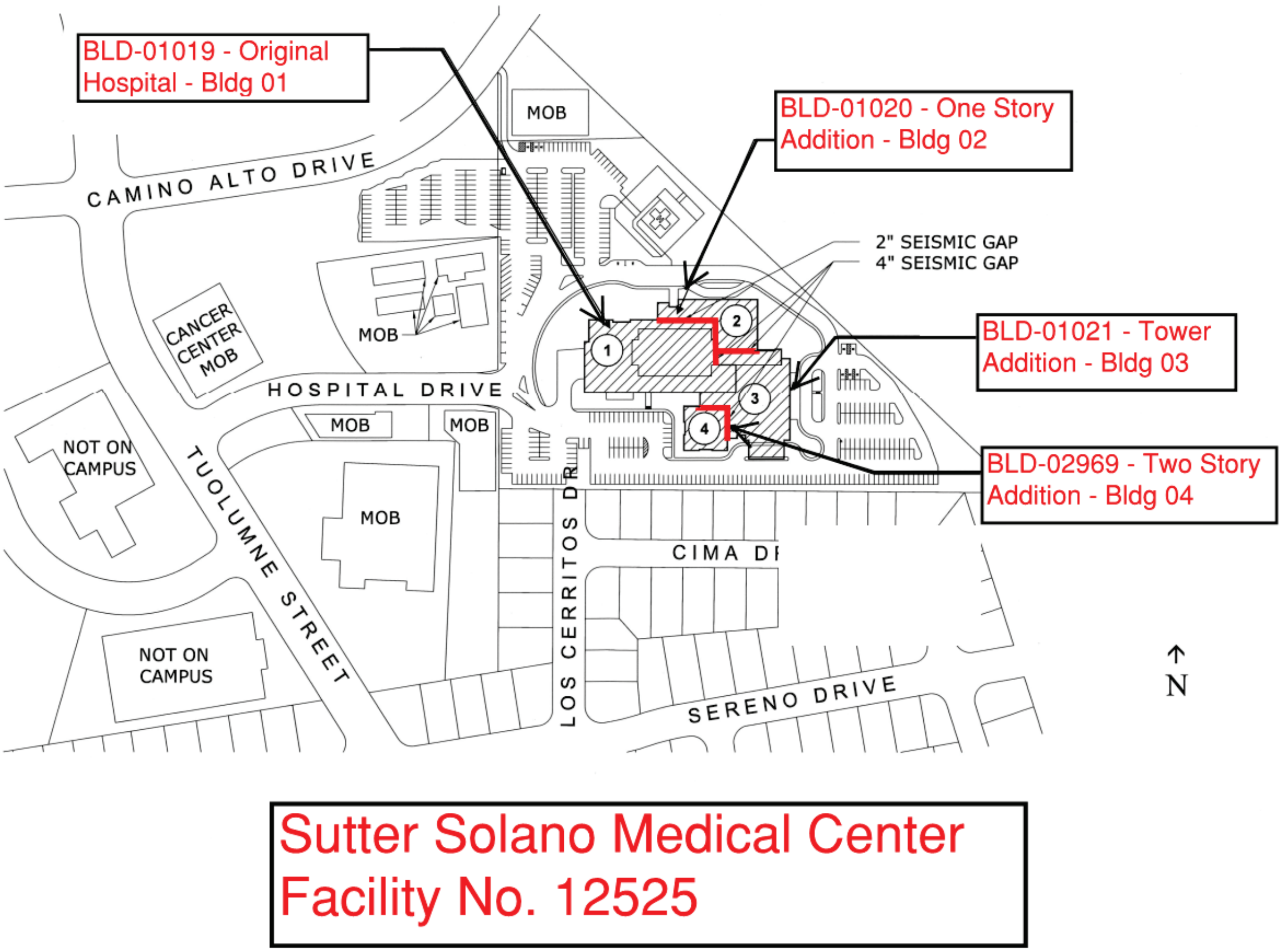


Figure 5-21 Sutter Solano Medical Center site plan.

Table 5-4 Sutter Solano Medical Center Building Descriptions and Posting

Building Number	Building Name	Stories	Structural System	Year Built	SPC Class	NPC Class	Posting
BLD-01019	Original Hospital	4	C2	1966	2	2	INSPECTED
BLD-01020	One Story Addition	1	RM1	1976	4	2	INSPECTED
BLD-01021	Tower Addition	4	S1	1989	3s	2	INSPECTED
BLD-02969	Two Story Addition	2	S2	1989	4s	2	INSPECTED

5.2.3 Kaiser Foundation Hospital – Rehabilitation Center Vallejo (OSHPD Facility ID 13142)

- **Structural System, Height, Year Built:** The Kaiser Foundation Hospital – Rehabilitation Center in Vallejo consists of seven individual buildings. The oldest was constructed in 1970, and the newest was completed in 2010. A plan of the hospital is shown in Figure 5-22. Descriptions of the buildings are presented in Table 5-5.

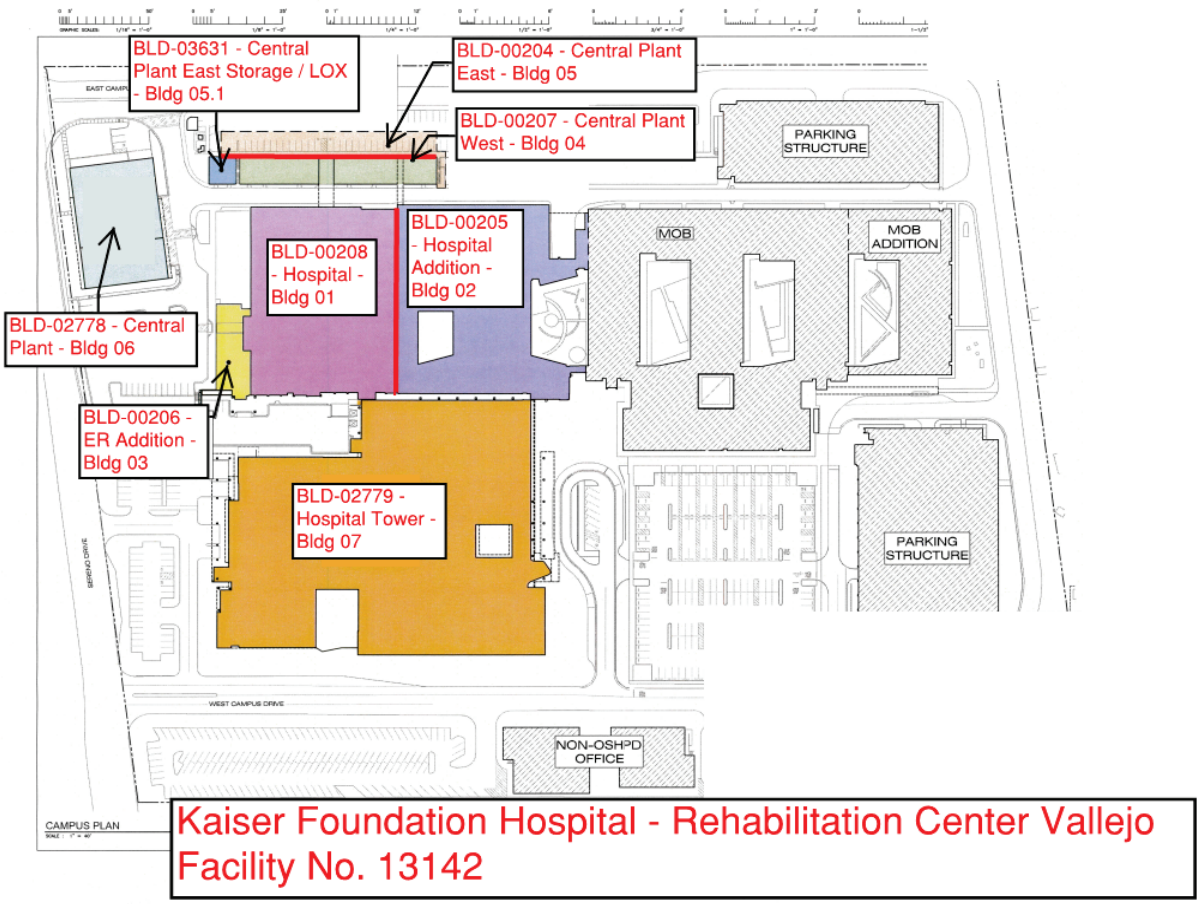


Figure 5-22 The Kaiser Foundation Hospital – Rehabilitation Center Vallejo site plan.

- **Posting Placard:** All of the buildings were posted INSPECTED.
- **Structural Performance:** Observed structural damage was limited to cracking in the slab-on-grade.
- **Nonstructural Performance:** Nonstructural damage included a second floor air handler unit that shifted, movement of unbraced mechanical, electrical, and plumbing distribution systems that caused minor damage to steel fireproofing, and cracks observed in non-bearing partition walls. Utility power was lost, and the emergency generator came on-line.
- **Time until Full Occupancy:** No services were disrupted.

Table 5-5 Kaiser Foundation Hospital Building Descriptions and Posting

Building Number	Building Name	Stories	Structural System	Year Built	SPC Class	NPC Class	Posting
BLD-00204	Central Plant East	1	C2	1993	4	2	INSPECTED
BLD-00205	Hospital Addition	3	S1	1996	3s	2	INSPECTED
BLD-00206	ER Addition	1	S1	2001	3s	2	INSPECTED
BLD-00207	Central Plant West	1	C2, RM1	1970	2	2	INSPECTED
BLD-00208	Hospital	8	S1, C2	1970	1	2	INSPECTED
BLD-02779	Hospital Tower	5	S2	2010	5s	4	INSPECTED
BLD-03631	Central Plant East Storage / LOX	2	C2	1993	4	2	INSPECTED

5.2.4 Sonoma Valley Hospital (OSHPD Facility ID 11064)

- **Structural System, Height, Year Built:** The Sonoma Valley Hospital in Sonoma consists of eleven individual buildings. The oldest was constructed in 1958 and the newest is still under construction. A plan of the hospital is shown in Figure 5-23. Descriptions of the buildings are presented in Table 5-6.
- **Posting Placard:** All of the buildings at Sonoma Valley Hospital were posted INSPECTED.
- **Structural Performance:** Observed structural damage was limited to cracking in the slab-on-grade.
- **Nonstructural Performance:** Nonstructural damage was limited to minor cracking in gypsum board walls.
- **Time until Full Occupancy:** No services were disrupted.

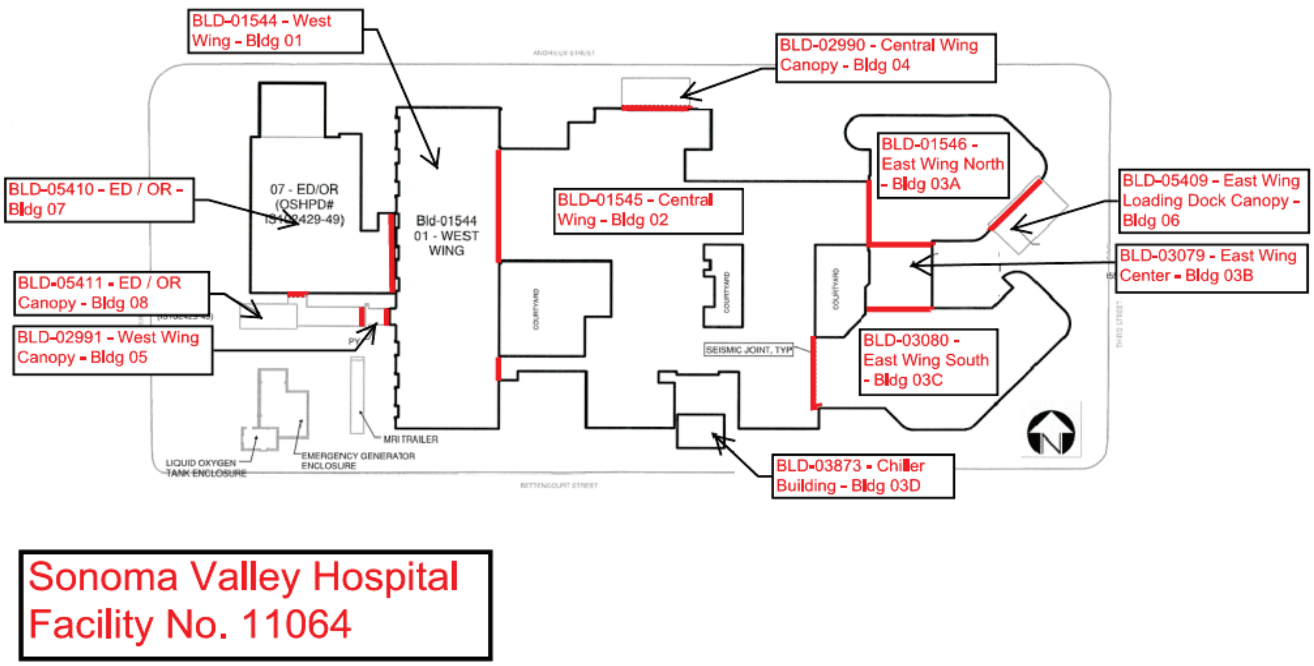


Figure 5-23 The Sonoma Valley Hospital site plan.

Table 5-6 Sonoma Valley Hospital Building Descriptions and Posting

Building Number	Building Name	Stories	Structural System	Year Built	SPC Class	NPC Class	Posting
BLD-01544	West Wing	3	C2, RM1	1970	2	1	INSPECTED
BLD-01545	Central Wing	1	C2, RM1, W1	1958	2	1	INSPECTED
BLD-01546	East Wing North	1	RM1	1986	4	1	INSPECTED
BLD-02990	Central Wing Canopy	1	S1	19893s	3s	1	INSPECTED
BLD-02991	West Wing Canopy	1	S1	1989	3s	1	INSPECTED
BLD-03079	East Wing Center	1	S1	1986	3	1	INSPECTED
BLD-03080	East Wing South	1	W2	1986	4	1	INSPECTED
BLD-03873	Chiller Building	1	RM2	1986	4	1	INSPECTED
BLD-05409	East Wing Loading Dock Canopy	1	S1	2012	5s	-	INSPECTED
BLD-05410	ED / OR	Unknown	Unknown	Unknown	5s	-	INSPECTED
BLD-05411	ED / OR Canopy	Unknown	Unknown	Unknown	5s	-	INSPECTED

5.2.5 St. Helena Hospital Center for Behavioral Health (OSHPD Facility ID 11013)

- **Structural System, Height, Year Built:** The St. Helena Hospital Center for Behavioral Health in Vallejo is a single, one-story building from the 1950s.

- **Posting Placard:** All of the buildings were posted INSPECTED.
- **Structural and Nonstructural Performance:** No structural or nonstructural damage was observed.
- **Time until Full Occupancy:** No services were disrupted.

5.2.6 St. Helena Hospital (OSHPD Facility ID 10366)

- **Structural System, Height, Year Built:** The St. Helena Hospital in St. Helena consists of six individual buildings. The oldest was constructed in 1949, and the newest was completed in 2012. A plan of the hospital site is shown in Figure 5-24 and an overview of the building is shown in Figure 5-25. Descriptions of the buildings are presented in Table 5-7.

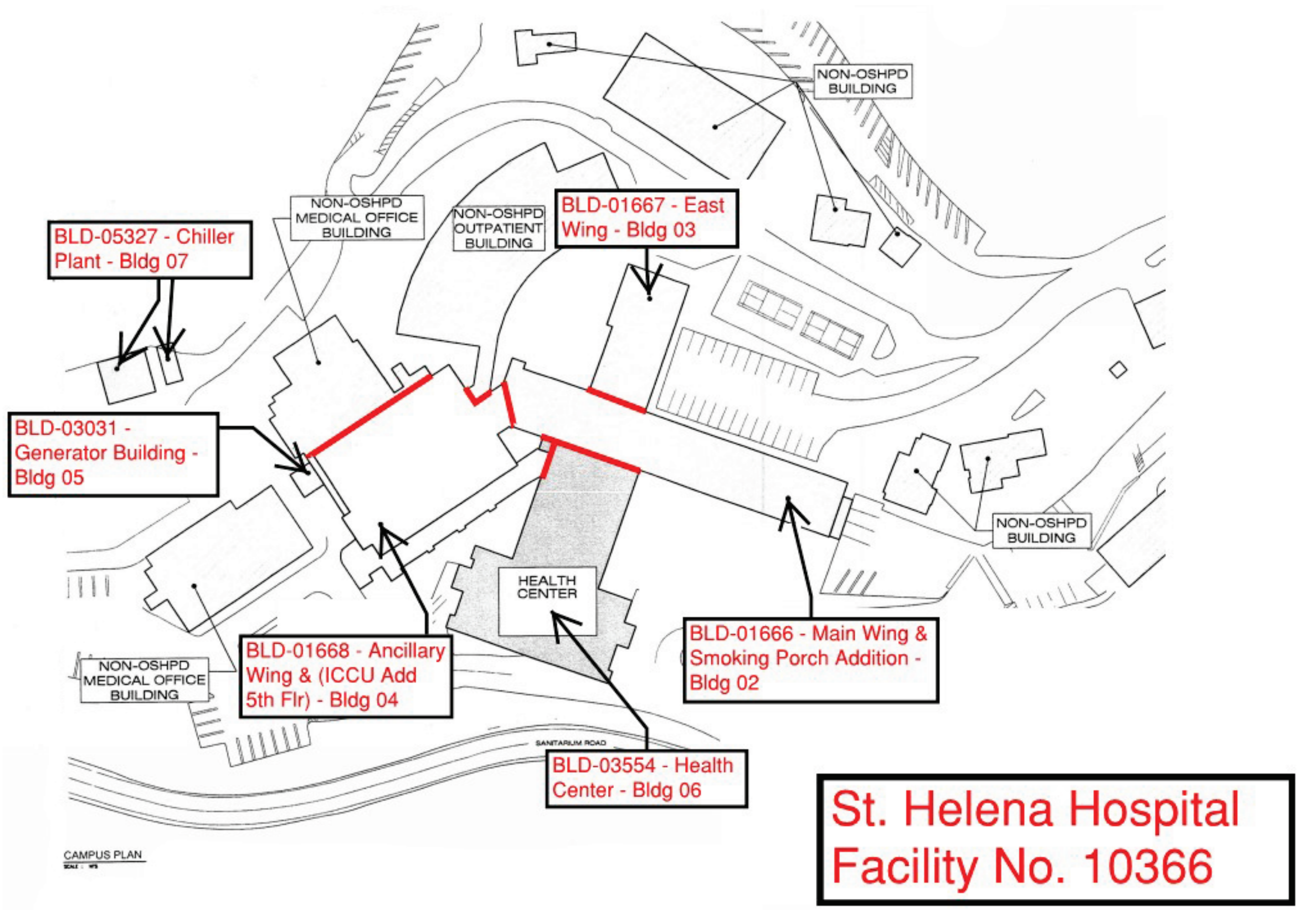


Figure 5-24 St. Helena Hospital site plan.

Figure 5-25 Front entrance.

Table 5-7 St. Helena Hospital Building Descriptions and Posting

Building Number	Building Name	Stories	Structural System	Year Built	SPC Class	NPC Class	Posting
BLD-01666	Main Wing & Smoking Porch Addition	5	C1, C2	1949	2	2	INSPECTED
BLD-01667	East Wing	6	C1, C2	1949	2	2	INSPECTED
BLD-01668	Ancillary Wing (ICCU Addition 5th floor)	5	S2	1980	4	2	INSPECTED
BLD-03031	Generator Building	1	RM1	1980	4	2	INSPECTED
BLD-03554	Health Center	5	C2	1968	2	2	INSPECTED

- **Posting Placard:** All of the buildings were posted INSPECTED.
- **Structural Performance:** No structural damage was observed.
- **Nonstructural Performance:** Limited nonstructural damage was observed. In particular, a 1 million gallon capacity tank that is part of the domestic water supply was disturbed such that sediment from the tank began appearing at plumbing fixtures. In response, the facility relied on bottled water, and the surgery area switched to using alcohol based hand cleaner. The separation joints between Buildings 01666 and 01667 and Buildings 01666 and 01668 suffered cosmetic damage (Figures 5-26 through 5-28). Two elevators went off-line following the earthquake, and had to be returned to service by elevator service personnel. A 35-minute power interruption occurred following the earthquake and the two emergency generators operated as designed and then switched back to normal power.

- **Time until Full Occupancy:** No services were disrupted.

Figure 5-26 Cracking along the separation joint between BLD-01666 and BLD-01667 at the roof penthouse.

Figure 5-27 Cracking of the 1/2 inch to 1 inch thick topping slab over original roof slab inside the roof penthouse along the separation joint.

Figure 5-28 Evidence of movement along the separation joint at the fourth floor between BLD-01668 and BLD-01666.

5.3 Performance of Skilled Nursing Facilities

Skilled Nursing Facilities (SNF) are also under the jurisdiction of OSHPD. Most SNFs are single-story wood or light metal framed buildings.

Figure 5-29 Map of skilled nursing facilities (indicated with red pins) investigated following the South Napa Earthquake (epicenter indicated with a red star) (image source: Google Earth).

Table 5-8 presents a list of the skilled nursing facilities are located near the epicenter and were investigated by OSHPD following the earthquake.

Table 5-8 List of Skilled Nursing Facilities Investigated by OSHPD

OSHPD Facility ID	Skilled Nursing Facility Name	Distance to Epicenter (miles)
23264	The Meadows Of Napa Valley Care Center	7.1
20361	Piner's Nursing Home	11.3
20363	Nazareth Skilled Nursing Center - Napa	11.6
21030	Windsor Vallejo Nursing & Rehabilitation Center	11.6
22929	Crestwood Manor - Vallejo	11.6
27780	Crestwood Hospital	11.6
25194	Napa Valley Care Center	12.2
20368	Golden Living Center - Napa	12.4
21062	Sonoma Healthcare Center	14.2
21015	Springs Road Living Center	14.9
21046	Golden Living Center - London House Sonoma	14.9
21012	Heartwood Avenue Healthcare	15.3
21061	Sonoma Acres	16.4

All of the skilled nursing facilities were posted INSPECTED. No structural damage was identified at any of the facilities. In some cases, nonstructural damage was observed, including cracks in gypsum wallboard ceilings, damage to light fixtures, and leaking water pipes.

Observed structural and nonstructural damage at each facility are presented in Table 5-9.

Table 5-9 Skilled Nursing Facilities Building Descriptions and Posting

The Meadows Of Napa Valley Care Center (OSHPD Facility ID 23264)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04897	INSPECTED	None	Small cracking at two locations. One crack at the ceiling drywall across the corridor and another with uplift in what appears to be a nonstructural slab that is within the center of a connecting corridor between buildings. The emergency generator started due to power outage in the area. The gas was turned off pending the utility check of the building.

Table 5-9 Skilled Nursing Facilities Building Descriptions and Posting (Continued)

Piner s Nursing Home (OSHPD Facility ID 20361)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04022	INSPECTED	None	The facility had minor cosmetic damage, with some light fixtures damaged. Hot water heater piping was damaged due to unsupported piping. The emergency generator started due to the power outage in the area.
Nazareth Skilled Nursing Center Napa (OSHPD Facility ID 20363) Exterior Review Only			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04023	INSPECTED	None observed (facility is unoccupied, could not inspect the interior of this structure).	There was a leak in the sprinkler line at the front entrance under the canopy.
Windsor Vallejo Nursing & Rehabilitation Center (OSHPD Facility ID 21030)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04256	INSPECTED	None	None
Crestwood Manor Vallejo (OSHPD Facility ID 22929)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04877	INSPECTED	None	None
Crestwood Hospital (OSHPD Facility ID 27780)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-05744	INSPECTED	None	None
Napa Valley Care Center (OSHPD Facility ID 25194)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04955	INSPECTED	None	None
Golden Living Center Napa (OSHPD Facility ID 20368)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04024	INSPECTED	None	Generator started due to loss of utility power.
Sonoma Healthcare Center (OSHPD Facility ID 21062)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04268	INSPECTED	None	The generator required manual start.

Table 5-9 Skilled Nursing Facilities Building Descriptions and Posting (Continued)

Building Number	Posting	Structural Damage	Nonstructural Damage
Springs Road Living Center (OSHPD Facility ID 21015)			
BLD-04252	INSPECTED	None	None
Golden Living Center London House Sonoma (OSHPD Facility ID 21046)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04264	INSPECTED	None	Utility power lost. The emergency generator alarm signal was on, the panel showed the generator was not in auto-start mode but it did start up.
Heartwood Avenue Healthcare (OSHPD Facility ID 21012)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04250	INSPECTED	None	None
Sonoma Acres (OSHPD Facility ID 21061)			
Building Number	**Posting**	**Structural Damage**	**Nonstructural Damage**
BLD-04267	INSPECTED	None	Utility power lost, the emergency generator came on without issues.

5.4 Performance of Medical Office Buildings

Many medical office buildings are designed and constructed under the authority of local jurisdictions and not OSHPD. They are generally not designed to remain operational after an earthquake. Most medical office buildings are one or two stories in height, many are of light-framed construction.

There was no systematic investigation of the performance of medical office buildings following the South Napa earthquake. However, two medical office buildings in Napa experienced significant nonstructural damage to rooftop equipment. Air handler units, water heaters, and other rooftop equipment shifted and failed at the connections to the structure, despite the fact that they were constructed within the past ten years.

5.5 Summary

Hospital buildings generally performed well and remained open and functional to serve the communities affected by the earthquake. There was no significant structural damage reported, and no building was closed as a result of structural damage. Minor nonstructural damage was reported, and three buildings at Queen of the Valley Hospital in Napa were posted RESTRICTED USE as a result. Nonstructural damage included damage to suspended acoustic tile ceilings, minor gypsum wallboard cracking, damage to a storefront glazing system, damage to exterior wall cladding, a small number of broken water pipes, movement of unanchored equipment and furnishings, and damage to expansion joint covers. Loss of power or a drop in power were reported at several hospitals. All emergency generators came on-line and provided electrical service to the facilities. Queen of the Valley Hospital in Napa had elevators out of service as a result of the earthquake in two hospital buildings. Two of the elevators in the old original building suffered damage and were taken out of service.

Given the limited structural damage and continued operation of critical equipment and systems, hospitals that experienced the South Napa earthquake met the stated objective of the *Hospital Seismic Safety Act*. The level and duration of earthquake shaking experienced at the hospital sites coupled with OSHPD's enforcement of the H*ospital Seismic Safety Act* are believed to be largely responsible for the good performance.

The earthquake did, however, highlight some of the challenges associated with sustained operations following an earthquake. In order to maintain functionality, hospitals must have operable ventilation systems, be capable of containing contagious diseases, and provide an environment that does not adversely affect immune suppressed patients. Adequate sanitation and lighting are required, as are functional emergency power systems and medical gas systems.

Both new and older hospital buildings experienced some nonstructural damage which, in a larger earthquake with longer duration, could have impacted the continuity of service. In particular, damage to exterior walls and ceilings in operating rooms was observed to have affected the ability to maintain required positive air pressure and the required number of air changes, thus impacting the sterile environment. Also, damage to suspended piping could have led to flooding and closure of portions of the hospital. Finally, damage to furnishings, which are generally not regulated by the building code, could have led to serious injuries.

5.6 Recommendations

The performance of healthcare facilities in the South Napa earthquake suggests several areas for possible improvement and further study, including:

1. Detailing associated with cladding should be studied to ensure the ability to accommodate building drifts without unacceptable damage. Acceptable performance should be defined based on requirements for

maintaining sterile environments where required. This is further discussed in Chapter 10.

2. Exemptions in current building codes that exempt seismic restraint requirements for most furnishings and contents should be examined to assess whether the resulting risk of injury is acceptable. This is more fully explored in Chapter 10.

3. Immediately following the earthquake, OSHPD deployed multi-disciplinary teams to conduct damage assessments of hospitals. Efforts should be made to develop a uniform format for the collected data to facilitate detailed follow-up studies.

4. OSHPD postearthquake assessment teams must often make damage assessment decisions based on limited information on the structural attributes of the particular building. Consideration should be given to including a Structural Engineer engaged by the hospital in the assessment process, one who is familiar with the specific buildings and can provide expert information on the structure.

5. Strong-motion instrumentation of hospitals should be expanded, and the information available from strong-motion instrumentation should be used as part of any future initiative to set related strategic goals for postearthquake recovery and assessment of design practices.

6. The potential value of early warning systems should be explored as hospitals should be among the first users. An early warning system could give surgeons time to stop surgery, protect and cover a patient in the operating room, and take other steps to minimize disruption of patient care, such as opening elevators at the closest floor.

Chapter 6

Performance of School Facilities

6.1 Introduction

This chapter provides an overview and summary of seismic performance of school buildings and contents in the South Napa earthquake. There was little or no structural damage to the schools affected by the earthquake. Several schools did, however, experience nonstructural damage to architectural, mechanical, or electrical components, including suspended ceilings, light fixtures, equipment, and furniture. Some of the damage could have been life threatening had the earthquake occurred during school hours.

Following the significant damage to school buildings in the 1933 Long Beach earthquake, the *Field Act* was established in California within 30 days of the earthquake to mandate earthquake-resistant construction of schools and to establish the Division of the State Architect (DSA, formerly Office of the State Architect). The *Field Act* requires DSA to review the design, construction, alteration, addition, or rehabilitation of public schools, kindergarten through 12th grade, and community colleges. Certain non-building projects and minor alteration projects below a cost threshold may be exempt from DSA review under the *Field Act*. For those projects, school districts must ensure on their own that work complies. Private schools are regulated by local governments. A 1986 *Private Schools Seismic Safety Act* provides some enhancements compared to prior laws.

After construction is completed, for both public and private schools, the owners of the school buildings are responsible for ensuring that school buildings are seismically safe and accessible. Many aspects in schools are not independently regulated, including furniture and minor nonstructural alterations.

6.2 Napa Valley Unified School District

There are 31 public school sites within the Napa Valley Unified School District. Figure 6-1 shows a map of all schools in the school district. The schools are generally one- or two-story buildings of wood-frame or reinforced masonry construction dating back to as early as the 1930s.

Following the earthquake, schools were closed for inspection for two days. The District deployed three, three-person teams, each of which included an architect, a structural engineer, and a school district official. The school district also retained mechanical and electrical engineering consultants to inspect each school site and recommend repairs, where needed. Classes resumed on August 27, 2014 except at the Stone Bridge School, which did not reopen until September 2, 2014 due to a broken water main.

Napa Valley Unified School District filed a preliminary estimate of $8 million for building repairs. Replacing and repairing contents damaged by the earthquake was estimated at $9 million. Virtually none of the damage reported is characterized as "structural damage" impacting the safety of the gravity or seismic force-resisting systems in the school buildings. The observed damage was essentially all nonstructural, affecting the building cladding, interior partitions and ceilings, mechanical, electrical, and plumbing equipment and distributions systems, and contents. The following sections describe the observed damage.

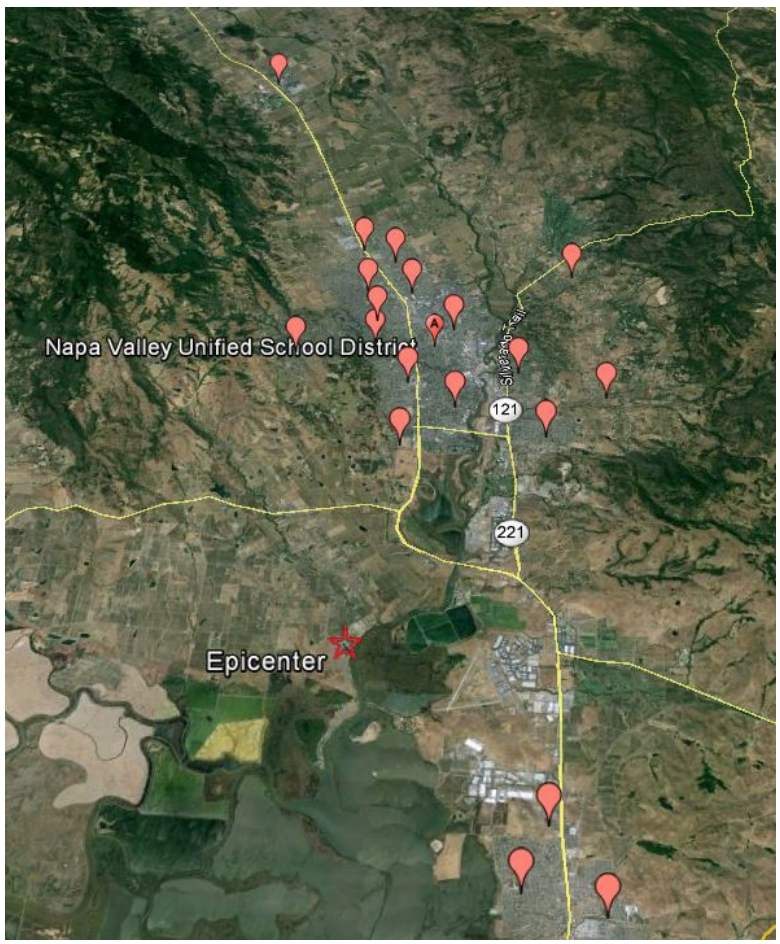


Figure 6-1 Map showing the locations of all schools in the Napa Valley Unified School District (image source: Google Maps).

6.2.1 Suspended Pendant Fixtures

Suspended pendant light fixtures are common in classrooms. The pendant fixtures in the Napa Valley Unified School District facilities are typically suspended with aircraft (high-strength) cables fastened to the structural framing above with lag screws and to the light fixture with mechanical fasteners specific to each particular light fixture. The cables are generally provided with a means to adjust their length through the use of a cable gripper. Some installations include strut backing as a means to align the fixtures and reduce the number of hanger supports required. Some of the fixtures have supplemental wire bracing to walls or to the ceiling framing to limit swaying.

Past earthquakes dating to the 1952 Kern County earthquake (in southern San Joaquin Valley) have demonstrated the vulnerability of pendant-hung fixtures to earthquake damage. Code requirements over the years have attempted to address many of the deficiencies observed in past earthquakes. An exception in Section 13.5.1 of ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*, (ASCE, 2010) allows suspended nonstructural components to be exempt from loading criteria under certain conditions. ASCE/SEI 7-10 also requires nonstructural connections that are "positively fastened" and do not rely on frictional resistance "produced by the effects of gravity." DSA has also published Interpretation of Regulations, IR 16-9, *Pendant Mounted Light Fixtures*, (DSA, 2010) regarding pendant fixtures. However, even relatively recent installations of pendant fixtures have been observed to be vulnerable to damage that could threaten student safety. Of the 31 schools sites in Napa, 11 reported some damage to pendant fixtures and in five buildings at least one fixture fell from the ceiling.

Several different types of damage were observed:

- The aircraft cable supporting the fixture slipped from the gripping connection (Figures 6-2 and 6-3)
- The connection of the aircraft cable to the fixture failed at the fixture (Figures 6-4 through 6-6). Figure 6-7 shows an undamaged connection.
- The bracing wire pulled out of its support (Figure 6-8)
- The lens cover dislodged (Figure 6-9)
- The bulbs dislodged and fell (Figure 6-10)

In the five schools where at least one fixture was dislodged, similar vulnerable light fixtures and their support systems were removed and replaced by the school district. Fixtures were removed and replaced in a total of 48 classrooms and two offices.

Figure 6-2 Damaged light fixtures in classroom at Pueblo Vista Elementary School (photo by Quattrocchi Kwok Architects).

Figure 6-3 Aircraft cable slipped out of the cable gripping support fixture at Pueblo Vista Elementary School (photo by Quattrocchi Kwok Architects).

Figure 6-4 Damaged light fixtures in Irene M. Snow Elementary School (photo by Quattrocchi Kwok Architects).

Figure 6-5 Damaged light fixtures in Napa Valley Language Academy (photo by Quattrocchi Kwok Architects).

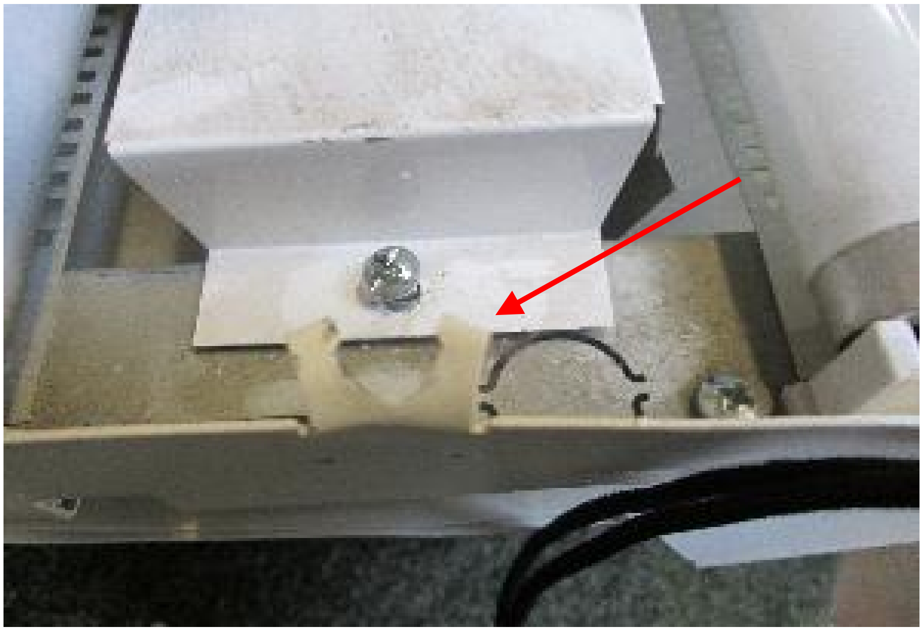


Figure 6-6 Failed connection at light fixture in Napa Valley Language Academy (photo by Quattrocchi Kwok Architects).

Figure 6-7 Undamaged connection to light fixture in Napa Valley Language Academy (photo by Quattrocchi Kwok Architects).

Figure 6-8 Bracing wire pulled out of wall at Harvest Middle School (photo by Quattrocchi Kwok Architects).

Figure 6-9 Open lens covers at Shearer Elementary School (photo by Quattrocchi Kwok Architects).

Figure 6-10 Dislodged fluorescent bulbs at Shearer Elementary School (photo by Quattrocchi Kwok Architects).

6.2.2 Furniture

Schools and classrooms are filled with bookcases, cabinets, shelving, file cabinets, and similar furniture. Unrestrained furniture is subject to sliding in an earthquake. Tall, slender items, such as bookcases and shelving units, are prone to overturning. When items overturn, they pose a particularly hazardous condition for occupants either because they inflict direct injury or can block egress routes.

Some furniture items are installed at the time a school is constructed, but most are installed on an as-needed basis throughout the life of the school facility. Since the installation of furniture over 6 feet tall is not regulated by the building code, the adequacy of the installation related to earthquake safety depends on voluntary implementation of good seismic practice. Such good practice is provided in publically available guidelines such as the *Guide and Checklist for Nonstructural Earthquake Hazards in California Schools* (Governor's Office of Emergency Services, 2003), and FEMA E-74, *Reducing the Risk of Nonstructural Earthquake Damage* (FEMA, 2012a).

There was considerable movement of furniture inside classrooms during the 2014 South Napa earthquake. In many instances, the damage could have caused serious injury to students or staff had the rooms been occupied at the time of the earthquake or could have hindered emergency evacuation after the earthquake. In all cases where furnishings slid or overturned, there were no visible measures in place to prevent such movement. Figures 6-11 through 6-15 illustrate examples where overturned furniture created a safety risk. In cases where furniture did not overturn or move, contents were shed, sometimes posing a potential safety hazard as well (Figures 6-16 and 6-17).

In contrast to free-standing furnishings, built-in casework approved by DSA installed at the time of construction performed uniformly well. No wall-mounted cabinets experienced damage other than the loss of contents.

Figure 6-11 Overturned cabinets in Irene M. Snow Elementary School (photo by Quattrocchi Kwok Architects).

Figure 6-12 Overturned bookcase in Stone Bridge School (photo by Quattrocchi Kwok Architects).

Figure 6-13 Overturned bookcase blocking exit in Stone Bridge School (photo by Quattrocchi Kwok Architects).

Figure 6-14 Overturned bookcase at doorway in Harvest Middle School (photo by Quattrocchi Kwok Architects).

Figure 6-15 Overturned file cabinet in Harvest Middle School (photo by Quattrocchi Kwok Architects).

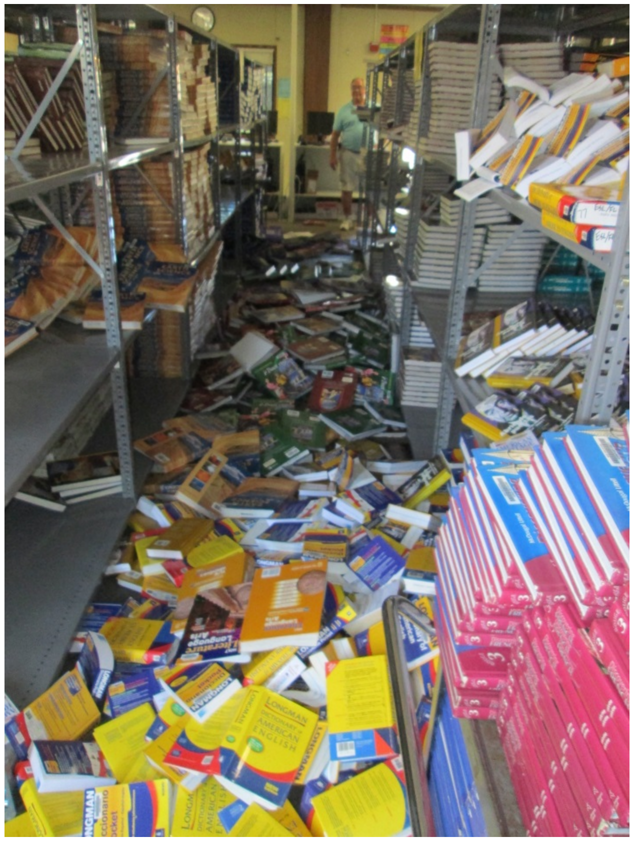


Figure 6-16 Dislodged books in Napa High School (photo by Quattrocchi Kwok Architects).

Figure 6-17 Contents dislodged from counters in Stone Bridge School wood shop (note drill press on floor) (photo by Quattrocchi Kwok Architects).

6.2.3 Ceilings

Several ceiling types were affected by the earthquake, including suspended acoustic tile, gypsum wallboard or plaster fastened directly to framing (some with glued acoustic tiles), and combined systems. In general, damage to ceilings was very limited. In suspended acoustic tile ceilings, acoustic tiles were reported to have been dislodged in five out of 31 schools and damage to the ceiling grid was reported in three schools (Figures 6-18 and 6-19). The adequate performance of ceilings is likely related to a combination of factors including the duration and magnitude of shaking, the small size of the ceilings, bracing provided by classroom walls, the lateral bracing system including ceiling tile restraints, and the quality of installations.

Although there was only limited damage to ceilings caused by the 2014 South Napa earthquake, many suspended ceilings are actively being replaced. Since some ceiling components were variously bent, broken, and "altered" by the earthquake, this triggered replacements in accordance with Interpretation of Regulations, IR 25-2.13, *Metal Suspension Systems for Lay-In Panel Ceilings: 2013 CBC*, (DSA, 2014) that states: "The entire ceiling shall be upgraded to meet the current requirements of the CBC and this IR if any portion of the grid system is cut or altered." Thus, the limited earthquake damage to predominantly older ceiling systems is being used to trigger improvements to reduce damage in future events.

Figure 6-18 Dislodged tiles and bent edge angle in a Silverado Middle School classroom (photo by Quattrocchi Kwok Architects).

Figure 6-19 Damage to suspended acoustic tile ceiling in Napa Junction Elementary School (photo by Quattrocchi Kwok Architects).

6.2.4 Interior Partitions

Interior partitions observed typically consisted of full height wood or metal studs with gypsum wallboard or plaster sheathing. Reported damage was limited to minor cracking (Figures 6-20). In one school small areas of adhered ceramic tiles were dislodged (Figure 6-21).

Figure 6-20 Gypsum wallboard cracking in New Technology High School (photo by Quattrocchi Kwok Architects).

Figure 6-21 Dislodged adhered tile in Harvest Middle School (photo by Quattrocchi Kwok Architects).

6.2.5 Glazing

Damage to glazing was limited. Broken windows were reported at three schools, but only one window was damaged at each school (Figure 6-22). Some schools experienced minor movement in the window framing system.

Figure 6-22 Broken window at Harvest Middle School (photo by Quattrocchi Kwok Architects).

6.2.6 Mechanical and Plumbing Systems

In general, there was limited damage to mechanical and plumbing systems in schools. The earthquake restraint straps on two water heaters failed, but the tanks did not topple (Figures 6-23 and 6-24). A water main broke at Stone Bridge School, and a 28,000-gallon water tank was damaged at New Technology High School. The tank, which was anchored to a concrete foundation, was damaged at the connection between anchor brackets and the tank wall, causing ruptures in the tank wall and water release. There was also local buckling of the tank wall near the base.

The rooftops of most schools support piping and equipment on wood sleepers. In many instances, the sleepers are unanchored. The South Napa Earthquake caused movement of piping, conduit, and equipment; however there was no loss of function reported for any of the rooftop equipment. Examples of the damage to rooftop equipment are shown in Figures 6-25 to 6-29.

Figure 6-23 Failed earthquake restraining strap on water heater (photo by Quattrocchi Kwok Architects).

Figure 6-24 Failed earthquake restraining strap at Shearer Elementary School (photo by Quattrocchi Kwok Architects).

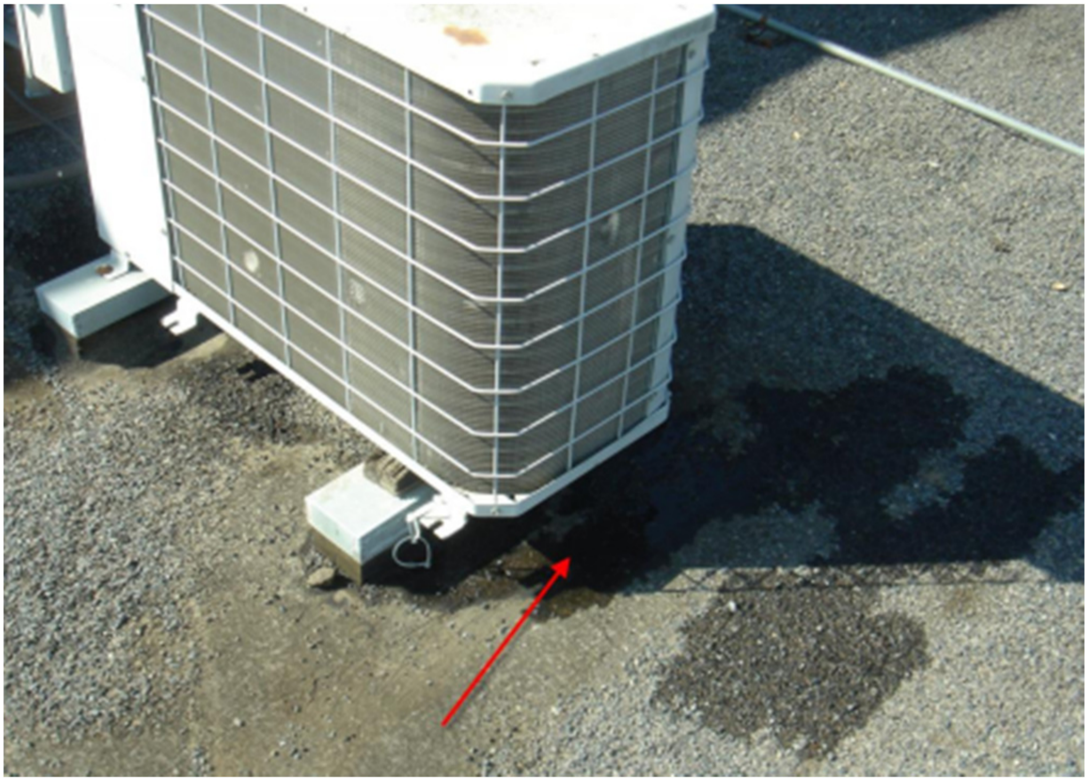


Figure 6-25 Shifted rooftop condensate unit at Redwood Middle School (photo by Quattrocchi Kwok Architects).

Figure 6-26 Shifted rooftop make-up air unit at Napa High School (photo by Quattrocchi Kwok Architects).

Figure 6-27 Broken condensate connection at air conditioning unit, Redwood Middle School (photo by Quattrocchi Kwok Architects).

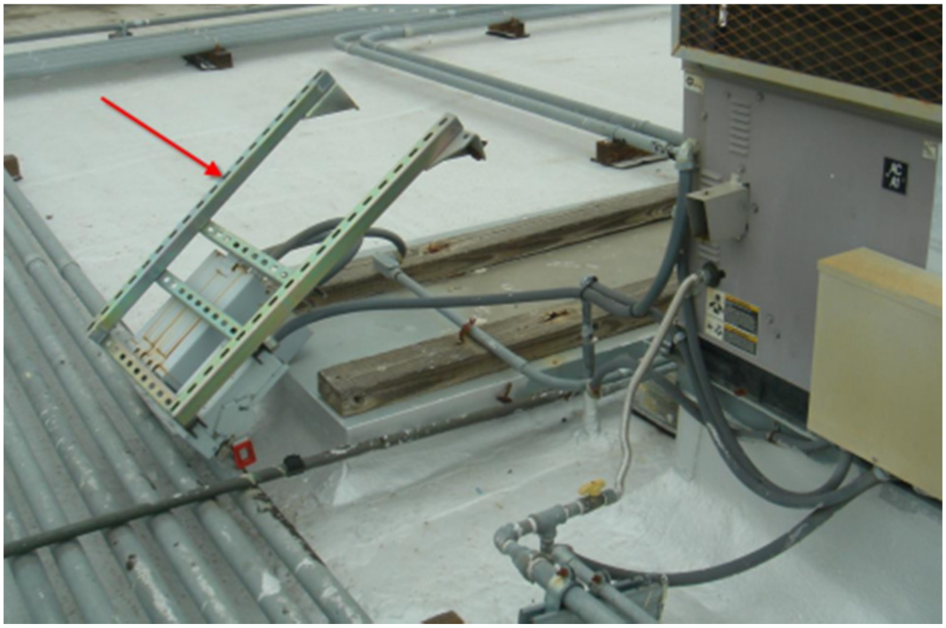


Figure 6-28 Toppled disconnect for rooftop air handler at Napa High School (photo by Quattrocchi Kwok Architects).

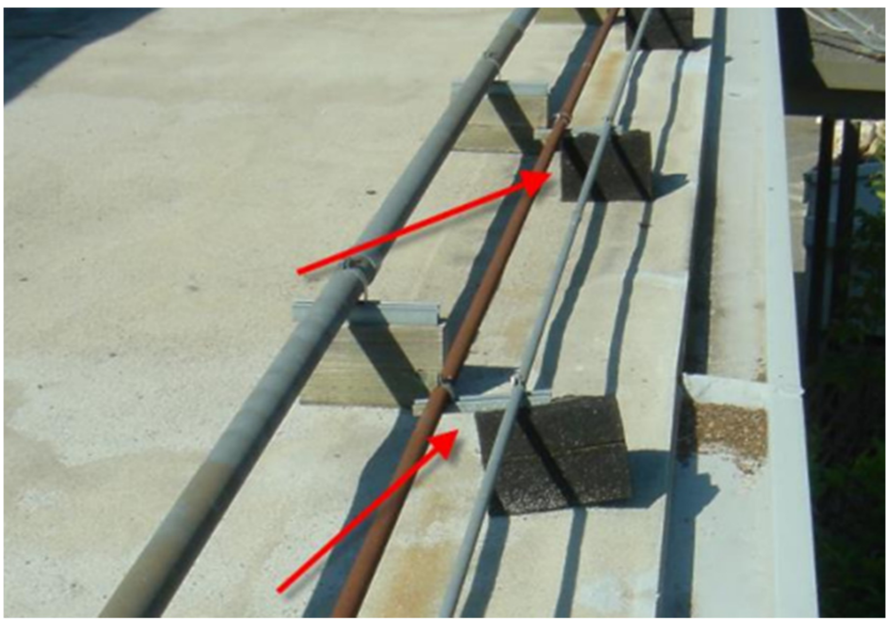


Figure 6-29 Shifted rooftop pipe supports at Silverado Middle School (photo by Quattrocchi Kwok Architects).

6.2.7 Electrical Systems

There was limited damage to electrical systems in schools in the South Napa Earthquake. Electrical damage was limited to loose conduit fittings and broken conduit elbows. Similar to piping, movement of unrestrained rooftop conduit caused damage at connections.

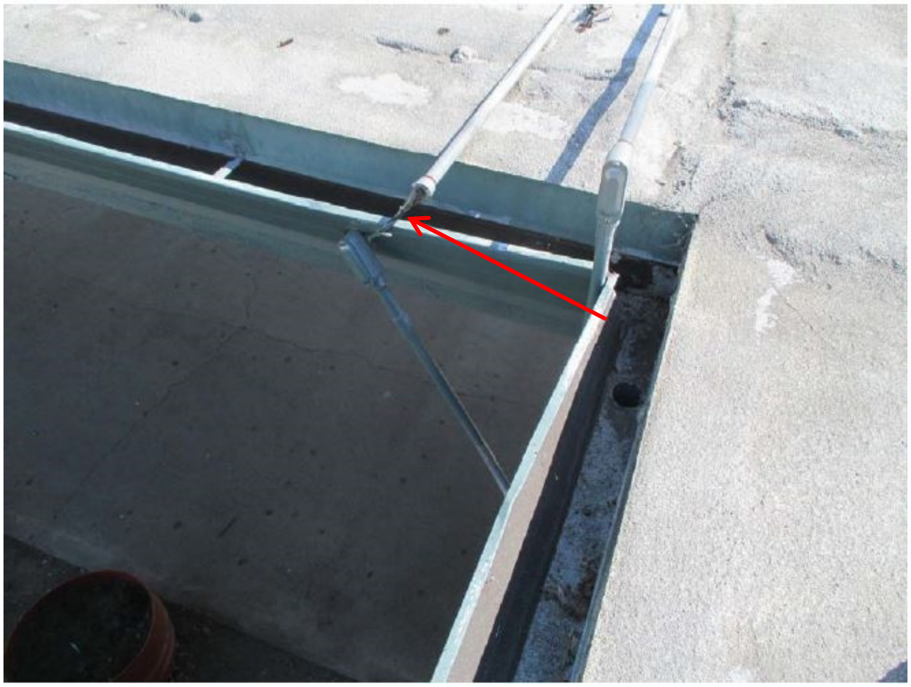


Figure 6-30 Exposed wires at failed conduit fitting, Harvest Middle School (photo by Quattrocchi Kwok Architects).

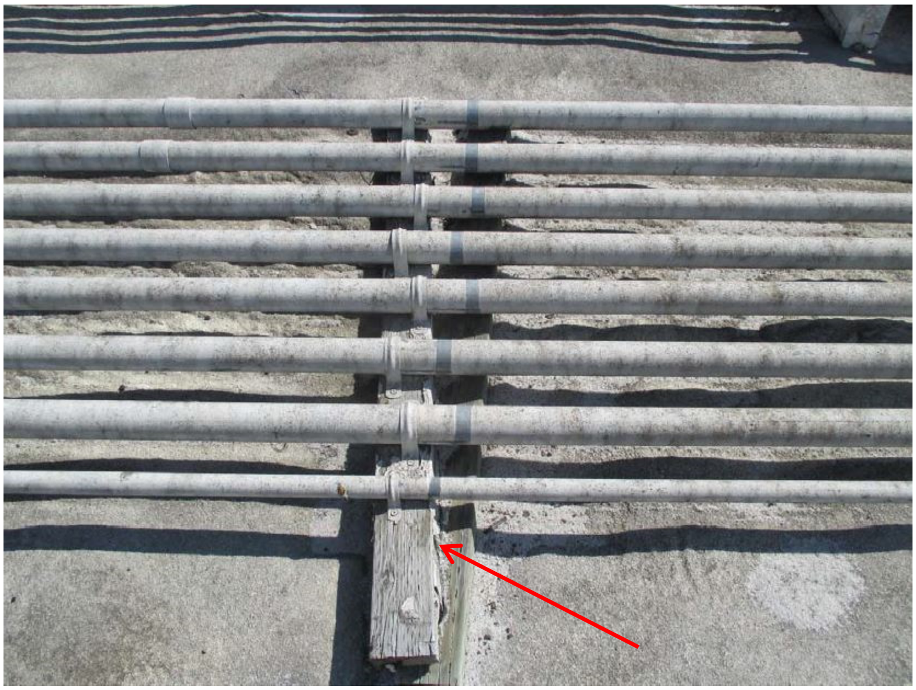


Figure 6-31 Shifted conduit at sleeper support, Harvest Middle School (photo by Quattrocchi Kwok Architects).

Figure 6-32 Shifted conduit at Napa Valley Language Academy (photo by Quattrocchi Kwok Architects).

Figure 6-33 Broken low voltage conduit coupling at Redwood Middle School (photo by Quattrocchi Kwok Architects).

Figure 6-34 Broken PVC conduit at Redwood Middle School (photo by Quattrocchi Kwok Architects).

6.2.8 Concrete and Asphalt Cracking

Several schools reported damage to concrete slabs-on-grade and one school reported damage to the asphalt parking lot, due to ongoing afterslip (see Chapter 2). The slab cracking generally occurred at existing slab construction joints.

Figure 6-35 Cracking at slab joint in walkway at Harvest Middle School (photo by Quattrocchi Kwok Architects).

Figure 6-36 Cracks in asphalt parking lot at Stone Bridge School (photo by Quattrocchi Kwok Architects).

6.2.9 Additional Observations

At several schools, some doors were reported to be hard to open or binding after the earthquake. In some cases, the cause appears to have been movement of the building and in others the cause appears to be related to ground movement. All doors were operable, but with stronger shaking, it is possible that means of egress could have been compromised.

Figure 6-37 Damaged door at Browns Valley Elementary School (photo by Quattrocchi Kwok Architects).

6.3 Other School Facilities within Napa Valley

6.3.1 Justin Siena

Justin Siena, a parochial high school in Napa, 10 miles north-northwest from the epicenter, sustained nearly $60,000 in losses. Damage included broken windows, broken water mains, cracked wall finishes, damaged light fixtures, shifted equipment, toppled bookcases and furniture, loss of supplies, and damaged statuary (Figures 6-38 through 6-40).

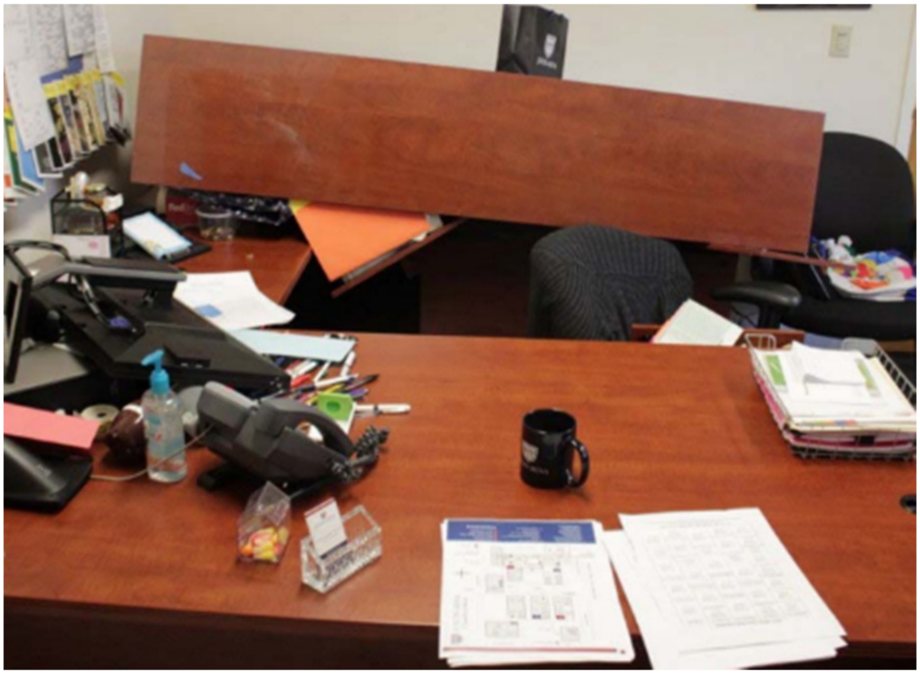


Figure 6-38	Unrestrained bookshelf overturned in office (photo by Quattrocchi Kwok Architects).

Figure 6-39	Toppled altar and statuary (photo by Quattrocchi Kwok Architects).

Figure 6-40 Toppled landscape statuary (photo by Quattrocchi Kwok Architects).

6.3.2 Napa Education Center and District Auditorium, Napa

This building was constructed in 1922 as the Napa High School prior to the *Field Act*, and served that purpose until 1976, when the high school was moved to a new facility and the building was converted to administrative uses. The auditorium continues to be used for local school productions. The building is one-story in height, with a two-story portion and was originally constructed with lightly reinforced concrete walls, and wood floor and roof systems.

The building sustained significant damage in the 2000 Yountville earthquake, including near collapse of the auditorium stage walls and roof. The building was repaired, completely renovated, and seismically retrofitted using $3 million in federal repair funds, a $10 million local bond and $3 million in operating funds. DSA approved plans for the project in 2003 and construction was certified as complete. The seismic retrofit included installation of a tube steel frame on both sides of the central hallway and along the exterior walls. The frames were connected by steel cross-members, and wood-frame connections were improved. The 80-foot high auditorium stage walls were taken down to the level of a damaged construction joint at 40 feet and new reinforced concrete walls were installed above, with the reinforcing steel doweled into the existing walls. A new steel roof was installed over the stage. The auditorium side walls, which were also damaged in the 2000 earthquake, were wrapped in carbon fiber mesh and stucco was reapplied and painted to restore the building's appearance (Figure 6-41).

Figure 6-41 Interior view of auditorium walls.

In the 2014 earthquake, the only damage was minor wall cracking around a beam added as part of the retrofit (Figure 6-42). One exterior fire sprinkler head under the front portico that broke and discharged on the outside of the building.

Figure 6-42 Minor wall cracking.

6.4 Vallejo City Unified School District

There are 21 schools in the Vallejo City Unified School District, but no damage was reported at these facilities.

6.5 Summary

Schools generally performed well structurally. DSA's enforcement of the *Field Act* has been largely responsible for the structural seismic safety of California's public schools. Since 1940, no building constructed under the *Field Act* has either partially or completely collapsed, and no students have been killed or injured in a *Field Act* compliant building. However, as the 2014 South Napa earthquake has highlighted, failures of nonstructural components were widespread in schools, and the lack of deaths or injuries caused by earthquake may be more related to the time of day of the earthquake, than to the seismic safety of the nonstructural components in schools.

Nonstructural components in schools that were particularly vulnerable to damage in the South Napa earthquake include overhead lighting, particularly pendant fixtures, and classroom furnishings. Despite the fact that damage to pendant light fixtures in schools has been documented in virtually every damaging earthquake since 1952, even recently installed pendant fixtures were damaged in many schools. Eleven of the 31 schools in Napa reported damage to pendant fixtures, which often fell across student desks and tables.

Classroom furnishings, which are generally not regulated by code, also pose a threat to students and staff. Unrestrained bookcases, storage units, file cabinets, lockers, and similar components overturned in the earthquake, often striking tables and desks. Had school been in session during the earthquake, these items would have posed a serious risk to students and staff. In some cases, overturned furnishings blocked paths of egress. At least three public schools experienced potentially life threatening damage to classroom furnishings.

Unrestrained equipment and piping were also damaged in the earthquake, but do not pose a direct threat to student safety since these components were generally not in areas accessible to the public.

6.6 Recommendations

The performance of school buildings and nonstructural components in the South Napa earthquake suggests several areas for possible improvement and further study, including:

1. There is a need to develop reliable means of installing pendant lighting fixtures that will remain in place and not pose a falling hazard during an earthquake. Many of the newer installations of these fixtures experienced failures, suggesting that the current methods of design are inadequate. Shake table testing is recommended to demonstrate seismic reliability.
2. Large, heavy furniture items are constantly being installed in and moved around classrooms, and most are outside the scope of the building code. Proper anchorage and bracing of these items are critical to occupant safety. Classrooms should be examined on a regular basis to ensure that furnishings and contents do not pose a serious seismic threat. An annual inspection of nonstructural components is recommended at the beginning of each school year. The inspection, or the mitigation measures identified as a result of such inspection, could be linked to the annual ShakeOut earthquake safety awareness program (www.ShakeOut.org) that takes place every October, since most schools now participate in this program. These nonstructural safety inspections could involve the students, staff, and faculty.
3. The consequences of building drift on the functionality of doors should be studied. Although all exit doors operated following the South Napa earthquake, binding of some doors suggest that in a stronger earthquake, some egress routes could be compromised as a result of building movement.
4. Guidelines for the installation of rooftop piping and conduit should be developed. The building code does not explicitly address this issue and simple construction measures could avoid costly earthquake repairs.
5. Nonstructural damage in both public and non-public schools was potentially life threatening. All schools, not only public schools, should implement measures to enhance seismic safety.

Chapter 7
Performance of Residential Construction

7.1 Introduction

Homes make up one of the most important components of a community's building stock. During the South Napa Earthquake, which struck at 3:20 in the morning, most residents were asleep in their homes.

The downtown Napa study area, as defined by the 1,000 foot radius around Station N016, does not include any residential construction. In order to understand the performance of this important building type, additional neighborhoods were visited on September 18, 2014, approximately one month after the earthquake. In particular, homes in the Browns Valley neighborhood, located to the west of downtown Napa, were included based on their proximity to the fault rupture (Figure 7-1). Other homes were selected for investigation based on postearthquake safety evaluation data that indicated significant failures.

The homes investigated were generally one- and two-story, single-family homes or duplexes and were constructed between 1900 and 1965. All homes investigated were wood-framed (designated as W1 on survey forms). The performance of manufactured homes is covered in Chapter 8 of this report. Larger apartment buildings were not studied.

In lieu of the more rigorous procedure used to survey each of the buildings in the downtown area, residential buildings were examined with a combination of drive-by surveys, exterior inspections, and when possible, exterior and interior inspections. Information about building performance, as well as the earthquake and postearthquake experiences of the occupants, were obtained in interviews with occupants and construction crews.

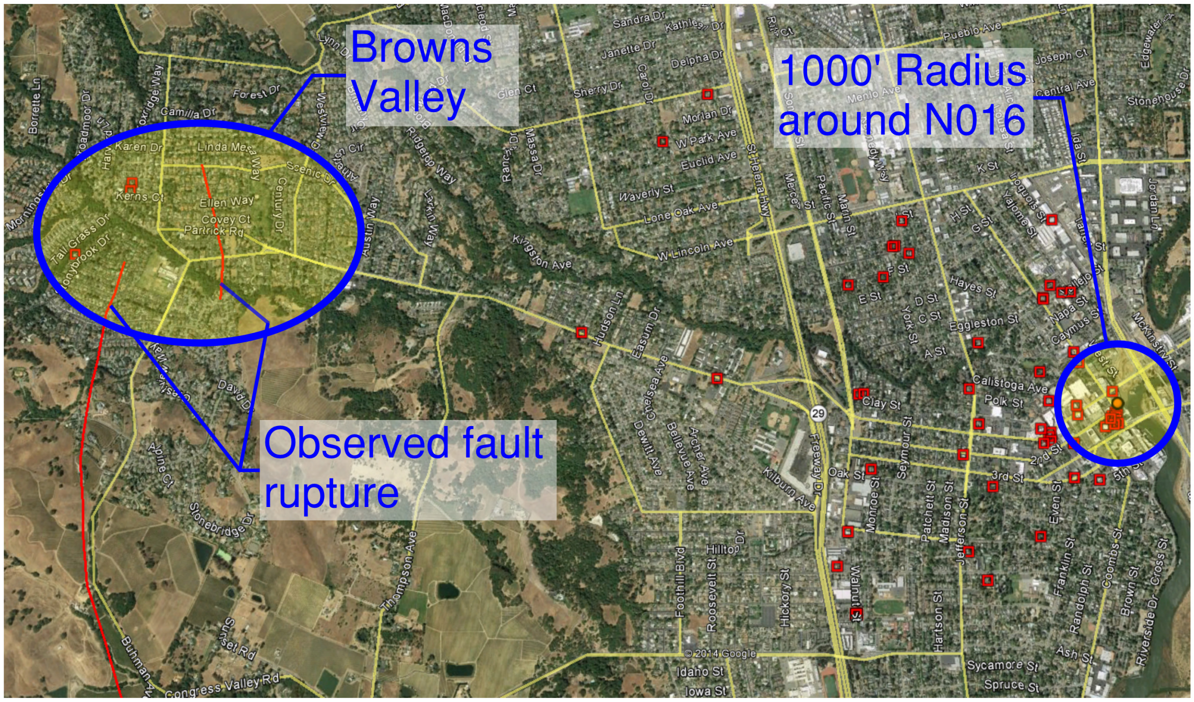


Figure 7-1 Location of Browns Valley neighborhood relative to downtown Napa.

7.2 Selected Observations

The following selected observations summarize performance of residential structures.

7.2.1 Browns Valley House No. 1

This one-story, single-family residence is located in the Browns Valley neighborhood, and was built circa 1960 on a slab-on-grade. The building was posted UNSAFE at the time of the field investigation. Accordingly, the inspection was limited to exterior only. This house was co-located with the surface feature of the fault rupture. The rupture can be followed from the asphalt on the street (Figure 7-2), to the driveway leading up to the garage (Figure 7-3), and around the garage (Figure 7-4). The crack running up the middle of the driveway is presumed to continue through the house. A fallen brick chimney (Figure 7-5) was also observed. This house is located immediately to the south of Browns Valley House No. 2.

Figure 7-2 Surface feature of fault rupture observed on street.

Figure 7-3 Crack running through the street and up the driveway.

Figure 7-4 Crack continuing around the base of structure.

Figure 7-5 Brick chimney collapse on to the roof.

7.2.2 Browns Valley House No. 2

This one-story, single-family residence located in the Browns Valley neighborhood was built circa 1960 and is located directly north of Browns Valley House No. 1. The residence was posted UNSAFE following the earthquake, but the homeowner hired an engineer to inspect the home, and the building was reposted as RESTRICTED USE five days after the earthquake. At the time of the site visit, the homeowner was conducting repair work himself with help of family members and friends.

Owners were available on site, and both exterior and interior inspections were performed. It was reported that the brick chimney collapsed, and was demolished prior to the site visit. Cracks running from the south end of the property (Figure 7-6), i.e., adjoining to Browns Valley House No. 1, through the house (Figure 7-7) to the front of the house (Figure 7-8) and street (Figure 7-9) were observed. Inside the house, large cracks (Figure 7-7) and buckled floors were observed. The water heater, which had been seismically braced, was undamaged. Rooftop solar panels were also undamaged.

Figure 7-6 Cracks in the pavement of the backyard and damage to stucco wall.

Figure 7-7 Ceiling crack.

Figure 7-8 Damage caused due to differential movement of floor slabs in garage and driveway pavement.

Figure 7-9 Surface rupture observed on street.

7.2.3 Browns Valley House No. 3

This L-shaped, one-story, single-family residence was built circa 1965. An exterior inspection was performed. The residence was posted UNSAFE following the earthquake. The brick chimney collapsed, and was demolished prior to the site visit. Fault rupture caused the building to twist. Damage was concentrated at the reentrant corner by the main door, buckling the floors. At the time of the site visit, the building was being professionally repaired, and had been lifted off the concrete perimeter wall (Figures 7-10 and 7-11). The contractor was interviewed on site and stated that the plan was to completely remove the existing perimeter foundation and interior piers and replace with a similar foundation system but one using better concrete with deeper embedment lengths. This retrofit is considered far superior to other "patch and paint" foundation repairs observed at other sites.

Figure 7-10 Photo showing demolished perimeter foundation and cracks in pavement.

Figure 7-11 Structure raised for foundation repair.

7.2.4 Napa House No. 1

This one-story, duplex residence was built circa 1910. An exterior inspection was performed. The residence was posted UNSAFE following the earthquake. The brick chimney collapsed. The cripple wall supporting the building, which was approximately 5 feet tall, collapsed (Figure 7-12), causing the house to fall into the adjoining property (Figure 7-13). As a result of the cripple wall failure, the side porch roof collapsed (Figure 7-14).

Figure 7-12 Home with collapsed cripple wall.

Figure 7-13 Building fell into adjoining property.

Figure 7-14 Failure of porch due to cripple wall collapse .

7.2.5 Napa House No. 2

This one-story, single-family home was built circa 1910. An exterior inspection was performed. The residence was posted UNSAFE following the failure of the approximately five foot tall cripple wall (Figure 7-15). The house shifted laterally but did not collapse (Figure 7-16).

Figure 7-15 Temporary lateral bracing at cripple wall.

Figure 7-16 House shifted lateral at damaged cripple wall.

7.2.6 Napa House No. 3

This one-story, duplex residence was built circa 1925. An exterior inspection was performed. The residence was posted UNSAFE following the earthquake after the 3 foot tall cripple wall supporting the building collapsed (Figure 7-17). As a result of the cripple wall failure, the front porch roof collapsed (Figure 7-18).

Figure 7-17 Collapsed cripple wall.

Figure 7-18 Failure of porch due to cripple wall collapse.

7.2.7 Napa House No. 4

This one-story, single-family residence was built circa 1900 (white building on the right of photo in Figure 7-19). An exterior investigation was performed. The house was initially posted INSPECTED following the earthquake, but was subsequently posted UNSAFE due to the imminent hazard posed by the adjacent residence, which had suffered a cripple wall failure and severe racking.

Figure 7-19 Undamaged residence on the right was posted UNSAFE due to risk posed by damaged residence on the left.

7.3 Residential Flood Retrofits

A significant portion of the City of Napa lies in the regulatory floodplain of the Napa River and its tributaries. Much of the older residential construction took place before the floodplain was mapped under the National Flood Insurance Program (NFIP), which was established by Congress in 1968 to make flood insurance available within participating communities in exchange for the adoption of floodplain management ordinances. The Program is managed by FEMA and is intended to reduce future flood risk by requiring new construction and substantially damaged or improved existing construction to be elevated above the Base Flood Elevation (BFE). FEMA also operates several mitigation grant programs that can include covering the cost of elevating existing buildings above the BFE. Approximately 100 homes in Napa have been elevated above the BFE under these programs. Following the South Napa earthquake, FEMA conducted a survey of these homes and determined that all but one had been inspected by the City and were posted as INSPECTED. Most of the homes appeared to have been elevated on properly braced cripple walls. One home had been posted as RESTRICTED USE due to a damaged masonry chimney that was not part of the elevation work. Several duplex homes inspected had been elevated a full-story height (Figure 7-20) on properly braced wood cripple walls (Figure 7-21) and were found to have sustained no appreciable damage.

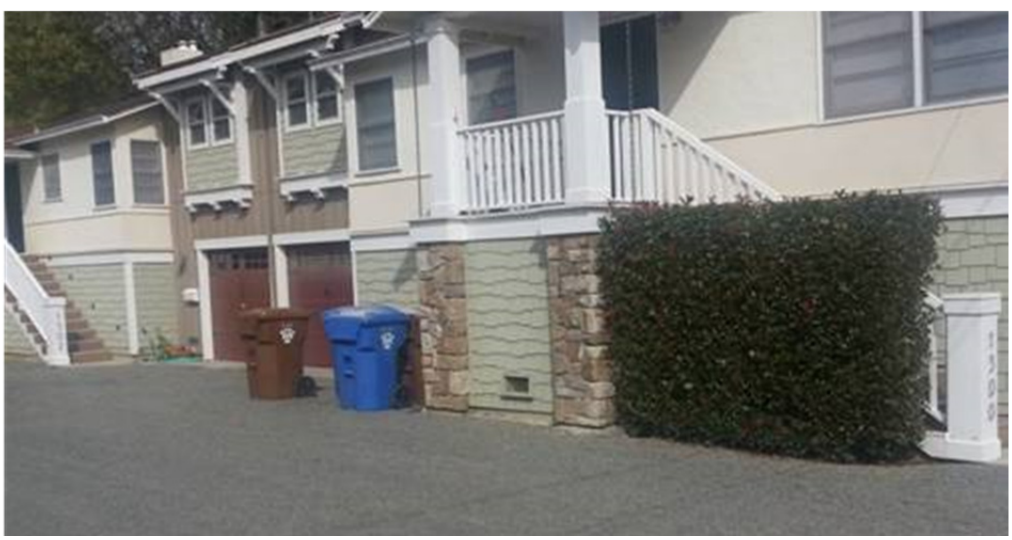


Figure 7-20 Elevated duplex homes.

Figure 7-21 Braced wood cripple walls on the first story of elevated duplex homes. The height of the electrical outlets indicated the Base Flood Elevation.

7.4 Summary

The overwhelming majority of residences affected by the South Napa earthquake suffered little damage. The good overall performance of single-family residences is in keeping with the trends noted in other areas subject to moderate earthquakes; light-frame residential construction generally performs well, provided that known hazardous conditions are either not present or mitigated.

Damage to residential construction was concentrated heavily on two key deficiencies: unbraced chimneys and unbraced cripple walls. The following were also noted on damage descriptions in the database: fire, damaged gas lines, façade damage, and carport collapses.

7.4.1 Masonry Chimneys

Unreinforced masonry chimneys performed poorly with many observed failures in the form of significant damage (Figure 7-22) or collapse (Figures 7-23 and 7-24). In many cases, chimneys fell out onto exterior areas, endangering adjacent homes, passersby on sidewalks, and parked cars. In other cases, chimneys fell into homes, endangering occupants. In one instance, a teenage boy who was asleep in the living room, was struck and injured by falling bricks from a collapsed fireplace (Figure 7-25). This was one of the most serious injuries caused by the earthquake. It was reported that some reinforced masonry chimneys also failed, likely due to insufficient reinforcement. Lightweight metal flue chimneys performed much better and caused no injuries. The City of Napa had issued a construction information handout *Retro Fitting Masonry Fireplace with Factory Built Metal Chimney* (City of Napa, 2000), but the guidance was not readily implemented. In response to the damage sustained to masonry chimneys in the 2014 South Napa earthquake, FEMA funded the development of a Recovery Advisory on *Repair of Earthquake-Damaged Masonry Fireplace Chimneys*. This Recovery Advisory is provided in its entirety in Appendix B.

Figure 7-22 Dislodged unreinforced masonry chimney (photo from Janiele Maffei).

Figure 7-23 Photo of damaged chimney (photo from Janiele Maffei)

Figure 7-24 Photo of chimney laying on front yard of home (photo from Janiele Maffei).

Figure 7-25 Photo of interior chimney collapse (photo from Janiele Maffei).

7.4.2 Unbraced Cripple Wall Foundations

Due to flood risk, many older homes in Napa have tall cripple walls, sometimes as much as one-story tall. In the earthquake, many homes with tall, unbraced cripple walls sustained large lateral displacements or collapse of the crawl space. Napa House No. 1 fell almost 5 feet and moved laterally an equivalent amount. Browns Valley House No. 2 was constructed with much shorter, although also unbraced, cripple walls and sustained lateral movement of a few inches. Some cripple wall retrofit successes, in which tall, braced cripple walls performed well were also reported, including homes that had been elevated under FEMA's mitigation grant programs to address flood risk.

The earthquake highlighted the possibility that residents could find their homes unavailable after an earthquake due to damage in adjacent buildings. Napa House No. 4 performed well and was initially posted INSPECTED, allowing the residents to return home. However, its proximity to the poorly performing home next door resulted in Napa House No. 4 being posted UNSAFE, forcing the residents out.

The 2012 edition of the *International Existing Building Code* (ICC, 2012b) presents retrofit provisions for cripple walls shorter than four feet; however, no guidance is provided for taller cripple walls. In response to the damage sustained to tall cripple walls in the 2014 South Napa earthquake, FEMA funded the development of a Recovery Advisory on *Earthquake Strengthening of Cripple Walls in Wood-Frame Dwellings.* A summary of the Recovery Advisory is provided in Appendix B.

7.4.3 Fault Rupture and Afterslip

The damage to Browns Valley House No. 2 highlights the risks of fault rupture. The home, which did not have an unbraced cripple wall, would have been expected to perform well in an earthquake. However, it was not able to withstand the fault rupture that traced directly through the house footprint, and damage (although not collapse) was experienced and occupants had to evacuate. The issue of afterslip is covered in Chapter 2; its greatest impact was to residential structures in the Browns Valley neighborhood.

7.5 Recommendations

Several areas for further study and possible improvement include:

1. Outreach to the community is needed to encourage mitigation of two significant hazards in residential construction: unreinforced masonry chimneys and tall unbraced cripple walls. FEMA recently contracted to develop two Recovery Advisories: The first recommending best practices for reconstruction of earthquake-damaged masonry chimneys using light-weight metal flue chimneys, and the second recommending best practices for seismically retrofitting cripple wall foundations in one- and two-family dwellings to minimize risk of damage in future earthquakes. The entirety of FEMA DR-4193-RA1, *Repair of Earthquake-Damaged Masonry Fireplace Chimneys*, is provided in Appendix B. A summary of the Recovery Advisory on cripple walls is also provided in Appendix B.
2. The most effective measure for reducing the hazards posed by fault rupture is the implementation of a land use regulation that limits building in areas at risk for surface faulting. However, since this event took place in an already established neighborhood, using foundation designs that are more capable of accommodating ground movement may be useful in protecting housing in some locations. As detailed in Chapter 2, for areas where afterslip potential is high enough that it could impact a conventional foundation, the best way to mitigate this hazard would be to replace the existing foundation with a reinforced concrete mat or raft slab

foundation, which has enough reinforcing steel and high-strength concrete to support itself even if the ground is still slipping beneath. Also, measures to protect gas lines and other vulnerable piping from damage caused by ground movement should be developed. However, given that most foundation repairs have already been implemented and the high cost of replacing a building foundation with a new system, this mitigation measure is likely not cost effective for existing buildings. It should, however, be considered for any new construction within the moderate hazard area.

Chapter 8
Performance of Manufactured Housing

8.1 Introduction

In the August 24, 2014 South Napa earthquake, significant damage was observed to manufactured homes (also called mobile homes) located in northwest Napa, specifically due to fire following the earthquake. This chapter provides an overview of manufactured home performance.

Three visits were made to survey Napa mobile home parks (MHPs) between September 28 and October 18, 2014, visiting a total of eleven MHPs. Observations of home support systems were limited to homes where surrounding skirts had been fully or partially removed, either for inspection (by others) or because reinstallation or re-leveling was required; as a result, observations primarily focus on homes that experienced damage. Because these visits occurred several weeks after the August 24, 2014 earthquake, it is expected that some conditions had changed by the time of the visits. In addition to information gathered from the project team's field investigations, State of California Department of Housing and Community Development (HCD) provided data summarizing its inspections following the earthquake.

8.2 Manufactured Homes

Manufactured homes are built in factories using light-frame construction on top of integral steel chassis beams, such that they can be transported to the home site and installed. The homes are typically built in standard floor sections with common widths of 12 and 14 feet and lengths between 40 and 60 feet. Each floor section is supported by two longitudinal chassis beams that form the primary floor structure; when installed on a home site, support is provided along these chassis beams. Homes typically include either one single unit (single-wide home) or two units, fabricated to be joined together to form an approximately 24-foot wide home (double-wide home). In addition, there are two primary categories of homes: (1) older homes primarily clad in aluminum and with aluminum panel skirts (Figure 8-1); and (2) newer homes with cement board (fiber-cement) or wood panel siding with similarly clad skirts (Figure 8-2). The skirts in the older type homes slip into continuous tracks and the siding skirts in newer homes are fastened to continuous wood plates sitting on the ground around the home perimeter. The newer home type is believed to have become common in homes manufactured in the 1980s and later. Both older and newer homes occur in both single-wide and double-wide configurations.

Figure 8-1 Typical older (single-wide) home with add-on porch and carport. Home is clad in aluminum siding and aluminum skirting.

Figure 8-2 Typical newer (double-wide) home clad in wood panel siding with a matching siding skirt.

When installed on the home site, the floor of the home generally sits between two and three feet above surrounding grade. Installation of the manufactured home on the home site involves a gravity support system and may also involve systems for wind or seismic resistance, further discussed in Section 8.3. Manufactured homes are commonly surrounded by extensive prefabricated or site-built porches, carports, and other add-on structures, as seen in Figures 8-1 and 8-2. Porches are often self-supporting while carports are often partially supported by the home.

Manufactured homes have experienced damage in past earthquakes (EERI, 1996; EERI, 2005; State of California, 1992). The damage to manufactured homes in this and past earthquakes has almost exclusively been associated with the support system, rather than the homes themselves.

8.3 Regulatory Background

Building code regulations for manufactured homes differ from site-built homes. New manufactured homes currently fall under the jurisdiction of the U.S. Department of Housing and Urban Development (HUD). Current regulations for home installation in the state of California fall under the code adoption authority of the State of California Department of Housing and Community Development (HCD). Enforcement of installation requirements is permitted to be under the jurisdiction of the local building department (city or county); where the local building department chooses not to take jurisdiction, enforcement authority remains with HCD. As a result, some of the MHPs in Napa fell under HCD jurisdiction and were included in their postearthquake inspections, while others did not.

The regulations for construction of manufactured homes have evolved over the years:

- Prior to September 1, 1958, there were no construction standards for manufactured housing.
- Between 1958 and 1971, standards were applied to electrical, mechanical, and plumbing systems only.
- Between 1971 and 1976, California construction standards were applied to construction of homes.
- Starting June 1976, HUD standards became applicable to the homes.

Similarly, oversight of home installation has evolved over the years, with permits for and inspections of home installations starting in 1974.

Since September 1994, the *California Code of Regulations* (CCR) (State of California, 1994) has required that a wind tiedown system be installed on newly installed or relocated manufactured homes in the state of California. This requirement comes from California SB 750, passed in July 1994 following the Northridge earthquake as a urgency statute "…to ensure that as many manufactured homes as possible are protected at the earliest possible time from sudden devastation by earthquakes." The tiedown system can either consist of a tiedown assembly chosen from a list approved by HCD, or an engineered system. Either system is required to resist a wind load of 15 pounds per square foot (or the wind pressure specified on the home label, where higher), applied to the projected area of the home in each horizontal direction. These requirements are found in *California Code of Regulations*, Title 25, Sections 1320 and 1336, Chapter 2, Article 7. The term "ETS" is often applied to tiedown systems, whether engineered or pre-approved. Homes installed prior to September 1994 were not required to have a tiedown or ETS at the time of installation (with the exception of limited high-wind locations for which wind bracing has been regulated since 1974); these homes are not required to meet this requirement retroactively, except under two circumstances: (1) the home is relocated to a new lot; and (2) the home requires reinstallation due to damage caused by wind or seismic forces, and federal funds are available for the additional installation costs (Section 18613.4(e) of *California Health and Safety Code* (State of California, 2014)). Although relocation of homes does sometime occur, it is uncommon.

California Code of Regulations Title 25, Chapter 2, Article 7.5 contains provisions for earthquake-resistant bracing systems (ERBS). These provisions do not require that ERBSs be installed, but do provide minimum requirements for them where they are installed: it is required that they be designed in accordance with the 1982 *Uniform Building Code* (ICBO, 1982), and it is required that they limit downward vertical movement of the home to not more than two inches. Although ERBSs existed prior to the early 1980s, approval of ERBSs by HCD started in January 1981, and inspections of ERBS installations started in 1990. ERBSs are most often installed as a voluntary retrofit measure, and almost exclusively to homes installed prior to September 1994 that do not have tiedown systems. A modest number of ERBSs were observed to be installed in the surveyed MHPs.

It is understood that an ETS, though designed for wind, will provide some (undetermined) level of earthquake resistance; as previously discussed, SB 750 clearly intended to provide increased earthquake performance. Similarly, an ERBS, designed considering earthquake resistance, will provide some (also undetermined) level of wind resistance.

Because the requirement for tiedown systems (ETS) has only been in effect since 1994 and because only a limited number of ERBSs have been voluntarily installed, there remains a significant population of existing manufactured homes in California that do not have ETS or ERBS. Further, because the regulations for ETS and ERBS have evolved over the years, the systems observed following the South Napa earthquake may or may not have been approved and inspected as would currently be required. Their performance in the earthquake should be viewed with these limitations in mind.

8.4 Gravity and Seismic Support Systems

Gravity support of manufactured homes is primarily provided by two lines of support piers for each floor section, one along each longitudinal chassis beam, located a few feet inboard of the floor section edge. This results in two lines of support for single-wide homes and four lines of support for double-wide homes. The gravity support piers observed in the MHPs visited were almost exclusively stacked concrete masonry units (Figure 8-3). This is notably different from homes observed in the 2003 San Simeon earthquake, where steel gravity piers were prevalent (EERI, 2005). The masonry units are hollow nominal 8”x8”x16”, dry-stacked, and often interspersed with 2x wood blocks. The masonry piers sit on either wood or ABS (plastic) mats supported on the ground surface (on soil or on a concrete pad). Wood wedges are provided at the top of the stack in opposing pairs, used to adjust the support height when the floor is leveled; there were no nails or other fasteners observed connecting the wedges to the supporting wood block. The long (16 inch) masonry block dimension is generally oriented in the transverse direction of the home to allow for the wood wedges to engage the chassis beam. The spacing of gravity piers installed prior to the earthquake appeared to range between eight and ten feet on-center along each chassis beam; the spacing for reinstallations observed following the earthquake was typically reduced to six feet on center. Supplemental piers were observed to occasionally be provided at the perimeter of the homes near doors, and at the unit interface in double-wide homes. Only a small percentage of the observed homes had been supported on steel (Figure 8-4) or precast concrete gravity piers. It appeared that many of the homes that had precast concrete gravity piers prior to the earthquake were in the process of replacing them with dry-stacked masonry piers. Similar to the masonry piers, the steel and precast concrete piers, when used, were also commonly seated on wood or ABS mats sitting on the ground surface.

Figure 8-3 Typical dry-stacked masonry gravity piers, with interspersed wood blocks.

Figure 8-4 Steel gravity piers interspersed with masonry piers during home reinstallation.

Where earthquake and wind bracing is provided for manufactured homes, the two predominant systems are wind tiedowns (ETS) and ERBS. Although perimeter concrete or masonry foundations of the type used for site-built homes are permitted to be used (supplemented with gravity support along the chassis), none were observed in the surveyed Napa manufactured homes. Early ETS used a set of steel straps looped over the chassis beams and attached to helical soil anchors. Figures 8-5 and 8-6 show such a system, with the strap cut to allow relocation of the home. HCD representatives indicated that this type of ETS is seldom used anymore, replaced by other approved systems. Straps were only seen in two of the homes where support systems were observed.

Figure 8-5 ETS strap connected to a home chassis beam, cut to allow home to be relocated.

Figure 8-6 ETS ground anchor. The ground anchor bracket is typically attached to a helical soil anchor, embedded in the soil.

It was observed that one home had in place a steel support pier with rods anchoring the pier to the ground (similar to the one shown in Figure 8-7). This system was identified by HCD as an ETS. The approximately 1/2-inch steel anchor rods at each corner of the pier base are identified in manufacturer online literature to extend about 15 inches into the ground. The observed home did not appear to have moved, and was reported by the residents to have performed well. This same system was observed in several postearthquake reinstallations.

Figure 8-7 Steel pier system with anchor rods to the ground.

Several homes identified by HCD to have ETS at the time of the earthquake were observed to have "cantilevered" (individual) steel piers in addition to the masonry gravity piers. Each of these homes appeared to have between four and eight of these steel piers installed. Figure 8-8 shows these piers, removed during reinstallation of homes that had moved considerably during the earthquake.

Many of these "cantilevered" piers did not have holes in the pier base that would allow installation of fasteners between the pier base and the supporting wood board, nor was the wood board anchored to or embedded in the supporting soil, thus some of these "cantilevered" steel piers appeared to have slid or toppled over, allowing the home to drop. Similar "cantilevered" piers being installed new in postearthquake reinstallations were observed to have screw connections between the pier base and the supporting wood boards (Figure 8-9), but continued to lack connection of the wood board to the supporting soils.

Figure 8-8 ETS removed from homes during reinstallation. These "cantilevered" piers had no allowance for anchorage to the supporting wood boards.

Figure 8-9 New "cantilevered" pier, anchored to supporting plywood panels.

In another MHP, homes with these "cantilevered" steel piers were not identified by HCD to have either ETS or ERBS HCD noted that the devices observed following the earthquake may or may not have been designed or approved as an ETS or ERBS; further, many systems utilize the same supports in different numbers and configurations making the exact determination of system (ETS, ERBS, or other) extremely difficult to an untrained observer. Figure 8-10 shows another type of "cantilevered" pier, also without anchorage to the ground, which rocked and toppled over, allowing the home to drop.

Figure 8-10 "Cantilevered" steel pier on home requiring reinstallation.

A number of homes were found to have identifiable ERBS at the time of the earthquake. These are designated as "diagonally braced" ERBS in this report, because they often had diagonal bracing members that engage more than one chassis beam. Dates of installation of ERBS were in some cases identified by residents to be the early 1980s, while others were more recently installed. Figure 8-11 illustrates some of these systems. These systems were typically seated on wood plates sitting on the ground, similar to the gravity piers and "cantilevered" pier systems. Unlike the "cantilevered" steel piers that were vulnerable to rolling over, these "diagonally braced" ERBS were observed to stay upright while the homes slid horizontally from a few inches to more than a foot. The "diagonally braced" ERBS appeared to be much more inherently stable under earthquake loading than the "cantilevered" piers.

Figure 8-11 “Diagonally braced” ERBS on homes requiring reinstallation.

As previously mentioned, damage to manufactured homes was observed to be primarily to the home’s support system rather than the home itself. Very few homes were observed to have suffered extensive damage such that they could not be reinstalled. The following damage and resulting consequences were observed:

- Homes sustained damage to the skirts from rocking and shifting (Figure 8-12). This predominantly affected older homes with aluminum skirts, but occasionally affected wood or cement board siding skirts also. This damage requires repair, but is not structural and does not affect continued occupancy of the home.
- Homes sustained disruption to support gravity piers without significant sliding or toppling movement. This was identified by HCD as requiring reinstallation, even when the damage was to a few piers and the scope of repair required was small.
- Homes toppled off of support piers, as shown in Figures 8-13 and 8-14. Most often the home itself was not damaged; however, occasionally steel support piers punched through the floor as the home dropped. When the home falls off the supports, the outward swinging doors are often blocked, as occurred in the home shown in Figure 8-15. This damage can impede safe egress of occupants, requires reinstallation of the home, including lifting and placement back onto supports, and repair or replacement of damaged components. In addition, reinstallation of any

disrupted utility hook-ups is also required, and minor to major damage to surrounding add-on structures, such as decks and carports, is common. Figure 8-16 shows an example of a large extent of damage to an attached porch. Homes that drop are most often posted RESTRICTED USE so residents can retrieve belongings, but not continue to occupy the home.

Figure 8-12 Damaged skirt on older home.

Figure 8-13 Home dropped from masonry gravity pier supports.

Figure 8-14 Close-up of toppled masonry gravity pier.

Figure 8-15 This door was blocked when the home dropped off of the gravity supports and had to be broken open by fire fighters.

Figure 8-16 Add-on porch structure severely damaged by the earthquake.

- Homes sustained minor to significant sliding. Homes that slid sometimes retained enough vertical support, primarily through a "diagonally braced" ERBS, so that the floors did not drop. Similar to homes that toppled off of their supports, these homes require reinstallation, repair of hookups, and repair of attached structures. These homes were observed to often be posted INSPECTED, allowing continued occupancy, even when the ERBS was the only vertical support remaining.
- In a few cases, fires occurred following the earthquake. Common causes in past earthquakes have included damage to gas lines or poorly braced water heaters. Although most of the homes were observed to have flexible utility hook-ups allowing the home to move relative to the hook-up without causing fire, if the home moves far enough, utility lines can still potentially be ruptured.
- In the few instances where damage to the home superstructure was observed, it included cracking of wall finishes around windows, as commonly occurs with site-built structures.

8.5 Overall Mobile Home Park Observations

In the eleven parks surveyed, the majority of the homes were installed prior to ETS being required. Bracing systems (ERBS or ETS) were observed to have been installed on a limited number of the homes. This is likely due to those homes being more recently installed or due to owners choosing to voluntarily install bracing.

Five MHPs visited in mid-town Napa (Napa Valley Manor, Rexford Mobile Home Estates, Valley Mobile Home Park, Pueblo Trailer Park, and Miller's Senior Park) included a total of approximately 345 homes. Of these, only one home was readily observed to require reinstallation. The owner reported that, although the home did not fall off of its supports, it did shift several inches. The homes in these MHPs were not observed to have readily apparent differences in home type, age, or construction from the other parks that had much higher incidents of damage. This relatively small amount of damage may be attributed in part to these homes being located at a greater distance from the fault rupture.

Six MHPs visited in northwest of Napa are shown in Figure 8-17. Four of the MHPs are located adjacent to each other, arranged in an approximately north-south string along Highway 29, with the first being approximately one block north of Fire Station No. 3 (12.3 km north of the epicenter and the location of a ground motion recording station). A fifth park is in the same north-south line, but spaced a block further north of the first four. A sixth park is located to the east across Highway 29.

Significantly more damage was observed at the six MHPs northwest of Napa than the MHPs in mid-town in spite of the fact that the mix of ages and types of homes was judged to be similar among the two groups of MHPs. While the ground motion records at Fire Station No. 3 show lower peak ground acceleration (PGA) than the records at Station N016 at Main Street, the peak ground displacement reported at Fire Station No. 3 is more than twice that reported at Station N016 (see Figures 2-6 and 2-7 in Chapter 2). Differences in the ground motions are likely to have contributed to the difference in performance. Spectral acceleration plots based on the two ground motion records are also distinctly different; however, the spectral accelerations are difficult to relate to manufactured homes because there is no single characteristic period of vibration during shaking once the home slides or rocks.

A summary of homes in MHP 1 through 5 requiring reinstallation is provided in Table 8-1, based on field inspections by HCD. MHP 6 is not under the jurisdiction of HCD, so data were not available. The data in the table must be qualified considering first that a home designated by HCD as requiring

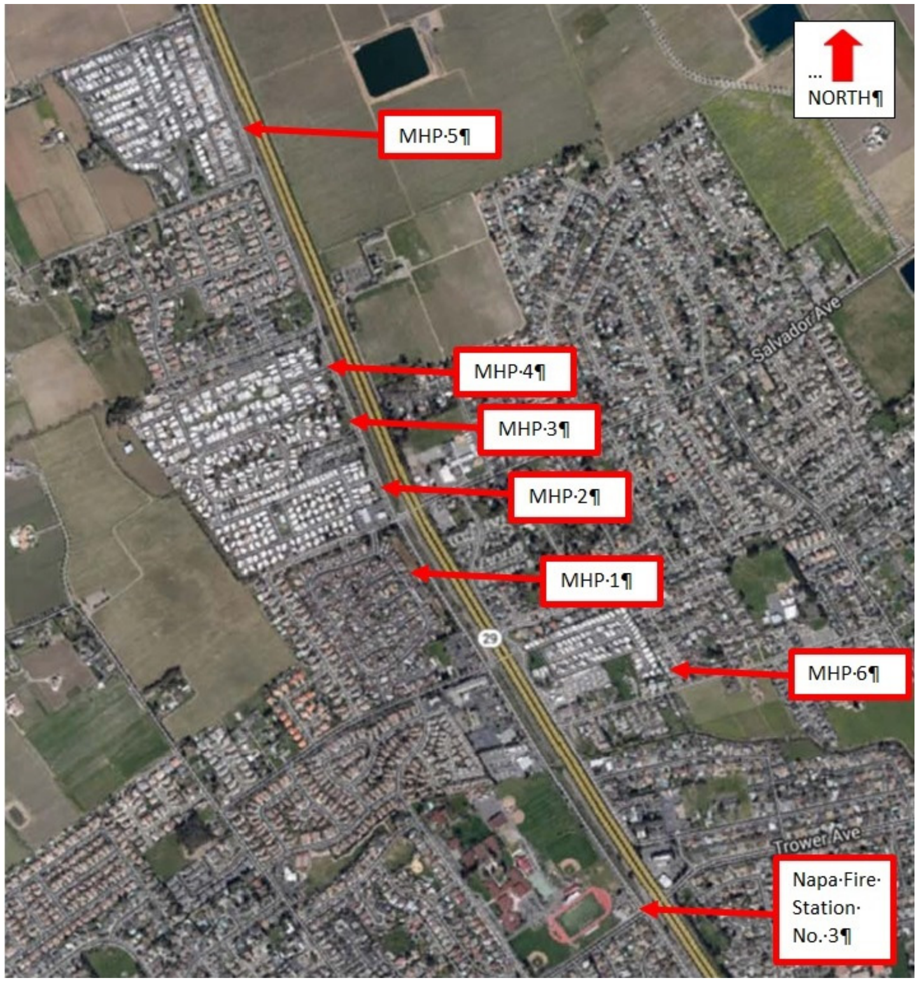


Figure 8-17 Map of six MHPs north of Napa Fire Station No. 3 (image source: Google Maps).

reinstallation may have had little permanent movement and only a few of the gravity piers disrupted, and second, although homes were identified by HCD as having ETS or ERBS, these systems may or may not have been approved or inspected at the time of installation, and so may not be representative of current approval and inspection requirements. Table 8-1 presents that 19% of all homes, 21% of homes with ETS, and 27% of homes with ERBS required reinstallation after the earthquake. In terms of this performance aspect, the performance of homes with ETS and ERBS was poorer than those without.

8.6 Detailed Observations at Oaktree Vineyard Mobile Home Park

No homes in Oaktree Vineyard MHP (MHP 1 on Figure 8-17) were reported by HCD to require reinstallation following the earthquake. Homes in this park were installed over an approximate four year period in the 1980s. The homes in Oaktree Vineyard MHP are unique in a number of ways: (1) all of the homes are of the newer type; (2) each home, in addition to a porch, has an attached site-built enclosed garage (Figure 8-18) instead of a carport that is common in other MHPs; (3) the home siding appears to have been installed on site and is continuous with the garage and porch siding; (4) the roofing appears to have been installed on site, and the roofing is continuous between the homes and garages. In addition, the homes are most often installed in pairs, as is common for two-family dwellings in residential developments.

Table 8-1 Summary of HCD Field Inspection Data Showing Homes Requiring Reinstallation

Park	Number of Homes	Homes Requiring Reinstall	Homes with ETS	Homes with ETS Requiring Reinstall	Homes with ERBS	Homes with ERBS Requiring Reinstall
MHP 1 - Oaktree Vineyard	190	0 (0%)	0	0 (0%)	0	0 (0%)
MHP 2 - Salvador Mobile Estates	147	32 (22%)	17	5 (29%)	36	7 (19%)
MHP 3 - Newells Mobile City	116	36 (31%)	5	2 (40%)	3	2 (67%)
MHP 4 - La Siesta Village	133	27 (20%)	3	0 (0%)	7	4 (57%)
MHP 5 - Napa Valley MHP	243	62 (26%)	27	4 (15%)	24	6 (25%)
Total	829	157 (19%)	52	11 (21%)	70	19 (27%)

Observed damage at Oaktree Vineyard MHP was limited to single-wythe brick masonry skirt walls (Figure 8-19). Support systems were observed for two homes; one had an ERBS and the other did not. Neither home appeared to have moved significantly. Both had damaged masonry skirt walls.

Figure 8-18 Oaktree Vineyard MHP typical home configuration.

Figure 8-19 Damage to brick masonry skirt wall.

8.7 Detailed Observations at Salvador Mobile Estates

According to HCD records, the homes in Salvador Mobile Estates (MHP 2 in Figure 8-17) had the highest occurrence of ERBS and ETS installations prior to the earthquake. The majority of these homes were installed prior to the 1994 requirement for ETSs.

A limited number of homes with the "cantilevered" pier systems were observed, as a number of these systems had been removed during home reinstallation prior to the site visits. The primary mode of behavior observed was rocking or sliding of the “cantilevered” piers (Figure 8-20), which were often not connected to the supporting wood boards. Some occurrences of the top of these piers punching through the home floor above were also observed.

Some homes with “diagonally braced” ERBS (Figure 8-11) were reported by the owners to not have moved at all or only moved a few inches; however, multiple homes with the “diagonally braced” ERBS installed were observed to have slid significantly during the earthquake, but not fallen or dropped appreciably. Figures 8-21 and 8-22 show significant sliding in one home with an ERBS. The gravity piers on this home are believed to have tipped and been reset upright for temporary support until the home can be moved back to its pre-earthquake position. Table 8-2 provides information on four homes with “diagonally braced” ERBS for which measurements or estimates of sliding were made. This extent of sliding was surprising, as significant horizontal movement in

Figure 8-20 “Cantilevered” pier system that slid, adjacent to a masonry pier that has rolled over.

past earthquakes had been primarily associated with homes falling off of supports and dropping, rather than pure sliding behavior. It is of note that significant sliding of the homes occurred in both transverse and longitudinal directions. It is also of note that the primary direction of movement was to the west.

Figure 8-21 Gap between front porch and home on home with ERBS that slid a significant distance.

Figure 8-22 Extent of home sliding. Red arrow is initial wood board position, blue arrow is final position. ERBS can be observed in the background.

Table 8-2 Measured or Estimated Movement of Homes with "Diagonally Braced" ERBS

Home	Transverse Sliding (in.)	Longitudinal Sliding (in.)	Dimension Source
A	negligible	18	Field measured
B	6-8	negligible	Observed damage to electrical hookup
C	15	15	Field measured
D	14	2	Estimated from photographs

In some homes, wheels and tires that were still in place near the rear of the home appear to have limited the displacement at the rear of the home to several inches, causing the front of the home to rotate relative to the rear. For the homes that required reinstallation, the "diagonally braced" ERBS did appear to have kept the homes from dropping appreciably.

None of the homes with "diagonally braced" ERBS were observed to have severely damaged their utility hookups; this appears to have been a matter of chance, with the homes often sliding away from rather than towards their hookups.

One home in Salvador Mobile Estates was damaged by fire following the earthquake. The fire-damaged home did not have an ETS or ERBS. The home both slid and dropped off of its gravity support piers, falling into its utility hookup (Figure 8-23). Information provided by HCD indicates that the gas was turned off immediately following the earthquake and that the fire initiated a couple of hours later in the laundry room. Neighbors indicate that, because of a broken water main, water from tanker trucks were used to fight the fire.

The homes requiring reinstallation in Salvador Mobile Estates were observed to have moved in a predominantly westerly direction, and significant movement was more prevalent at the west end of the park than other areas. It is possible that ground motion characteristics, including a pulse of ground displacement, contributed to this pattern.

Figure 8-23 Manufactured home that slid and fell onto its utility hookup.

8.8 Detailed Observations at Napa Valley Mobile Home Park

Similar to Salvador Mobile Estates, a number of homes at the Napa Valley MHP (MHP No. 5 in Figure 8-17) were observed to have very visible damage. Like in Salvador Mobile Estates, a notable number of units at the west end of the park were damaged and had moved, whether sliding or dropping, in a predominantly westerly direction (Figure 8-24). A handful of homes, however, were observed to be predominantly displaced in an easterly direction, and also experienced significant damage (Figures 8-25 and 8-26). It is possible that a ground displacement pulse is responsible for the damage in both the west and east directions.

Napa Valley MHP had two incidents of fire following the earthquake. It is reported that the water main to the park broke in the earthquake, resulting in there not being water available to fight the fires. One fire destroyed five adjacent homes, and the other destroyed a single home. The causes of the fires are not known; however, either gas line or water heater rupture are thought to be likely.

Figure 8-24 Napa Valley MHP home displaced significantly to the west.

Figure 8-25 Napa Valley MHP home displaced significantly to the east.

Figure 8-26 Entry stairs at Napa Valley MHP home that displaced significantly to the east.

8.9 Summary

Overall the performance of the observed manufactured homes was in line with the intent of the seismic design provisions of recent building codes: protection of life safety.

However, significant damage and disruption occurred to a notable number of manufactured homes, with the most severe damage being homes lost completely to fire. Based on available information, the damage that triggered reinstallations appears to have occurred approximately equally to homes with ETS and ERBS, as compared to those without. The ground motion experienced in this earthquake and the presence of some older (possibly not approved or inspected) ETS and ERBS may have contributed to this behavior. This, however, is still a surprising result and suggests that the ability of these systems to improve earthquake performance needs further evaluation.

In California, ERBS are required by *California Code of Regulations* Title 25, Chapter 2, Article 7.5 to be designed in accordance with the 1982 *Uniform Building Code* (UBC) (ICBO, 1982). The 1982 UBC requires a complete load path for wind and earthquake loads, transmitting the loads to the supporting soils. The "diagonally braced" ERBS that were observed appeared to have provided adequate bracing strength and adequate connection to the home above for the earthquake loads experienced. Further, the practice of bracing between the two chassis beams in the "diagonally braced" ERBS appeared to make the bracing inherently more robust than the "cantilevered" systems, and should be encouraged in future bracing systems. The load path for transmitting loads to the supporting soils, however, was observed to be missing or inadequate, and some homes were observed to slide significantly as a result. This portion of the load path was observed to be similarly missing from the new ERBS being installed following the earthquake.

In California, ETS are required by *California Code of Regulations* Title 25, Chapter 2, Article 7 to be designed for 15 pounds per square foot of wind load applied horizontally to the home. A complete load path capable of resisting this wind load should also provide significant resistance to earthquake loads. Most observed ETS did not appear to have a complete load path capable of resisting this wind load. The observed systems similarly did not appear to be capable of providing a complete load path for earthquake resistance, even though the intent of SB 750 was to provide improved earthquake resistance.

In addition, noteworthy differences in earthquake performance of homes were observed. In general, it appeared that the damage was concentrated in the older type of home rather than the newer homes. It is not clear what caused the better performance of newer homes, but this same pattern was observed in the 2003 San Simeon earthquake (EERI, 2005). It was observed that the wood siding skirts in newer homes (fastened to continuous wood plates sitting on the ground around the home perimeter) appeared to provide some contribution to vertical support, reducing rocking of the homes, and provide some contribution to lateral support; the skirts in the older type homes (which slip into continuous tracks) appeared to provide little to no vertical or lateral support. In addition, it was noted that the add-on structures of porches and garages likely contribute to better performance of some homes.

8.10 Recommendations

Based on the performance of manufactured housing in the South Napa earthquake the following recommendations are offered,:

1. Based on damage observed following the South Napa earthquake and the data presented in Table 8-1, it appears that manufactured homes with ETS or ERBS performed no better than those lacking these systems. It is recommended that current design and approval criteria should be evaluated and potential improvements to the earthquake performance of

ERBS and ETS should be considered. As part of the further evaluation, the previously discussed qualifying factors that potentially impacted earthquake performance should be considered, as should basic principles of earthquake resistance and load path. Specific recommendations include:

- Analysis and testing should be undertaken to better understand the earthquake behavior of these systems.
- Verification that the criteria for ETS provides a meaningful increase in earthquake performance compared to unbraced homes. Since this system was mandated by SB 750 with the intent of providing improved earthquake performance, the ETS should provide better resistance to earthquake force and deformation demands compared to unbraced homes.
- The performance objectives of the ERBS should be reviewed, and the performance of braced homes studied to determine whether the intended performance objective was met, and if so, whether the intended performance is appropriate for future installations. Based on this review, modifications to the requirements for these systems should be implemented to improve performance. This could include field verification that the bracing systems have been correctly installed. The potential for homes sliding into utility hookups and the resulting fire hazard should be included in these considerations.
- The criteria for evaluating and approving both ETS and ERBS should be reviewed relative to performance objectives, and in light of the performance of these systems in the 2014 South Napa earthquake. It is recommended that the process for bracing system approval be reviewed relative to those of other organizations that provide approvals of wind and earthquake resisting systems. It is recommended that the testing procedures conducted for approval of bracing systems be reviewed to ensure that they adequately address performance of the complete installed system, including adequate resistance to local overturning and sliding of the bracing assembly.
- The seismic design criteria for ERBS should be updated to correspond to building code editions currently adopted and enforced by the State of California, especially with regard to the lateral forces used for design. This will allow current quantification of seismic hazard and current thinking on seismic force-resisting systems to be incorporated, which should provide performance that is aligned with other types of structures.

- The current allowance for homes braced with ERBS to drop up to two inches be revisited. This drop can easily result in the blockage of outward swinging exit doors. The restrictions that led to this drop being permitted have not been applicable for 20 years. Prior to September 1994, positive anchorage to the home chassis beams was discouraged because anchorage was defined as an improvement to the property, triggering increases in property taxes; however, due to changes in state laws and regulations in 1994, the allowance for a two-inch drop is no longer necessary.
- Impediments to anchorage of manufactured homes to the ground should be investigated so that solutions involving anchorage can be more widely used. Such impediments may include property tax considerations.
- The complete load path for ERBS and ETS currently being installed should be reviewed to ensure that loads are adequately transferred into the home's floor diaphragm. Many of the current systems attach to the bottom flange of the chassis beams only. Earthquake loading transverse to the home could potentially yield and severely damage the chassis beam.
- Consideration should be given to referencing or incorporating the requirements of the NFPA 255, *Model Manufactured Home Installation Standard*, (NFPA, 2013) which, according to FEMA, provides the necessary installation criteria to meet wind and seismic loads.

2. It is recommended that retrofit strapping of water heaters be required, and, if possible, funded. HCD indicates that seismic strapping of water heaters is only required upon sale of the home or reinstallation of the water heater. Water heaters without strapping remain a significant fire concern.
3. It is recommended that the criteria for postearthquake safety assessment of manufactured homes be reviewed to make sure that occupant safety is adequately considered. It is recommended that a discussion of assessment criteria specific to manufactured homes be revisited in a future update to the assessment methodology, and that such discussion include representatives of HCD. It is also recommended that HCD field staff be trained and certified in the state's Safety Assessment Program to ensure adequate safety assessments of damaged manufactured housing after future disasters.

Chapter 9

Performance of Wine Industry Facilities

9.1 Introduction

Napa Valley is home to one of the world's premier wine-producing region, and has approximately 400 wine production facilities, with an estimated 300 built since 1966. The wine industry is comprised of a diverse mix of wineries, ranging from larger production facilities producing millions of cases annually to smaller operations producing thousands of cases.

An estimated 50 wineries were exposed to significant seismic ground shaking in the 2014 South Napa earthquake and sustained measurable damage to tanks, barrels, or buildings. In this chapter, performance of the wine industry facilities in the Napa Valley is examined.

The majority of Napa Valley wineries are located north of the City of Napa, along Highway 29 and the Silverado Trail, located to the east of Highway 29. Damaged facilities were concentrated northwest of the epicenter, west of Highway 29, reaching approximately 10 miles north of the epicenter. Wineries located in within the 10 mile radius include approximately 30 facilities along Highway 29 or west of Highway 29; most of these facilities were smaller or co-op (where multiple winemakers and winery labels produce wine using the same facility) facilities. The remainder are located southeast of the City of Napa near the airport along Kaiser Road. The facilities along Kaiser Road are generally long-term barrel storage and distribution facilities.

Wineries located more than 10 miles north of the epicenter suffered only minor contents damage or suffered no measureable damage to wine tasting or production facilities.

9.2 Winery Seismic Risk

The earthquake risk profile of a winery is varied based on the time of year. The only constant in the risk profile is the seismic vulnerability of winery buildings. The contents are in constant motion throughout the year as the wine industry is an agricultural business on an annual cycle. The wine production process is a labor-intensive operation that requires movement and processing of the wine throughout the year. The wine production operation is generally characterized as follows, with some variation based on the specific winemaking operations and facility type:

- **August through November:** Harvest and wine grape crush season. The majority of stainless steel wine tanks and a significant proportion of wine barrels are empty at this time, ready to be filled with the new harvest.
- **December through March:** All wine storage vessels, including tanks and barrels, are full of wine. The wine is aging and undergoing filtering and racking operations. White wines are generally stored in stainless steel tanks and red wines in barrels and tanks. Limited bottling operations occur during this time period.
- **March through July:** Wines are moved from tanks to barrels and from tanks to bottling. In general, white wines may be bottled earlier than red wines. Accordingly, the risk exposure includes more full barrels and less full stainless steel tanks. Prior to the harvest in August, the barrels and tanks will be emptied (not all, but most) to make room for the next crush. The majority of content exposure is in finished, bottled goods, with tanks and barrels second. Smaller wineries do not have sufficient on site storage for bottles and case goods, thus are typically immediately shipped to distribution warehouses off site.

The 2014 South Napa earthquake occurred approximately two weeks into the harvest and crush operation, in late August. Barrel stacks consisted of a mix of empty and full barrels, minimizing the total wine lost. The majority of stainless steel tanks were empty at the wineries in the regions of significant seismic ground shaking (MMI VIII or greater). Overall damage to tanks was minimal as empty wine tanks generally perform well in earthquakes. The loss of wine would have been more significant if the earthquake occurred in any month from December through July.

The earthquake occurred in the middle of the night. If the earthquake occurred during normal operating hours, the threat to life safety would have been significant in the barrel rooms. There were no reported injuries or loss of life in wineries.

9.2.1 Direct Loss and Business Disruption

Direct loss of wine and damage to the production equipment varied by location, but was relatively low for Napa Valley as a whole. Wineries reported wine losses from as little as 0.5% (bottles and some barrels) to as high as 15% (barrel stack collapse). Wine loss due to damaged wine tanks was reported as minimal because most tanks were empty.

Business disruption was minimal. Wine tasting facilities were able to clean up broken bottles and displaced contents on Sunday and Monday, open for business on Tuesday, two days after the main shock. The impact on wine tasting facilities was not noteworthy. Winery production facilities were open and operating at nearly all wineries with the exception of Trefethen Family Estates, which suffered significant structural damage to the historic winery building that housed production and tasting operations.

Production facility disruption was attributed to collapse of wine barrel stacks (Figure 9-1) and limited damage to full stainless steel wine tanks. The recovery efforts for the wine barrel stacks were very dangerous and time consuming. The piles of barrels were unstable and the threat of aftershocks resulted in a life safety risk to the employees engaged in the recovery operations.

Figure 9-1 Collapsed barrel stacks.

Each winery developed unique recovery operations based on the level of severity of the damage. The recovery methods included:

- Custom extensions attached to the forklifts to allow personnel to walk along the forks and attach a custom chain to the chime of the barrel (the small lip at the end of the staves adjacent to the head of the barrel). The process typically took two to five minutes per barrel based on the location and the potential for further collapse of the pile upon removal of a barrel.
- Used automotive tires were utilized in piles at the base of the stacks as a safe landing pad. The barrels were manually pushed off the top of the

stack by a person standing on an adjacent stack or by the forks of the forklift. Generally every attempt was made to empty the barrel before it was pushed off to land on the stack of tires. This method proved to be very time consuming but effective in preserving the wine within the barrels.

- Dismantling the partially collapsed stacks with a forklift. The operation was similar to the method in which barrels are moved within the facility during normal operations, with the added risk of subsequent stack collapse. Partially collapsed stacks generally were leaning on other adjacent stacks. Removal of an adjacent level would redistribute the loading and cause collapse of the adjacent stacks. The operation was tedious and risky. Therefore, lessons from these operations should be studied and documented for wineries to use in future emergency response plans.
- Rigging a small crane with two operators to recover the collapsed barrels. The crane was mobile and was the largest unit that could fit within the roll-up doors at the front of the facility. The operation proved to be highly efficient in recovery of both full and empty barrels. The cost of the crane rental was significant but necessary in order to get facilities fully operational in preparation for the harvest and crush in September.

9.2.2 Earthquake Insurance

The vast majority of wineries do not carry earthquake insurance coverage for the contents, including case goods, tanks, and barrels. Earthquake insurance coverage for winery contents is cost-prohibitive or unobtainable for smaller wine production facilities. Some of the larger wineries, with multiple production facilities throughout California and the United States, carry multi-hazard peril coverage, which covers fire, flood, windstorm, and earthquake. In general, earthquake insurance is not common, and had limited impact on the economic recovery of the winery operations.

9.3 Wine Barrel Storage

Wineries in the Napa Valley generally utilize one of the following methods to store wine barrels, listed from most common to least common:

- **Portable Steel Wine Barrel Racks:** These racks were invented in the late 1980s by a former Robert Mondavi Winery employee. There are about six major manufacturers of these racks. The dimensional properties of the racks are virtually identical between manufacturers. The racks are constructed with a mix of mild steel bars and electric resistance welded square tubing. The racks are painted or powder coated for corrosion resistance. They are manufactured in a two-barrel configuration (one rack supports two barrels) and a four-barrel configuration (one rack supports four barrels). The four-barrel configuration consists of two, two-barrel racks linked together to create a longer footprint in one direction. The most common configuration used in California wineries is the two-barrel rack. The use of the four-barrel rack is generally reserved for high production, large volume wine facilities. Small wineries typically do not have the floor space or forklifts with the capacity required to move the four-barrel rack. The barrel rack systems allow the winery to maintain high density storage, limited only by the clear height of the roof and the available floor space. Stack heights range from one level up to a maximum of six barrels high. Above six high, the weight of the full barrels will deform and potentially crush the barrel on the bottom level. Each level of barrels is approximately three feet tall.

- **Pyramid Stacks:** In this approach, the barrels are stacked atop one another in a pyramid form. The barrels are "chocked" into place with wood wedges or using custom fabricated steel cradles. Pyramid stacks are used in facilities that can access and maintain the wine in-place, rather than moving the barrels to a dedicated washing and maintenance region (typical of wineries using the portable steel barrel rack system). Pyramid stacking is most commonly used in subterranean caves, with stack heights generally no taller than three barrels. Taller stacks require a ladder to access the upper level barrels.
- **Fixed, Engineered Rack Systems:** A limited number of facilities use fixed storage rack systems, generally designed and constructed in accordance with the building code of the era. These systems are generally constructed of heavy timber framing or structural steel. Only one facility in the region of high seismic ground shaking used this system.
- **Other Storage Systems:** There are a number of other barrel storage methods in limited use in the Napa Valley. This includes rack configurations that are a hybrid of a portable rack in a pyramid configuration. Other racks are similar to the portable steel barrel rack, but use a steel post and beam frame. The barrels are supported on the frame and subsequent levels of the racks are supported on the four corner posts, not the barrels below.

There was no reported loss of barrels in caves with pyramid stacks. The wineries in the affected region (within 10 miles of the epicenter) generally used portable steel barrel racks. No damage to engineered rack systems was reported.

There was no report of collapse of wine barrels on four-barrel rack system. One winery within 0.5 miles of the epicenter lost two barrels from the top of five-high stacks of portable steel four-barrel racks.

One of the largest wine co-op facilities suffered the most significant loss documented in the earthquake, where over 60% of the barrel stacks collapsed. This facility houses wine barrels for hundreds of different custom crush clients. The recovery operation was time consuming and dangerous, compounded by the numerous winemakers arriving at the site after the earthquake with the sole purpose of recovering their own barrels, disrupting the recovery process.

Smaller wineries typically do not have operational requirements to stack the wine barrels to the heights used by the larger operations (5 to 6 levels high). The losses to barrel stacks were generally less with these facilities due to the lower fall heights and durability of the wine barrels.

9.4 Wine Tanks and Catwalks

The majority of stainless steel wine tanks in the affected region were empty in preparation for the wine grape harvest and crush. Thus damage to wine tanks was limited. The number of tanks damaged beyond repair is currently reported as eight. All damaged tanks were full of wine. There was no reported damage to empty tanks. The total loss of wine due to tank damage is still being collected as of the publication of this document, but is estimated to be just over 10,000 gallons inclusive of all wineries.

Tank anchorage appeared to play a part in the overall performance. Tanks with appropriately sized and spaced anchors performed well. Damage was limited to minor deformation of the bottom course of stainless steel in the structural wall. Tanks without anchorage shifted on the cellar floor or shifted on concrete pedestals (Figure 9-2). The movement of the tanks resulted in rupture of fixed glycol cooling lines and tank-supported catwalks. One tank-supported catwalk collapsed when the tank shifted off its concrete pedestal. Poorly anchored tanks had the worst performance. Some tanks with poorly-designed anchorage and limited number of anchorage points suffered failure of the tank wall at the anchorage. Wine was lost through the resulting hole in the tank wall on four 5,000 gallon tanks in the Oakville region. The tanks were anchored using a 1/4 inch weld, 4 inches long, between four steel embed plates in the concrete pedestal and the base of the tank.

Figure 9-2 Anchorage failure allowed tank to shift on pedestal.

9.5 Bottle and Case Good Storage

Wrapped and palletized case good storage performed well in this earthquake. There was no reported loss of wine due to collapsed case good storage on bulk pallets. The bulk of bottling operations were recently completed and most of the case goods had been shipped off site to distribution facilities and wine consumers.

Over 100 wineries reportedly suffered losses to individual wine bottle storage and loose case goods, generally within the tasting facilities or the estate wine libraries. Bottles toppled from shelves and lay-down, wall-mounted wine racks. Total loss of wine from bottle breakage is not fully documented but is a marginal percentage of the total production of the wineries in the region. The damage resulted in restricted use of the tasting facilities for generally two to three days.

9.6 Winery Building Performance

Compared to the performance of wine facility contents, the buildings performed relatively well. Construction types, configurations, and era of construction of the winery facilities in the effected regions are varied. The most common construction types include concrete tilt-up, concrete masonry bearing wall, wood-frame, and pre-fabricated metal buildings. The majority of winery buildings are of 1960s or newer vintage.

The following sections summarize the performance at four winery facilities that sustained notable structural damage.

9.6.1 Historic Winery Building, Napa

The three-story timber-frame winery building, built in 1886 sustained significant structural damage. The building was posted UNSAFE and will be repaired and returned to service. Full wine barrels, stacked on portable steel barrel racks (three levels tall), were stored on the second floor level. The mass of the barrels contributed to the shear failure of the exterior lap siding. Residual interstory drift at the first story was approximately 15%, with virtually no permanent drift at the second and third stories (Figure 9-3).

Figure 9-3 Historic timber-framed winery building with large residual drift at the ground floor.

9.6.2 Wine Co-Op Facility, Napa

The facility sustained significant damage to the interior structural steel columns due to impact with toppling wine barrel stacks (Figure 9-4). The building was constructed circa 1970 and is a one-story, pre-fabricated metal structure. Damage was sufficient to require replacement of one third of the columns. No other structural damage was noted.

9.6.3 Winery Facility, Napa

This facility sustained damage to the exterior corrugated metal walls due to interaction with collapsed wine barrel stacks. Barrels stacked on two-barrel racks toppled and blew out the wall at the rear of the main barrel cellar. The building was constructed in the 1980s and is a one-story, pre-fabricated metal structure. No other structural damage was noted.

Figure 9-4 Steel columns damaged by impact of barrels.

9.6.4 Barrel Warehouse, Napa

This facility sustained partial collapse of the wood-frame roof structure at the south end of the property. The building was constructed in the 1950s and is a one-story concrete masonry bearing wall building with a panelized wood-frame roof. The connection between the central glue laminated beam and the south wall pilaster failed, resulting in a loss of support at the south end of the glue laminated beam (Figure 9-5). On Monday August 25, the day after the earthquake, the roof was hanging due to secondary action of the roof plywood. The plywood diaphragm nailing started systematically failing in the early afternoon of August 25, resulting in a complete collapse of the south roof at about 3 in the afternoon. The roof was demolished and reconstructed with a seismic upgrade at the south end. A seismic upgrade at the undamaged regions is planned as a second phase.

9.7 Summary

9.7.1 Structural Damage

The earthquake showed that the mass of the toppled wine barrel stacks is sufficient to damage buildings structurally, and can result in local or global collapse of the building. Storage of wine barrels on the upper floors of a structure can create a serious vertical mass irregularity. Barrels placed directly against the perimeter concrete tilt-up or

Figure 9-5 Failure of glulam beam to wall connection resulted in roof collapse.

concrete masonry unit (CMU) walls may topple against or impact the wall, imparting additional mass and increasing the out-of-plane force demands on the roof diaphragm connections. The additional mass will increase the potential for wall-anchorage failure unless the mass of the barrels was considered in the original design of the wall anchorage.

9.7.2 Wine Barrel Storage

The overwhelming proportion of wine loss and wine barrel collapse was due to the two-barrel portable steel barrel rack system. This rack has demonstrated poor performance in the 2000 Yountville, 2003 San Simeon, and 2014 South Napa earthquakes. The barrel stacks can collapse due to "walking" of the racks over the barrels below or by sliding of the racks off the barrels below. Failure can occur at any level of the stack. The rack sliding or walking is affected by the point of contact with the barrel below. The performance of the two-barrel rack has been well-documented through prior California earthquakes and academic research. There were no reported collapsed stacks using the four-barrel rack.

Wine barrels are resilient and can endure a fall from heights generally below eight feet. Facilities with significant stack collapse suffered relatively low losses due to the cascading collapse of the stacks and the reduced fall height of the barrels. Wine loss was generally attributed to the first wave of barrels to fall directly on the warehouse floor and barrels with dislodged bungs, resulting in slow leaks and wine oxidation. The steel hoops on the barrels are spaced at various locations along the length of the barrels (this varies by barrel type and manufacturer). The coefficient of friction between the steel rack and the steel hoops is sufficiently low compared to the rack supported on the wood staves. Further research is planned to study the effects of this interface with a future recommendation to barrel manufacturers on proper hoop placement.

9.7.3 Steel Tanks and Catwalks

Poorly anchored tanks exhibited poor performance in comparison to properly anchored tanks. Wine losses were generally attributed to poorly anchored tanks. The performance of unanchored tanks could not be well studied, since the majority of the unanchored tanks in the effected region were empty.

Wineries should consult with the tank manufacturers to obtain appropriate anchorage design for high seismic regions. The design of the anchorage should consider the tight spacing between the tanks. The anchor capacity is often times controlled by the edge clearance at the concrete pedestal or the center-to-center distance between adjacent anchors.

Catwalks should not be supported on tanks that are unanchored. Movement of the tank can result in failure of the catwalk supports, resulting in a threat to life safety if a winery employee were on top of the catwalk or below it.

9.7.4 Bottle and Case Good Storage

Bulk, palletized case goods tend to perform well when subjected to strong seismic ground shaking. Wine bottles stored on shelves and lay-down storage racks are vulnerable to toppling or sliding. Winery tasting facilities could implement simple restraint systems, such as cables or wires, to prevent toppling of the bottles. Lay-down wine racks could be outfitted with rubber pads to provide increased friction to prevent sliding of the bottles in the racks. A number of wineries lost individual bottles in their on-site estate wine collections. The economic loss of these bottles was significant compared to the wine available for retail purchase in the tasting room.

9.8 Recommendations

Recommendations for winery operations and future research opportunities are listed below.

9.8.1 Wine Barrel Storage

1. The four-barrel portable steel barrel rack demonstrated superior seismic performance in comparison to the two-barrel rack. Use of the four-barrel rack is highly recommended.

2. Whenever possible, the height of the stacks should be limited to four or less. Shorter stacks tend to perform better and are less likely to topple. The lower height reduces the fall height of the wine barrels, decreasing the possibility of wine loss due to barrel breakage.

3. When using the two-barrel portable steel barrel rack, the stacks should be oriented with the heads touching in adjacent rows. The rows should be oriented with the barrel bilges in contact. The current common practice is to stack barrels in deep rows with the heads in contact. Collapse of the stacks results in a domino effect as observed in this earthquake and prior earthquakes.

4. Appropriate space should be provided between the exterior walls and the barrel stacks to avoid interaction should the stacks become unstable or collapse.

5. Stacking barrels on warehouse floors with aggressive slope for drainage should be avoided. Barrel stacks will have a static lean towards the central drains, increasing the likelihood of collapse when subjected to seismic ground shaking.

6. Wine barrel storage on portable steel barrel racks or pyramid configurations is not subject to the nonstructural seismic design provisions in the 2013 *California Building Code* (California Building Standards Commission, 2013a) and ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010). The barrels are classified as contents and are not required to be anchored or permanently fixed to the foundation. Further discussion on the performance of this system is warranted within the framework of the structural and fire safety provisions in the building code.

7. The top barrels should be restrained to their rack in stacks of barrels on portable steel barrel racks. The barrels can be restrained with a variety of methods, including straps, clips, or other means. Without restraint, the top barrels can easily roll off the supporting rack below and fall to the cellar floor.

8. Further research is warranted to study options to improve the performance of the two-barrel rack. The wine industry uses this rack throughout the western states and switching to a four-barrel rack system

and the equipment to handle them is generally cost-prohibitive. Additional research could focus on simple and cost-effective strategies to improve the performance of this system and allow wineries to continue using it.

9. Seismic isolation systems could be an option to limit the seismic ground shaking experienced by the barrel stacks. Prior research into the application of isolators (full building isolation or low-mass isolator pads beneath the barrel stacks) revealed that performance could be improved and damage minimized. However, the cost of isolation systems is a significant barrier. In addition, the use of isolation pads beneath the barrel stacks is not logistically feasible for most high production wineries. The highly corrosive, damp environment is not ideal for ball-and-cone or friction pendulum isolator applications.

9.8.2 Wine Tanks

1. Wine tank shells and anchorage should be designed in accordance with the appropriate American Water Works Association (AWWA) or American Petroleum Institute (API) design standards for thin-walled vessels. Stainless steel wine tank manufacturers should be attentive to the tank anchorage and the placement of the tank in the facility to ensure sufficient edge clearance for the embedded anchor bolts.
2. Wine tanks with properly designed anchorage have generally better performance over tanks with no anchorage or poorly designed anchorage. Existing tanks should be evaluated for anchorage capacity and future seismic performance. Retrofit of the tank anchorage may be warranted to limit future damage.
3. Unanchored tanks should conform to the requirements of ASCE/SEI 7-10 Chapter 15, Sections 15.7.5 and 15.7.6. They should not be connected to the cooling supply lines with rigid connections. Snubbers could be used to restrict the movement of the unanchored tanks. The snubbers could allow some level of sliding to dissipate energy.
4. Independent gravity support for catwalk systems is desirable. Tank-supported catwalk systems should only be used with tanks that are properly anchored.

9.8.3 Occupant Seismic Safety, Response, and Recovery

1. In order to improve personal protection if occupants cannot get out of the building in time, steel cages mimicking the design of a forklift rollover protection cage should be provided throughout the barrel storage facility. They could be placed at identified locations throughout the barrel room

and would provide a safe haven for employees working in the room during an earthquake.

2. Wineries should study the fall and collapse patterns of the wine barrel stacks. The study can provide a means for access and egress planning and disaster response and recovery within the winery. The study should include potential interaction between the barrel stacks, the building and surrounding process equipment and wine tanks.
3. The noise from winery operations could hamper an employee's ability to sense when an earthquake is occurring. The winery facilities could be outfitted with a public announcement (PA) system with air horns or sirens. The PA system could be linked to a seismic ground motion sensor or an early warning system. A loud horn or siren, together with the appropriate training, could provide the necessary alert and improve life safety in the facility.
4. Whenever possible, exposure of the public and winery guests to the wine barrel stacks should be limited. Tasting facilities and private events are often located in close proximity to the wine barrel stacks. The poor performance of this storage system warrants revisiting this practice and limiting access to the barrel storage areas to winery employees with proper training.
5. A response and recovery plan should be developed, as well as staff training. Annual recovery drills should be conducted and special equipment should be made available to speed the recovery efforts. Wineries should share their recovery best practices within the industry.
6. Response and recovery may include industry mutual aid partnerships with wineries in other regions and or plans to quickly move barrels and finished wine to other facilities outside the affected region.

Chapter 10
Performance of Nonstructural Components

10.1 Introduction

Nonstructural components include all of those building components that are not part of the structural system, including architectural, mechanical, electrical and plumbing systems, as well as building contents, such as furniture and inventory in commercial and retail establishments. Damage to nonstructural components and systems in buildings accounts for the majority of repair costs in earthquakes, and can contribute to extended downtime for repairs. While most modern structures sustained little or no structural damage in the 2014 South Napa earthquake, several sustained significant nonstructural damage. Also, the only fatality attributable to the earthquake was caused by nonstructural damage.

Seismic design requirements for nonstructural components are contained in Chapter 13 of ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*, (ASCE, 2010), although furniture and most building contents are exempt from code requirements. FEMA E-74, *Reducing the Risks of Nonstructural Earthquake Damage*, (FEMA, 2012a) explains the sources of earthquake damage that can occur in nonstructural components and provides information on effective methods for reducing risk associated with nonstructural earthquake damage, and provides building code provision information for commonly used nonstructural components.

The following factors influence the seismic performance of nonstructural components:

- Codes and standards must provide a suitable basis for design.
- Design requirements for nonstructural anchorage and bracing must be properly followed.
- Items that require seismic design must be identified and the construction documents must clearly illustrate the proper installation details.
- Nonstructural components must be installed in accordance with the construction documents.

- Adverse interaction (impact) between nonstructural components or nonstructural component and the structure must be avoided.
- In cases where components are exempt from code-mandated seismic design, consideration of the potential seismic performance should be considered and protective measures implemented where needed.

A deficiency in any of these steps may result in a component installation that is vulnerable to unacceptable earthquake damage.

Seismic provisions of the building codes in general have undergone significant changes in the last two decades, and the requirements for nonstructural components in particular have changed dramatically. The types of nonstructural components subject to the seismic provisions have been greatly expanded, the design forces for components have increased, and the requirements for attaching the components to concrete and masonry construction are much more comprehensive. These changes complicate the postearthquake evaluation of nonstructural damage, since the rules under which the component was installed may be much different than those recommended today. If the component in question does not fully conform to those current requirements, care must be taken when drawing conclusions on the adequacy of current code requirements.

In the following sections, observations on the performance of different types of nonstructural components are presented, along with example buildings that highlight the nature of the damage sustained. Some of the buildings discussed are located within the 1,000 foot radius around the Station N016; some are not. The structural and nonstructural performance of some of the buildings included in this study are discussed in previous chapters. Observations on the nonstructural performance of these buildings are not repeated here, and where the nonstructural performance of these buildings is of special interest, a reference to the appropriate section of this report is provided.

10.2 Exterior Enclosures

The buildings surveyed had a variety of building exterior enclosure systems, including glazed curtain wall and storefront systems, light-frame wall systems with stucco finishes, natural and artificial stone veneers attached to light-frame wall systems or to structural walls, and precast concrete cladding.

10.2.1 Glazing, Glazed Curtain Walls, and Storefront Systems

Damage to glazing was fairly widespread. Of the 68 buildings included in the 1,000 foot radius survey around Station N016, 22 buildings sustained damage to 5% or more of the exterior glazing and four buildings to 50% or more of the exterior glazing.

The performance of glazing systems depended in large part on the amount of story drift the building experienced, and the tolerance of the glazing assembly for story drift. In some cases, the glass did not crack or shatter, but the gaskets holding the glass in frame loosened, allowing the pane to shift.

Large panes of glass were more vulnerable than smaller panes. Older buildings, which tended to have open storefronts with large panes of glass, suffered most of the glazing damage. The head of a single-bay storefront glazing system at Queen of the Valley hospital displaced several inches out-of-plane (see Section 5.2.1).

One-story unreinforced masonry buildings with open storefronts within the 1,000 foot radius sustained losses of up to 60% of the storefront glazing (Figure 10-1). In retrofitted unreinforced masonry buildings, glazed storefront damage was more pronounced on building elevations where lateral resistance was provided by moment frames, compared to buildings with stiffer lateral force-resisting systems. The building in Figure 10-2 was retrofitted with a steel moment frame along the front. The stiffness of the steel frame appears to have been insufficient to limit drift sufficiently to protect the glazing.

Figure 10-1 Broken glass at open storefront on an unreinforced masonry building.

Figure 10-2 Typical condition showing temporary plywood barricades and plastic over openings where storefront glazing was damaged.

Nine of the ten plate-glass windows in the Napa County air traffic control tower, each 3/4” thick, where shattered during the earthquake (Figure 10-3). No employees were present during the earthquake, because the Napa tower operates only between 7 a.m. and 8 p.m. Consequently, there were no injuries. No structural damage was reported. A temporary tower was brought to the site to allow for continued use of the airport (Figure 10-4).

Figure 10-3 Broken glass in Napa County air traffic control tower (photo by J.L. Sousa/Napa Valley Register/ZUMAPress.com).

Figure 10-4 Temporary air traffic control tower.

10.2.2 Light-Frame Curtain Wall Systems

Minor cracking of stucco finishes on light-frame exterior walls was observed in many buildings within the 1,000 foot radius survey around Station N016. The performance of light curtain wall systems on modern structures varied significantly, with several buildings sustaining substantial cracking, connection failures to the building structure, and loss of veneer. Other modern buildings sustained insignificant damage to their exterior curtain walls.

A three-story office building with a steel moment-resisting frame sustained significant curtain wall damage. The performance of this building is discussed in detail in Section 4.2.2. The wall system lacked a mechanism to accommodate story drift, and lateral displacements of the structure in the earthquake resulted in connection failures, separation of the curtain wall from the structure, and permanent displacements. Significant damage to a light-frame curtain wall system was also observed in a five-story hotel, which also sustained damage to stone veneer. This building is discussed in Section 4.2.3. Failures of both anchored and adhered veneers were observed in these buildings. The performance of adhered veneer was directly related to the performance of the substrate and the strength and condition of the adhesive material.

In one building, an entire built-up light-frame exterior wall unit detached from the building and fell on the sidewalk (Figure 10-5). It was reported that the wall connectors were weakened as a result of corrosion.

Figure 10-5 Metal stud and stucco framed cladding dislodged from concrete wall.

10.2.3 Precast Cladding

There were few buildings with precast cladding in the 1,000 foot survey area. Immediately outside the 1,000 foot radius, one precast panel dislodged and fell from a telephone operations facility, built circa 1961, clad with precast concrete panels anchored to a concrete frame with steel angles and embedded anchors. During the earthquake, one panel in the penthouse fell outward (Figure 10-6). There were early reports that the panel was designed as a removable panel to facilitate adding or removing equipment from the room; however, this could not be confirmed. Nearly identical precast panels in the facility are connected to the structure using angle and anchor details similar to those that failed. The dislodged wall fell as a unit and severed the building's utility connection. The backup generator in this facility also failed, forcing the facility to maintain operations using stand-by batteries. A temporary generator and cooling unit were installed to keep the computer servers running during repairs.

(a) (b)

Figure 10-6 Dislodged precast panel in telephone operations facility: (a) missing panel with orange brackets remaining in place (photo from PEER (2014)); (b) temporary covering after the earthquake.

10.3 Interior Partitions

In the observed buildings, most interior partitions were constructed with wood or metal studs with gypsum wallboard or plaster sheathing. Partition damage was present in many buildings, but most damage was incidental with a very small number of buildings experiencing substantial partition damage.

Within 1,000 feet of Station N016, the interiors of 40 buildings were observed. Partition damage was described as "insignificant" or "none" in 91% of the buildings with interiors observed. Damage was reported as "minor" in 6% of the buildings. Two buildings were reported with "moderate" damage and none was described as "heavy." The buildings with moderate partition damage also suffered consequential damage due to damaged sprinkler piping, which released water for an extended period. As a result of water damage, large areas of gypsum wallboard sheathing required removal and replacement in these buildings (Figure 10-7). In virtually all other cases, repair could be accomplished with local patching and painting (Figure 10-8 and 10-9).

Figure 10-7 Gypsum wallboard was removed from partition walls where water damage from a broken sprinkler head occurred.

Figure 10-8 Typical damage to gypsum wallboard.

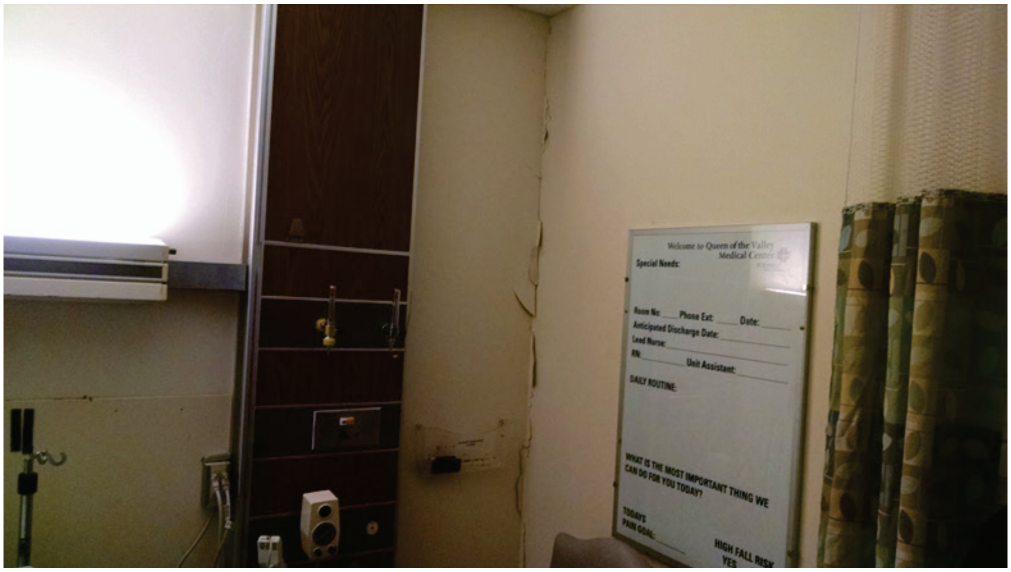


Figure 10-9 Damage to gypsum wallboard at corner (photo from OSHPD).

Older buildings with lath and plaster wall finishes sustained more significant damage in the form of cracking and spalling (Figure 10-10).

Figure 10-10 Cracked and spalled plaster on lath and plaster wall.

10.4 Ceilings

Two types of ceilings represent the majority of types affected by the earthquake: (1) gypsum wallboard or plaster fastened directly or indirectly to structural framing; and (2) suspended acoustic tile ceilings.

10.4.1 Gypsum Wallboard and Plaster Ceilings

Gypsum wallboard ceilings fastened directly to structural framing generally performed well. Minor cracking was observed at some locations (Figure 10-11). Older plaster ceilings did not perform as well as gypsum ceilings. Spalling and cracking of these ceilings were observed in multiple buildings (Figure 10-12 and Figure 10-13).

Figure 10-11 Cracking of gypsum wallboard ceiling.

Figure 10-12 Cracking and spalling of a plaster ceiling in an older masonry building in downtown Napa.

Figure 10-13 Cracking and spalling of a plaster ceiling in an older reinforced concrete building in downtown Napa.

10.4.2 Suspended Acoustic Tile Ceilings

Within the 1,000 foot survey area, 16 buildings (24% of the total) were identified to have at least some area with suspended acoustic tile ceilings. Some damage to the ceilings of these buildings was common, though in only one building was it classified as "heavy." On the opposite side of the spectrum, only one building was assessed as having no ceiling damage. The majority of these buildings had "insignificant" damage, and two had "moderate" damage.

The most commonly observed form of damage to suspended acoustic tile ceilings was fallen tiles (Figure 10-14). Failure of splices in the ceiling grid members, as well as damage at the "fixed" and "free" ends of the ceiling grid, were observed in several buildings. Figures 10-15 through 10-17 illustrate a ceiling estimated to be 20 years old that included diagonal wire bracing without compression posts. Although reinstalling or replacing tiles occurred quickly, failure of the grid itself required an extended period to restore. In extreme cases of damage, removal and replacement of the entire ceiling was required.

Figure 10-14 Acoustic ceiling where tiles fell in an office building.

Figure 10-15 Failure of a splice of acoustical tile ceiling grid in a two-story retail store in downtown Napa.

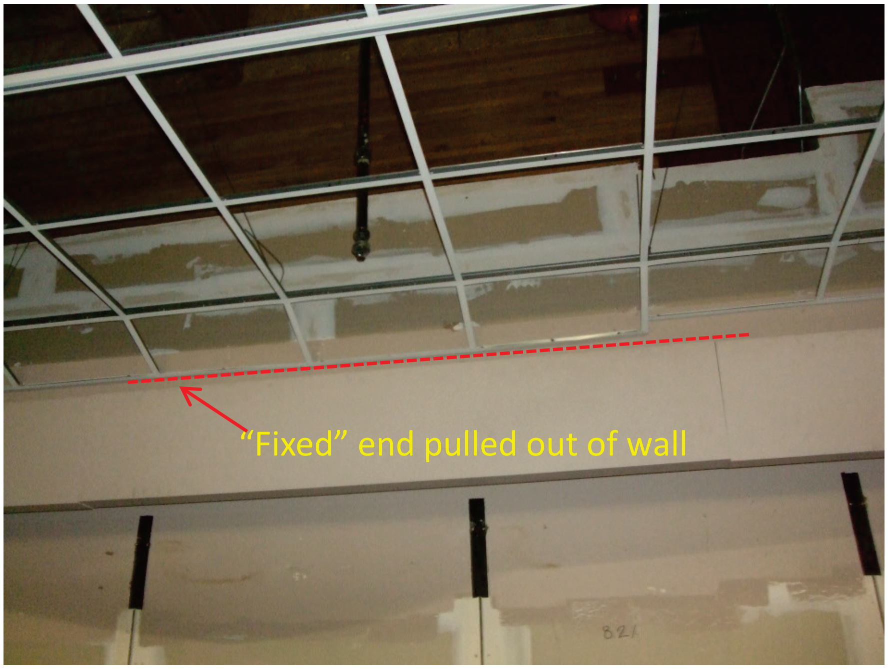


Figure 10-16 Failure of "fixed" end support of the acoustical tile ceiling grid in a two-story retail store in downtown Napa.

Figure 10-17 Failure of "free" end support of the acoustical tile ceiling grid in a two-story retail store in downtown Napa.

10.5 Mechanical, Electrical, and Plumbing Equipment

The performance of mechanical, electrical, and plumbing (MEP) equipment varied widely in the buildings investigated. Some unanchored or lightly restrained equipment shifted or overturned, and rooftop equipment was more heavily damaged than equipment located elsewhere in the building. There were failures observed in anchored equipment, but based on the estimated age of the equipment installation, the failures occurred in components that would not comply with current code. MEP components installed to recent standards generally performed well, with the exception of pendant light fixtures. The Napa County Hall of Justice (Building F3) consists of two wings, one constructed circa 1974 and the other constructed circa 1989. The MEP components in the newer wing performed substantially better than those in the 1974 Wing. Performance of the MEP components in this facility is discussed in Section 4.3.5. In several schools (see Chapter 6) and grocery stores, suspended pendant light fixtures became dislodged and dropped. Had the earthquake occurred during the day, related injuries would have been likely.

At a grocery store on Trancas Avenue, Napa, the primary lighting in the store, which was constructed within the past ten years, consisted of pendant fixtures suspended over the aisles. The 1 foot by 4 feet fixtures were connected end-to-end and mounted directly to struts that were in turn connected via aircraft cable to an "S" hook that connected to an eyebolt fastened to the structure. Approximately 10% of the fixtures fell because the "S" hook opened up under the combined forces due to seismic loads and the weight of the lights (Figure 10-18). The connection detail is shown in Figure 10-19.

Figure 10-18 "S" hook opened during earthquake and caused fixtures to drop.

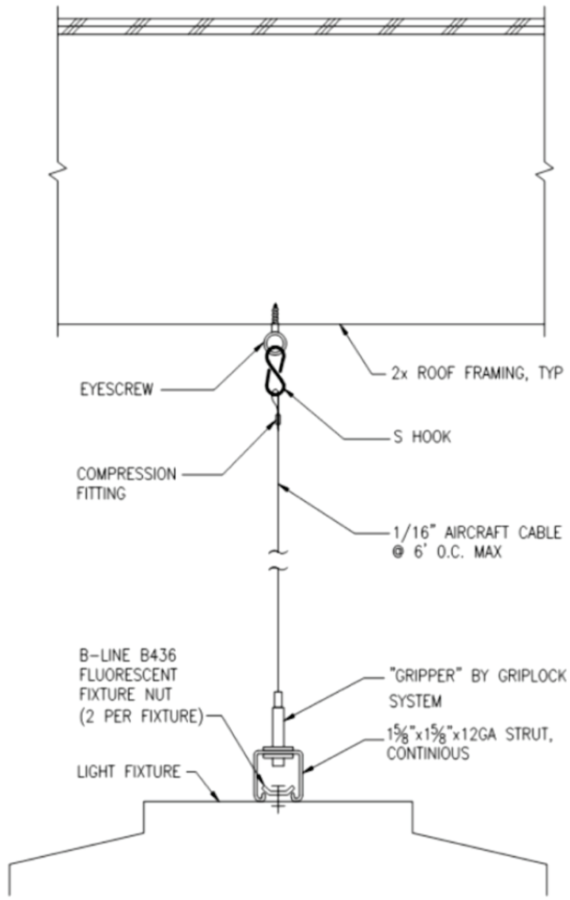


Figure 10-19 Pendant fixture support detail.

10.6 Piping Systems

Piping systems were the source of a considerable portion of damage sustained in the earthquake. Automatic fire sprinklers and related piping caused substantial damage, in some cases even significant flooding. The vulnerability of smaller diameter piping, the lack of adequate clearance between the fire sprinkler heads and other equipment, such as ductwork and suspended air handling units, and the failure of piping connected to inadequately anchored equipment were also demonstrated.

The county office building on 1st Street, Napa (Building C4) is a two-story concrete shear wall structure with a mezzanine. This building sustained extensive damage when a single sprinkler head on third floor above the ceiling began discharging water after it impacted either a U-hanger for the branch line or an adjacent wood beam (Figure 10-20). The sprinkler system could not be shut off, and the sprinkler ran for five hours. All partition walls and floor coverings sustained extensive water damage. The building was expected to be out-of-service for several months.

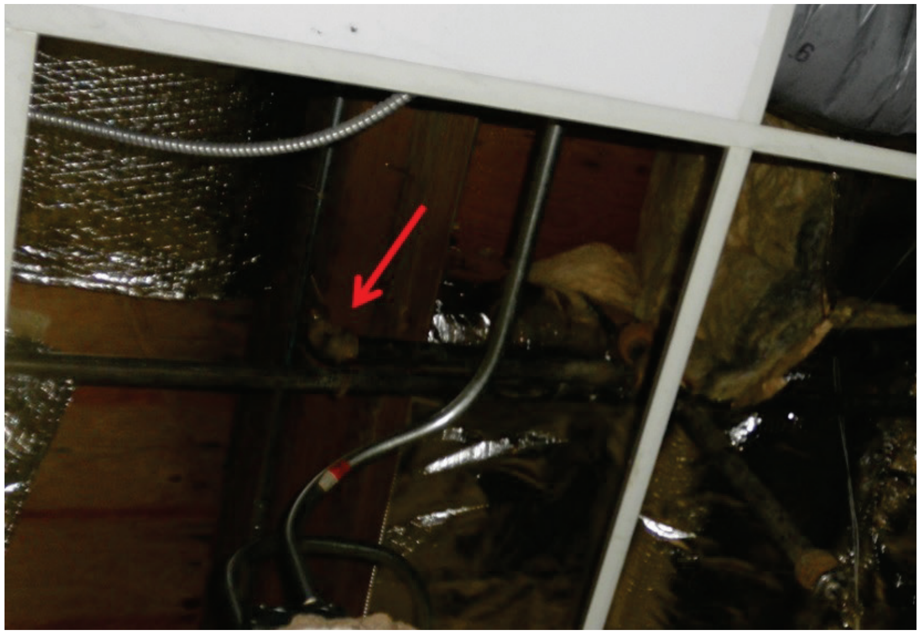


Figure 10-20 Sprinkler head interaction with adjacent components resulted in extensive flooding.

A single sprinkler line failed at a threaded fitting and flooded the first floor of a three-story theater building on Main Street in Napa (Building L2) (Figure 10-21). The water to the fire sprinkler system could not be shut off because the valve was locked in the open position. The fire pump activated and continued to run, and the flooding continued for about nine hours. The hardwood floor in the first floor warped and in some areas buckled. In addition, some of the partition finishes were damaged by the water and had to be replaced. The building was closed for a week to dry out the flooring and repair water damage.

Figure 10-21 Repaired condition of a failed pipe at threaded connection that resulted in flooding of the first floor restaurant.

Five sprinkler lines suspended from the roof of a two-story retail building in the Napa Town Center reportedly broke and remained on for several hours. The sprinkler piping had threaded connections and damage appears to have been related to swaying and interaction with adjacent HVAC components (Figures 10-22 and 10-23). The elevator and escalator pits were flooded and both systems required repair. Contents were damaged as a result of the water; and considerable amounts of gypsum wallboard were damaged and required removal.

Figure 10-22 Sprinkler piping in close proximity to HVAC components.

Figure 10-23 Sprinkler armover in close proximity to HVAC components.

Domestic water pipe breaks in the penthouse of the Napa County Administrative Offices (Building F2) resulted in water damage to the third floor ceilings. The conditions contributing to the pipe failures are discussed in Section 4.3.4.

Unbraced sprinkler pipes in a parking garage in downtown Napa supported with threaded rods and powder driven fasteners sustained several types of damage including permanently bent threaded rod hangers and pullout or failure of powder driven fasteners (Figures 10-24 and 10-25). No water release was reported. This type of fastener is no longer permitted by the building code and the city is not allowing its use for repairs.

Figure 10-24 Sprinkler installation in parking garage.

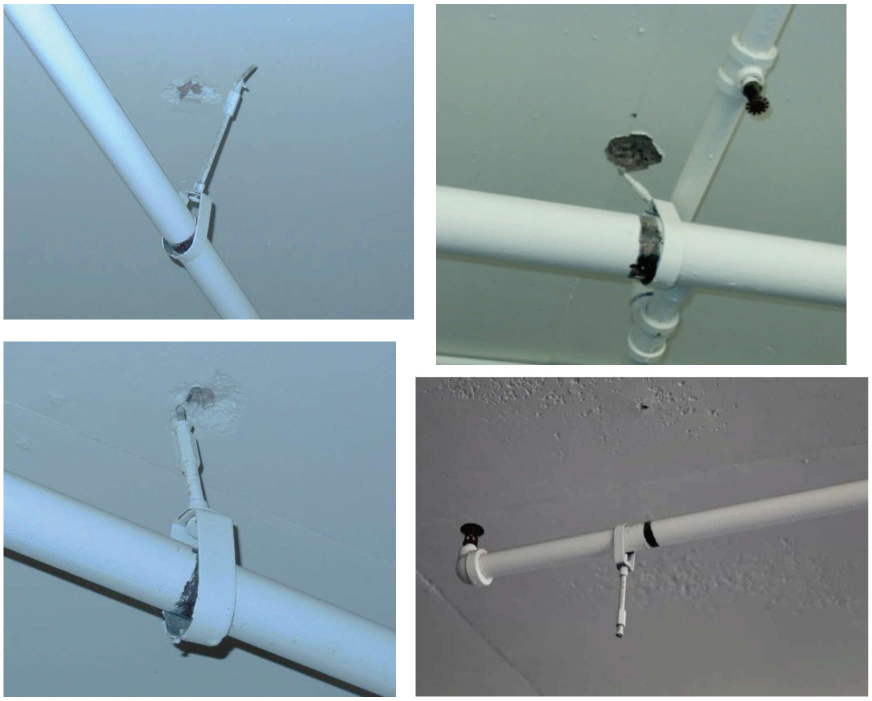


Figure 10-25 Failures of powder driven fasteners. Bottom right photo is from PEER (2014).

10.7 Contents

Contents generally refer to components that are not part of the building architecture or MEP systems, furnished and installed by the occupant, and generally not regulated by the building code. This includes temporary or movable items, equipment that is not permanently attached to the structure such as desktop items (e.g., computers, copiers, lamps), and most furniture, except permanent floor-supported storage cabinets, shelving or book stacks over 6 feet tall. Even though they are not regulated by the code, contents pose a seismic risk. Unrestrained items may slide, impact other items, tip, or overturn, and in some cases, block exits. Failure of one item may damage others or cause the collapse of other items. Contents supported on furniture or fixtures may fall, break, or spill.

Virtually all buildings within 1,000 feet of Station N016 experienced some contents damage. In some cases, the damage was limited to shelf-mounted items shifting or falling. In others, larger furnishings overturned or shifted. Had the earthquake occurred at another time of day, contents damage could have caused injury or death. The one death attributable to the earthquake was caused by a television that shifted off a table and struck the victim in the head.

The following general observations are noted:

- The majority of retail stores, including wine and olive oil shops, and restaurants, experienced loss of product from shelving (Figures 10-26 and 10-27). An example of a wine storage rack in a retail store where no bottles were lost is shown in Figure 10-28.
- In some stores, unanchored shelving racks or storage units overturned into aisles (Figure 10-29)
- Some office furnishings overturned or shifted, posing a life-safety hazard had the spaces been occupied at the time of the earthquake (Figure 10-30)

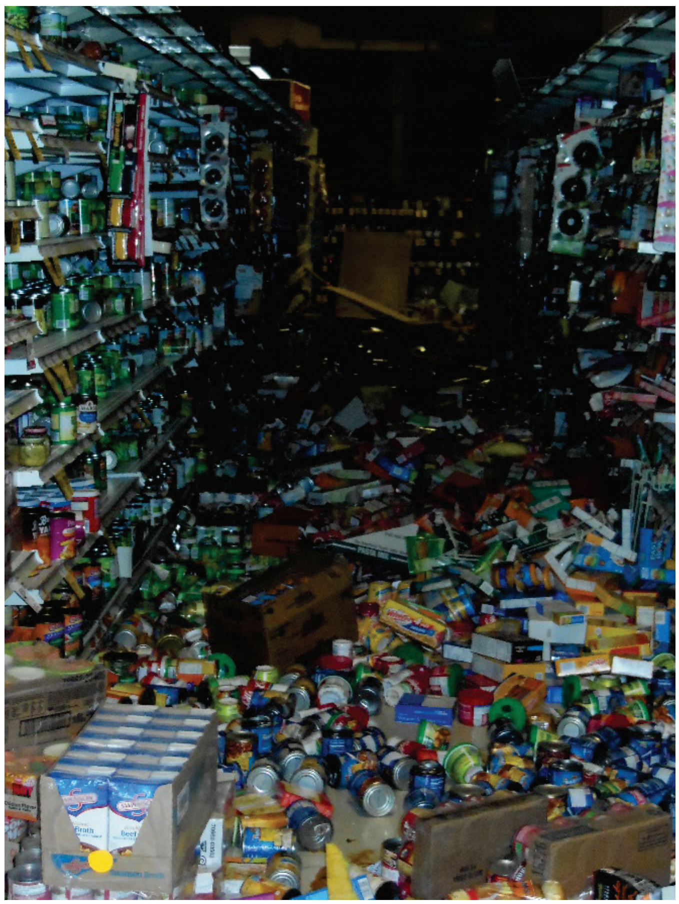


Figure 10-26 Contents dislodged from grocery store shelves.

Figure 10-27 Loss of products from shelves.

Figure 10-28 Wine storage rack in retail store that did not lose a single bottle of wine in the earthquake. The storage racks were modular units of interlocking wood and metal slats. The wine bottles sat on two metal slats, one supporting the lower bottle body and a front one supporting the bottle neck. The bottle shoulder sat below the front slat that kept the bottles from sliding out.

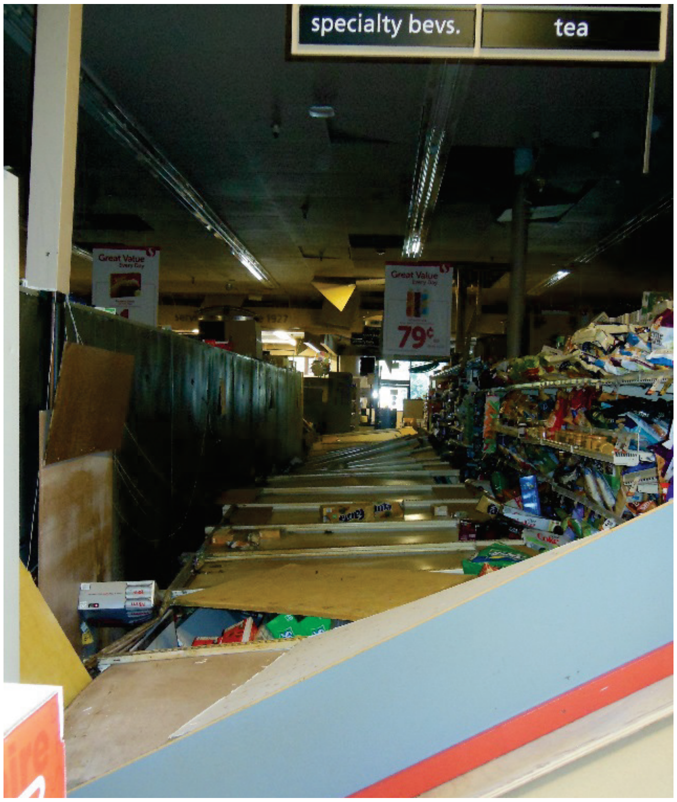


Figure 10-29   Unanchored fixtures overturned in aisle.

Figure 10-30   Overturned unanchored bookshelf.

10.8 Solar Arrays

Building code requirements for solar arrays are still under development and there has been some discussion over their content. For this reason, information was collected on the performance of different solar arrays in the area. Eight solar arrays in the Napa region were investigated. The arrays included a range of installation types: Ground-mounted parking canopies, roof-mounted parking canopies; roof-mounted low-profile arrays with ballast and mechanical attachments; and roof-mounted low-profile arrays without mechanical attachments (ballast only). The sites were located in Vallejo and Napa. Except for one of the installations, which had an eccentric baseplate connection that suffered brace buckling, all of the arrays performed well with no reported damage.

Figure 10-31 shows a variety of different solar array installations present at a medical center in Vallejo. The arrays are described in Sections 10.8.1 through 10.8.3. Ground motion data from an instrument at the medical center site is presented in Figure 10-32 for reference.

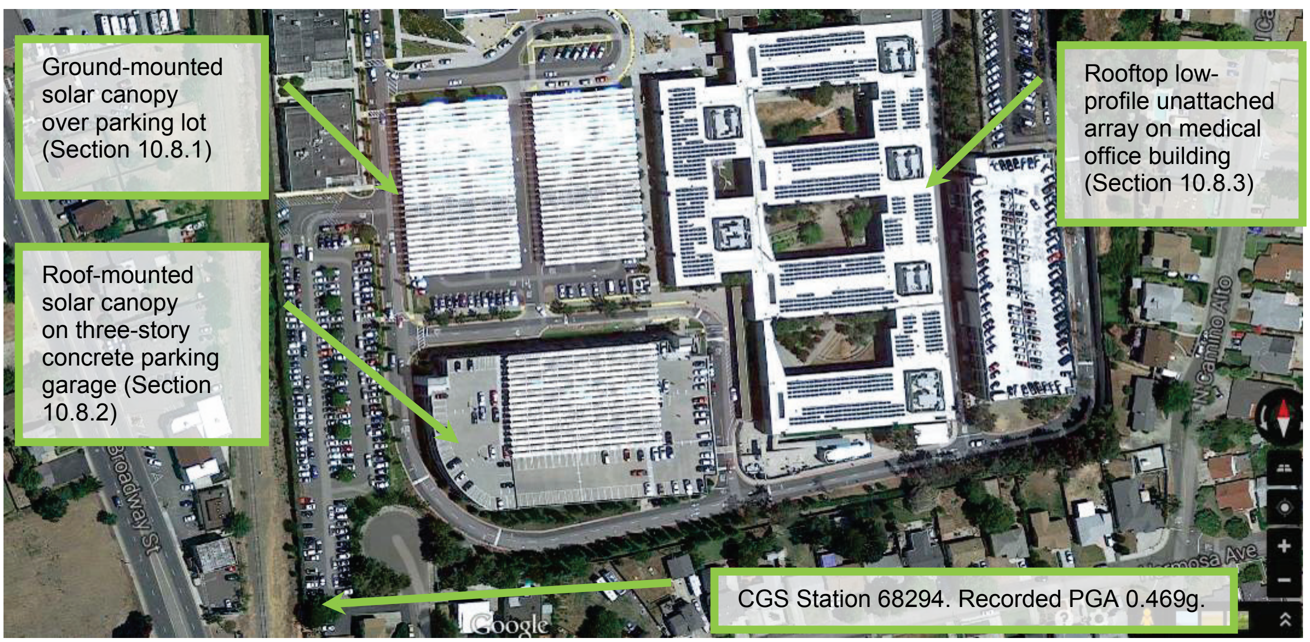


Figure 10-31 Solar arrays at medical center in Vallejo (image source: Google Earth).

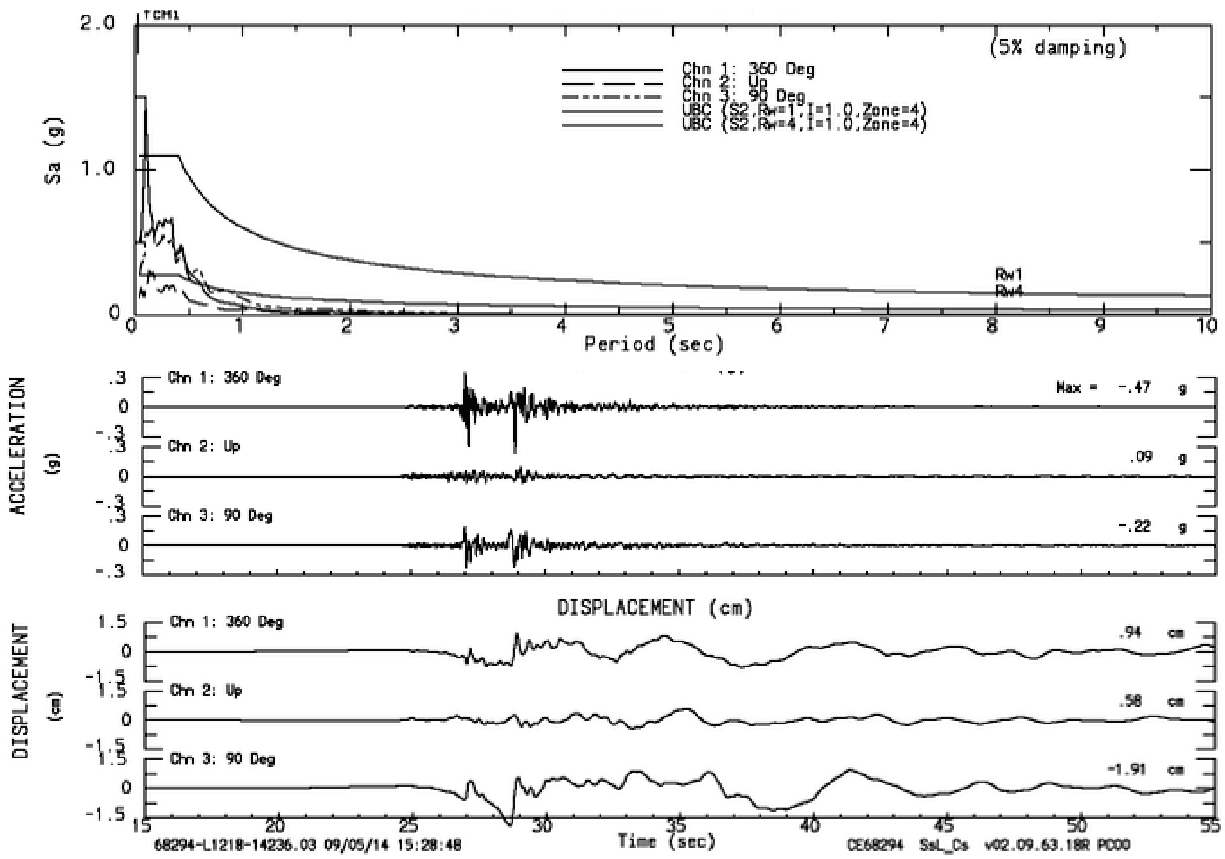


Figure 10-32 Recordings from CGS Station 68294, located approximately 300 feet from medical center parking garage (from http://strongmotioncenter.org/, last accessed September 7, 2014).

10.8.1 Ground-Mounted Solar Array, Vallejo

This array was mounted on a steel framework over a parking lot (Figure 10-33). Steel tube columns were anchored to cylindrical concrete pedestals that extend approximately three feet above grade. Wide-flange girders frame into columns and wide-flange beams run across the top of columns and girders. Rows of solar panels are located on the top of the beams. The lateral force-resisting system appears to be either an ordinary steel moment frame or a cantilever column system. There was no reported interruption in power from this array or damage to the supporting system.

Figure 10-33 Ground-mounted solar canopy over parking lot (photo from Google Street View). No damage reported.

10.8.2 Roof-Mounted Solar Canopy, Vallejo

This array consists of a steel open grid framework with multiple bays in both plan directions constructed on top of a parking garage roof (Figure 10-34). Wide-flange columns have base plates anchored into concrete grout pads on the concrete parking structure roof; anchors did not extend into the structural roof slab. Wide-flange girders frame into columns. At mid-span of the girders, wide-flange beams brace the girders against lateral-torsional buckling (Figure 10-35). Rows of solar panels are supported by members that span to the girders. Lateral resistance provided by steel tension rod X-bracing is provided at selected bays in the direction perpendicular to girders. In the direction parallel to girders, lateral resistance is provided by an ordinary steel moment frame.

The earthquake caused elongation of steel X-bracing rods (Figure 10-36), and breakout or crushing of grout around column anchor bolts under column base plates (Figures 10-37 and 10-38). The configuration of the baseplates and anchor bolts was eccentric with respect to X-bracing rods, amplifying the shear demands on the anchor bolts.

Site personnel reported that the top level of the parking garage was posted RESTRICTED USE because of damage to the array, and power from the array was shut down. The company that owns and operates the array through a power-purchase agreement obtained a permit from the City of Vallejo and performed repairs approximately two weeks after the earthquake.

Figure 10-34 Parking garage with solar canopy structure on roof (photo by Jiun-Wei Lai).

Figure 10-35 Solar canopy structure on roof of parking garage (photo by Jiun-Wei Lai).

Figure 10-36 Elongated steel X-bracing rods. Note residual drift in the shored column to the left (photo by Jiun-Wei Lai).

Figure 10-37 Possible plastic strain of steel rod X-bracing adjacent to gusset plate. Column has twisted due to eccentric configuration of rod bracing and anchor bolts. Anchor bolt breakout or grout crushing occurred beneath column baseplate (photo by Jiun-Wei Lai).

Figure 10-38 Clip angle is anchored to the concrete wall but not attached to column baseplate. Note eccentricity between the X-bracing rod and the column anchor bolts (photo by John Silva).

10.8.3 Roof-Mounted Low-Profile Unattached Array, Vallejo

The 30-foot tall medical office building supports a number of unattached, ballasted solar sub-arrays (Figure 10-39). Each sub-array consists of a grid of aluminum members, interconnected in east-west and north-south directions. The array's aluminum "feet" bear on the building's single-ply roofing. Resistance to wind uplift is provided by concrete ballast blocks stacked in aluminum pans. Ballast blocks stacked over a certain height are expected to be secured in the pans with wire or glue. Resistance to earthquake forces is provided by friction between the array and the roof surface.

It is reported that the rooftop array did not displace. No interruption in power production from this array was reported. The building itself was reported to suffer minor nonstructural damage (some drywall cracking, falling ceiling tiles, and damage to sprinkler escutcheons were reported). No structural damage to the building was reported.

Figure 10-39 Low-profile unattached (ballast-only) solar array on roof of medical office building (photo from Kaiser Permanente). No damage was reported.

10.8.4 Roof-Mounted Solar Canopy, Napa

This rooftop array on top of the parking structure on 5th Street in Napa is supported on a steel framework constructed with round HSS steel cantilever columns (Figure 10-40). Columns have baseplates with anchor bolts into the top of a concrete wall that is part of the building structure. There was no observed or reported damage and no apparent loss of power.

Figure 10-40 Roof-mounted solar canopy on top of parking garage structure. No damage was reported.

10.8.5 Roof-Mounted Low-Profile Arrays Including Ballasted and Mechanical Attached Systems

Roof-mounted low-profile arrays with solar panel support systems were field examined by a system manufacturer. The arrays use a combination of ballast and mechanical attachments to the building roof.

No damage was reported at any of the four sites visited, and there was no reported interruption of power production. The sites included two wineries, a movie theater, and an office warehouse building (Figure 10-41). Some instances of support movement (1 inch or less) were reported (Figure 10-42), and there was some evidence of load applied to mechanical attachments (Figure 10-43), but it is not clear whether the observed movement was caused by the earthquake or other sources such as thermal expansion and contraction. Table 10-1 provides a summary.

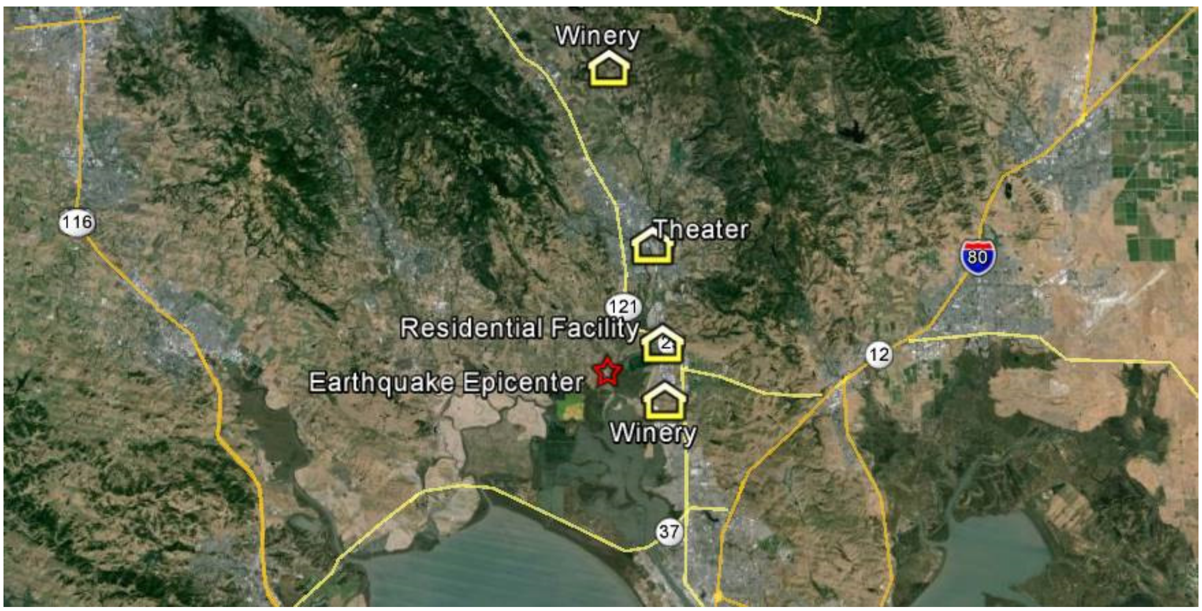


Figure 10-41 Locations of rooftop solar arrays inspected by manufacturer's engineers following the earthquake (Image from Google Earth.)

Figure 10-42 Solar array on the roof of an office and warehouse building two miles east of earthquake epicenter (photo from PanelClaw, Inc.). Array includes ballast and mechanical attachments to the building roof at selected locations. No damage reported.

Figure 10-43 Mechanical attachment of solar array on the roof of an office/warehouse building two miles east of earthquake epicenter (photo from PanelClaw, Inc.). Array includes ballast and mechanical attachments to the building roof at selected locations. Mechanical attachments are evenly distributed throughout the array and typically showed similar evidence of possible movement.

Table10-1 Summary of Low-Profile Attached Rooftop Solar Arrays

Building	Location	Height (ft.)	Construction	Roof	Modules per attachment	Roof slope	Observations
Winery	North of Napa, 12 miles from epicenter	26	Concrete tilt-up, wood-framed roof	PVC	26	Barrel shaped, mostly east-west slope	Free ends of some south supports displaced 0.5 inch or less toward east.
Theater	Near downtown Napa, 4 miles from epicenter	40	Concrete tilt-up, steel-framed roof	TPO with slip sheets	20	Nearly flat (1/2 inch per foot or less)	Some supports displaced 1.0 inch or less on top of slip sheets, generally in the same direction
Office/ warehouse	2 miles east of epicenter	25	Concrete tilt-up, steel-framed roof	Mineral cap sheet	6	Arrays cross roof ridge; modules mostly on east slope	Attachments show deformations indicating possible displacement toward north (Figure 10-42)
Warehouse / laboratory	American Canyon, 3 miles southeast of epicenter	25	Tilt-up	TPO	12	Nearly flat (1/2 inch per foot or less)	No evidence of movement

10.9 Summary

While most modern structures suffered little or no structural damage in the 2014 South Napa earthquake, several experienced significant nonstructural damage. This type of damage continues to account for the vast majority of the earthquake damage. In some cases, nonstructural damage caused buildings to be closed for six months or more.

By far the most costly nonstructural damage was caused by damage to fire sprinkler systems. The extent of damage was primarily related to three factors: (1) the time it took to turn off the water after the damage was identified; (2) the number of pipe or sprinkler head breaks; and (3) their location in the building (rooftop versus other locations in the structure). In cases where water release lasted for several hours, damage to a single pipe or sprinkler head was sufficient to flood substantial portions of the building and require building closure for several months. Most of the damage was sustained in small diameter unbraced sprinkler piping. It appears that much of the damage was caused by interaction (impact) of sprinkler heads or pipe fittings with adjacent suspended components.

Damage to cladding posed a risk to the public as adhered veneer was dislodged from several buildings and one precast panel fell from a building. In one modern building, the exterior stucco wall was severely damaged because it was not detailed to accommodate the expected story drift of the structural framing system. In another case, an entire built-up light-frame exterior wall unit detached from the building and fell on the sidewalk.

Consistent with observations in past earthquakes, unanchored equipment and inadequately restrained equipment suffered damage. The damage to rooftop equipment was generally more severe than damage to equipment in other building locations because of the higher amount of accelerations in the roof compared to other levels.

Pendant lighting fixtures fell in several retail stores and schools. The causes of failure vary from installation to installation due the variety of fixtures and connectors used. Based on the observed performance, especially at schools, it appears even modern pendant light fixture supports have low reliability. A more detailed discussion in provided in Chapter 6.

Unrestrained furnishings shifted or overturned. In some cases, these components posed safety threats. One unrestrained television that struck a building occupant on the head and caused the only death directly attributable to the earthquake.

Rooftop solar arrays performed well, except for one canopy structure on the roof of a parking garage that had an eccentric base plate connection and suffered anchor and rod failures and brace buckling.

10.10 Recommendations

The performance of nonstructural components in the 2014 South Napa earthquake suggests several areas for possible improvement and further study, including:

1. Losses associated with damaged sprinkler piping and sprinkler heads were substantial. In order to reduce related losses in future earthquakes, several aspects of fire sprinkler design and use should be investigated:

    - Immediately following an earthquake, damaged sprinkler systems should be shut off as soon as it is clear that the risk of fire following earthquake is low. Without fire protection, building owners should establish a 24-hour fire watch such that the fire department can be called if a fire develops. Since the Fire Department may be responding to fires and unavailable to turn off water, alternate approaches to safely shutting off building fire protection systems should be studied. Building owners or operators currently have the authority and ability to shut off a system. Normally, this is done by the building engineer or head of maintenance, but following the 2014

South Napa earthquake, few, if any, individuals knew how to do it. Owners and operators should designate someone on site and give them a key to the lock with the responsibility of turning off the water supply should sprinkler systems suffer damage and start releasing water. A good emergency plan designates alternates for each responsibility, including control of the sprinkler system.

- Where sprinkler heads are connected to a ceiling, use of flexible piping between the branch line and sprinkler head (sprinkler drops) may reduce the potential for damage. Shake table testing has established the effectiveness of flexible sprinkler drops.
- Threaded iron sprinkler pipe fittings provide little ductility, and have a high risk of leaking in moderate ground shaking and fracturing in strong shaking. The performance of different types of fire sprinkler piping, connections and bracing should be investigated. Recommendations for approaches to reduce the potential for pipe breaks and the probability of unintended water release should be researched and developed. Consideration should be given to investigating new technologies or approaches to improved seismic performance.
- The interaction of fire sprinkler piping with other MEP components warrants further study. The current practice of using unbraced sprinkler drops and armovers may be contributing to adverse interactions, as well as failure to specify adequate clearance between piping and other components. The role of bracing or other means to prevent damaging interactions should be explored. Also, the importance of maintaining clearances with other obstructions is critical and standard requirements need to be maintained.

2. The adequacy of code provisions for pendant lighting fixtures, especially those with multiple points of support, should be examined. Engineering analysis should be used be used to predict the performance of a pendant system. As an alternative, shake table testing should be considered as a means to validate its performance.

3. The adequacy of current code requirements for design and inspection of adhered veneer should be examined. Acceptable postearthquake performance states should be defined.

4. The adequacy of current code requirements for exterior cladding should be studied, especially for adhered foam-backed veneer. The code requirement to “accommodate” story drift should be clarified by establishing performance expectations for exterior cladding. As a

minimum, clarifying language should be considered for the commentary to the code. Engineers and building officials should ensure that cladding connections are specifically detailed to accommodate building drift and seismic demands, and field inspections should be performed to verify that the connections are properly constructed. Best practice guidance should be developed to illustrate the issues and possible solutions for various types of cladding systems.

5. Guidelines for the installation of rooftop piping and conduit should be developed. The building code does not explicitly address this issue and simple construction measures could avoid costly postearthquake repairs.

6. The adequacy of code requirements to protect glazing from damage should be investigated. This is particularly true for building systems and seismic retrofitting techniques, such as moment frames, that experience significant amounts of drift during an earthquake.

7. Some furnishings and contents not regulated by the building code pose seismic safety risks. The public should be made aware of these risks and, as a minimum, voluntary installation of seismic restraints should be encouraged, particularly for slender items, such as bookcases. Approaches for promoting a culture of seismic safety should be explored.

8. Installation of MEP equipment is often completed without inspection by a Building Official or design professional. Approaches for requiring equipment inspections during construction should be explored. The impact of damage to contents and furnishings on emergency egress should be investigated.

9. Architects, mechanical engineers, plumbing engineers, electrical engineers, fire protection engineers, information technology (IT) consultants, and others associated with nonstructural components should be trained to better understand the seismic performance implications of improperly designed or installed nonstructural components.

10. Damage to interior partitions was generally less than predicted by researchers. This is of interest in the field of earthquake damage estimation, where high levels of partition damage are predicted using current partition fragility data. The performance of gypsum wallboard and plaster partitions should be studied to better understand their vulnerability and to recommend detailing to reduce the potential for costly damage.

11. The adequacy of code design requirements can be effectively evaluated with measurements of actual floor and roof accelerations and

displacements. Installation of strong motion recording devices throughout buildings should be encouraged.

Chapter 11

Postearthquake Safety Evaluation of Buildings

11.1 Introduction

In the aftermath of an earthquake, one of the most important steps taken is the safety evaluation of the community building stock. Buildings that have been damaged and pose a potential safety threat are identified and posted to restrict or prohibit use. Of equal importance, and a vital step towards maintaining the life of the community, is identifying those buildings that are safe to occupy. Because significant events like the 2014 South Napa earthquake affect thousands of buildings, safety evaluations must be done in a rapid and efficient manner. This chapter summarizes how the safety evaluations were managed and implemented with a focus on the City of Napa where the majority of urban damage was concentrated. In general, the process of conducting the postearthquake evaluations went smoothly, and the rapid response of the local authorities facilitated the recovery of the community. As might be expected in the aftermath of significant natural disaster, some inconsistencies in the procedures occurred. In this chapter, observations of the placarding process and issues that were encountered during the postearthquake damage evaluations are discussed.

Following the earthquake, buildings were evaluated to determine if they were safe to occupy and then posted (or placarded or tagged) using the procedures in *ATC-20-1 Field Manual: Postearthquake Safety Evaluation of Buildings* (ATC, 2005). The postearthquake evaluation process in ATC-20-1 has three levels of placards defined as follows.

- INSPECTED (green): No apparent hazard is found, although repairs may be required. The original seismic resistance is not significantly decreased. No restriction on use or occupancy.
- RESTRICTED USE (yellow): A hazardous condition exists (or is believed to exist) that requires restrictions on the occupancy or use of the structure. Entry and use are restricted as indicated on the placard.
- UNSAFE (red): Extreme structural or other hazard is present. There may be imminent risk of further damage or collapse from creep or

aftershocks. Unsafe for occupancy or entry, except as authorized by the local building department.

11.2 Postearthquake Safety Evaluation Program Management

Building safety evaluations following a disaster in the United States are managed using the National Incident Management System (NIMS), a systematic tool developed by FEMA and used for the command, control, and coordination of emergency response. All aspects of the disaster response are coordinated through this system. Incidents are typically managed at the lowest possible geographical, organizational, and jurisdictional level. In instances where success depends on the involvement of multiple jurisdictions, levels of government, functional agencies, or emergency-responder disciplines, NIMS provides for effective and efficient coordination across a broad spectrum of organizations and activities. The implementation of the NIMS process in California is coordinated by the Office of Emergency Services for the State of California (Cal OES), using their Standardized Emergency Management System (SEMS) protocol.

It is also worth noting that several cities, including San Francisco and Los Angeles, have developed or are in the process of developing and implementing Building Occupancy Resumption Programs (BORP) that are based on the idea of a city deputizing licensed structural engineers to investigate buildings and develop a plan with building owners before an earthquake. Following an event, armed with prior knowledge of the facility and up-to-date building drawings and documentation, the structural engineer can immediately assess the safety of the building and allow re-occupancy or more quickly begin the shoring up and repair process. A BORP program was not in place in areas affected by the South Napa earthquake.

Following the earthquake, safety evaluations were conducted by local building department staff along with volunteers, mutual aid, and state personnel provided through Cal OES. The following sections provide a summary of how evaluations were managed.

11.2.1 Response of Cal OES

Cal OES coordinates the process of providing safety assessment program evaluators trained in postearthquake safety evaluations. Cal OES maintains a database of approximately 6,000 Safety Assessment Program (SAP) evaluators who have completed a training program based on ATC-20-1 and have registered as Disaster Service Worker (Barnes, 2014).

The earthquake occurred on Sunday, August 24, 2014 at 3:20am. In accordance with the state's SEMS protocol, later that day Napa County requested assistance from Cal OES for the City of Napa, the City of American Canyon, and the County itself; Solano County requested assistance for the City of Vallejo and the County itself. Sonoma County did not request assistance. Safety Assessment Program evaluators are state employees, local government employees such as building inspectors, and volunteers. Cal OES assessed the requests and then contacted organizations that partner with Cal OES including the Structural Engineers Association of California (SEAOC), the American Institute of Architects (AIA), the American Society of Civil Engineers (ASCE), the California Building Officials (CALBO), and the American Construction Inspection Association (ACIA).

At SEAOC, the contact for Cal OES is the chair of the State Disaster Emergency Services (DES) Committee. SEAOC has four local member organizations with parallel committees: the Structural Engineers Association of Northern California (SEAONC), Structural Engineers Association of Central California (SEAOCC), Structural Engineers Association of Southern California (SEAOSC), and Structural Engineers Association of San Diego (SEAOSD). The state SEAOC DES chair contacted the local DES chairs with the request for assistance, with a preference for SEAONC volunteers based on the size and location of the earthquake. The local chairs then emailed members in their organization who have certification and were on the volunteer list.

SEAOC provided volunteers immediately following the request and continued to provide volunteers for approximately two weeks after the earthquake. In the end, SEAONC provided about 20 volunteers, and the total number of volunteers from for the four member organizations in SEAOC was approximately 60.

After the first week, the City of Napa began requesting only CALBO personnel. CALBO responded with numerous SAP evaluators, but due to problems with communication, it became difficult to obtain SAP evaluators from CALBO over the entire ten-week span of activation. Cal OES then requested SAP resources from the California Office of Statewide Health Planning and Development (OSHPD) to complete the activation. It is noted that civil engineer SAP evaluators from the California Department of Transportation (Caltrans) also assisted in this activation.

11.2.2 Response of City of Napa

Postearthquake safety evaluations in the City of Napa were managed by the Building Department, under the direction of the Chief Building Official. The City of Napa provided status reports on the process of evaluations (City of Napa, 2014a and b) on the city's website. As of 5pm on August 25, 2014, 70 buildings were placarded as UNSAFE and the number of buildings with RESTRICTED USE placards was approaching 200. There were 60 inspectors working to evaluate structures. By September 5, 2014, there were 125 buildings posted UNSAFE and 1,036 buildings posted RESTRICTED USE. For several months following the earthquake the City of Napa periodically posted a GIS map of structures with UNSAFE and RESTRICTED USE placards on a website (this website is not available anymore).

11.2.3 Response of City of Vallejo

Safety evaluations in the City of Vallejo were managed by the local Building Department. As of September 17, 2014, 404 structures had been posted RESTRICTED USE, and 34 structures had been posted UNSAFE. In total 1,075 properties were inspected (City of Vallejo, 2014).

11.2.4 Response of the Office of Statewide Health Planning and Development

Safety evaluations for healthcare facilities were managed by OSHPD, which has a statutory responsibility to perform postearthquake safety evaluations. Details on healthcare facility building evaluation efforts are included in Chapter 5. OSHPD also responded to mutual aid requests from the cities of Napa and Vallejo. They also responded to a request from three state-operated facilities that provide long-term care for veterans, individuals with developmental disabilities, and individuals mandated for psychiatric treatment by a criminal or civil court judge. The following placards were posted:

- Veterans Home of California in Yountville: Six INSPECTED placards.
- Napa State Hospital: This older facility provides mental health treatment. One UNSAFE placard; eight RESTRICTED USE placards; and 49 INSPECTED placards. The UNSAFE placard was posted on an unoccupied unreinforced masonry building. RESTRICTED USE placards were generally posted on structures with potential damage to masonry chimneys, damage to structural walls or bracing
- Sonoma Developmental Center: 21 INSPECTED placards.

At the peak, OSHPD provided 30 staff members in response to mutual aid requests.

11.2.5 Response of the Napa Valley Unified School District

Safety evaluations for schools were addressed by several agencies. K-12 schools in California are regulated by the Division of State Architect (DSA). Unlike OSHPD, DSA does not have regulatory authority to inspect schools after an earthquake unless there is a local request. Local requests were not made, so DSA structural engineers did not review school buildings after the event (Barnes, 2014; Turner, 2014).

In the Napa Valley Unified School District, district officials set up teams of a district official, a local architect, and a local structural engineer. The school district also retained mechanical and electrical engineering consultants to inspect each school site and recommend repairs, where needed. Evaluations began on the day after the earthquake. Details on building evaluation efforts are included in Chapter 6.

11.3 Conduct of the Postearthquake Safety Assessment Program

While all of the jurisdictions followed the general procedures of ATC-20-1 for their building evaluations, there were variations in the application of the methodology, which are discussed in this section.

11.3.1 Use of Evaluation Forms

There were some differences observed in how the ATC-20-1 safety placard system was used by different jurisdictions in this event. In some cases, LIMITED USE (Yellow) placards were issued for buildings not necessarily to limit access, but apparently more to document damage to ensure that repairs would be made. In these cases, the placard did not list any access restrictions but instead appeared to be used as an administrative tool to inform the owner that repairs of the documented damage were needed before the placard would be removed. This prevented people from occupying the building without the proper repairs having been made.

In Vallejo, the placards were based on the ATC-20-1 forms (with the City of Vallejo logo added), but the ATC-20-1 assessment forms were not used. Instead, City of Vallejo inspectors used simplified forms that were not necessarily consistent with the placard. These were revised as the process proceeded until the assessment form began to look similar to the ATC-20-1 assessment form.

11.3.2 Assignment of Resources

In both Napa and Vallejo, structural engineers were often assigned to evaluate single-family residences, rather than the larger, more complicated urban structures.

Although ATC-20 evaluations can be conducted by anyone with proper training and registration in the State's program, using the limited resource of available experienced structural engineers for more complicated structures is recommended. ATC-20-1 presents three evaluation procedures: Rapid, Detailed, and Engineering. The first two are used as part of the postearthquake assessment process. The proportion of Rapid versus Detailed Evaluations conducted is not known, but it is assumed that the vast majority were Rapid Evaluations. ATC-20-1 recommends that Detailed Evaluations be conducted by structural engineers.

11.3.3 Compensation of Safety Assessment Program Evaluators

Initially, the local jurisdiction requesting assistance is responsible for compensating Safety Assessment Program evaluators. Eventually, if a formal disaster declaration is made, the local jurisdiction can make a request to recoup the costs of compensation. Volunteer evaluators, such as those from SEAOC or AIA, donate their time, but travel and lodging expenses are reimbursed by the city requesting the help. Compensation for government employees, such as local building officials from other jurisdictions or agencies, serving as safety assessment program evaluators are provided through mutual aid. This is a significant difference, so initial requests are usually aimed at volunteer evaluators.

11.3.4 Local Ordinance Authorization for Placarding

In theory, a local community should have an ordinance that formally authorizes the postearthquake safety evaluation process and use of placards. It was reported that the City of Vallejo had such an ordinance, but the City of Napa only had a model ordinance (not enforced) (Barnes, 2014). Placards typically also carry the logo of the local jurisdiction. In the City of Napa, placards were observed with and without the City of Napa logo, though they matched the forms in ATC-20-1.

11.4 Observations

The postearthquake safety evaluation process generally appeared to meet the goals of providing rapid information on whether buildings in the affected area were safe to occupy. Using GIS technology, the status of the postearthquake inspections in Napa was accessible to the public online. In general, the process went smoothly, and buildings were evaluated in a timely manner. Some issues did arise, especially following the initial safety assessments when buildings were being reevaluated, and are discussed in this section.

11.4.1 Multiple Postings

Many buildings in Napa received follow-up evaluations and the buildings were reposted with a different placard. However, in some cases, the superseded placard was not removed. Figure 11-1 shows an example of an UNSAFE placard placed over an INSPECTED placard. The INSPECTED placard had the City of Napa logo; the UNSAFE placard which came later did not. This could cause confusion over which placard takes precedence.

ATC-20-1 notes that there can be only one posting classification for a building and that all entrances should receive the same placard. Some buildings in Napa were observed to have an INSPECTED placard at one entrance and a RESTRICTED USE placard at another entrance. Those entering through the entrance posted INSPECTED thus may not have been aware of the restrictions in areas of danger.

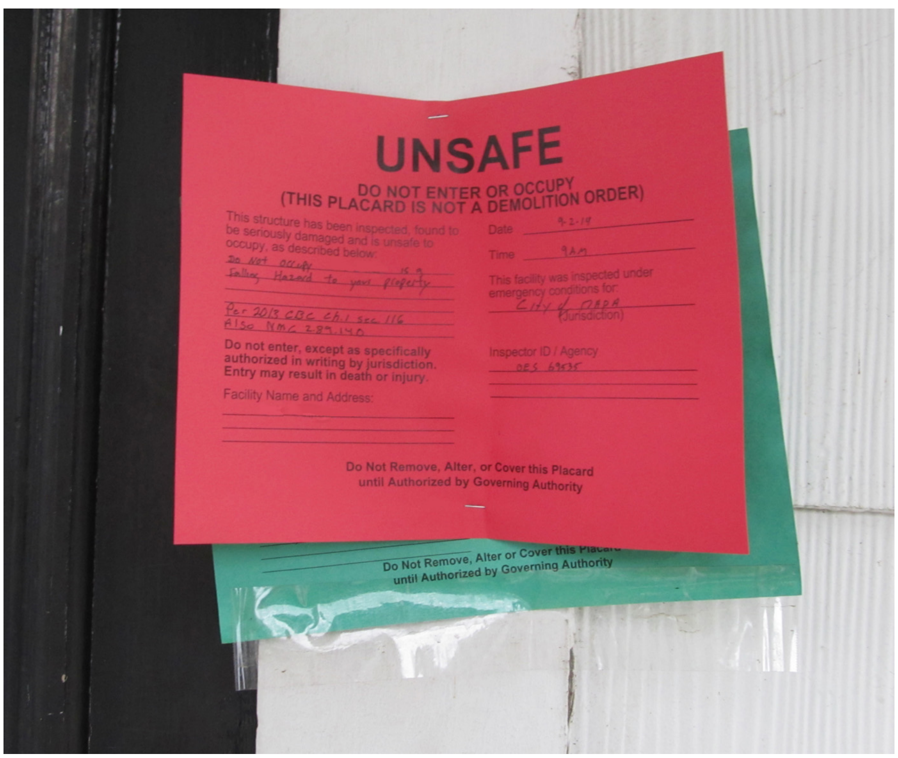


Figure 11-1 A building with multiple placards.

11.4.2 Inappropriate Use of Placards

The purpose of the RESTRICTED USE placard is to restrict use to local areas of the building with significant damage. The restrictions need to be documented on the placard so that building occupants understand the limitations. Figure 11-2 shows an example of a proper use of the RESTRICTED USE placard at a building where an area with falling hazards had been taped off and access was restricted to selected entrances.

Figure 11-2 A RESTRICTED USE tag with restrictions noted as "Public may enter. Stay clear of taped off areas and construction."

However, a number of RESTRICTED USE placards had minimal or no information on the restrictions (Figure 11-3 shows an example), reducing the effectiveness of the placard.

Figure 11-4 shows an INSPECTED placard that was placed on the rear entrance of a building where access to the front entrance was restricted due to a significant falling hazard. A RESTRICTED USE placard should have been placed at both entrances clearly identifying the restrictions.

Figure 11-5 shows a RESTRICTED USE placard on a restaurant with falling hazards. Outdoor seating adjacent to the placards was restricted, "except during the following times: (1) Monday – Friday, 4:30pm to closing, (2) Saturday to Sunday – All Day." The restrictions should never relate to when the restaurant is open, but rather to the hazard itself. Placing exceptions for specific times is inappropriate and in this case potentially endangers patrons during the busier times.

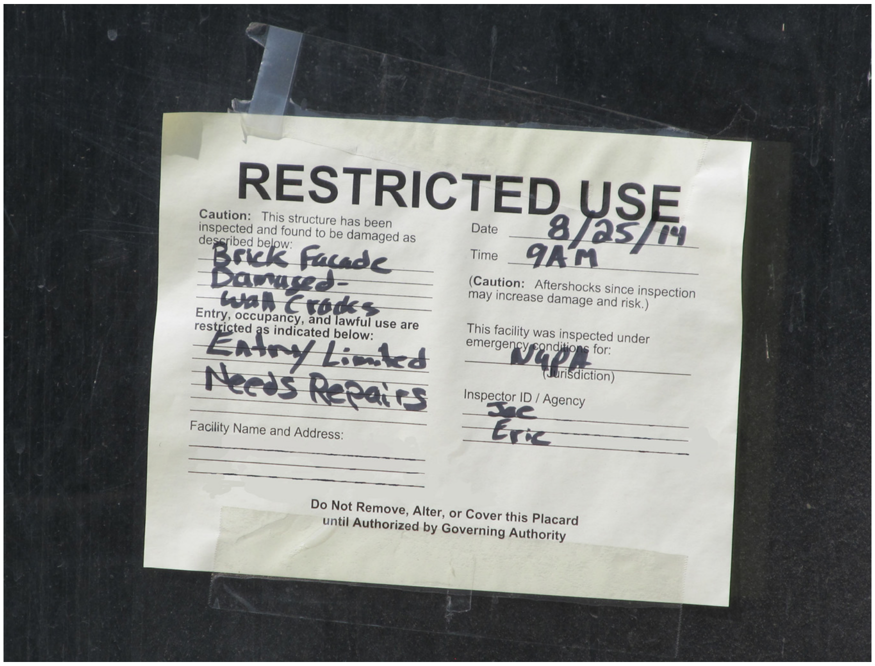


Figure 11-3 A RESTRICTED USE placard with no information on the restrictions.

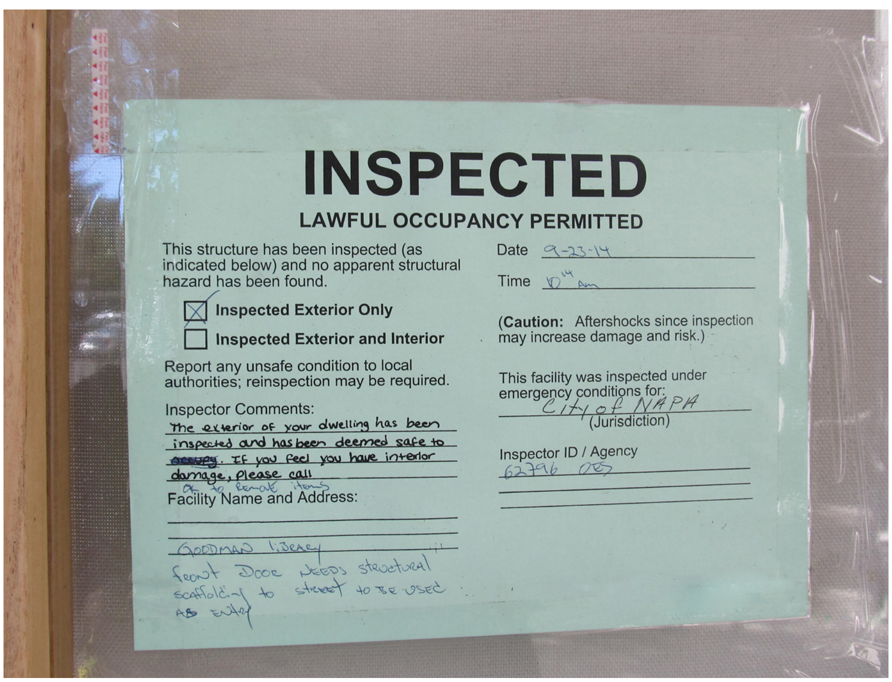


Figure 11-4 An INSPECTED placard on a building with another exit that is still restricted. A RESTRICTED USE placard should have been used.

Figure 11-6 shows a building with an UNSAFE placard with a separate sign from the landlord implying that the landlord could grant permission to the space despite the UNSAFE placard. As the UNSAFE placard indicates, entry is not permitted except in writing by the local jurisdiction, not the owner.

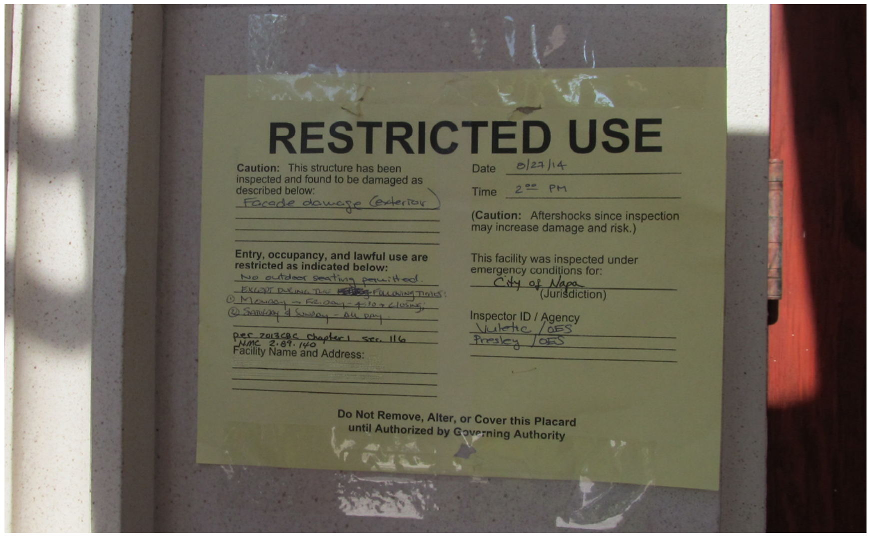


Figure 11-5 A RESTRICTED USE placard with exemptions permitting access while the restaurant is open.

Figure 11-6 An UNSAFE placard with an adjacent inconsistent placard by the owner.

11.4.3 Inconsistent Communication with the Public Regarding the Meaning of the Placards

The public often has questions about the meaning of the placard language, and what steps they should take after receiving a placard. A website was established to provide information relevant to the City of Napa and Napa County, http://www.napaquakeinfo.com/tag-information.html (accessed November 25, 2014). The website states that for the City of Napa, the "tags" (placards) have the following meanings:

- Red Tag: A red tag means the building structure has been damaged and is not safe to enter. Please do not enter your building.
- Yellow Tag: A yellow tag means "cautionary." Property owners can access and clean their buildings, and once clean can contact the City for re-inspection.
- Green Tag: A green tag means that either the building has not been affected or has slight damage. The building is structurally safe to enter.

For Napa County, the website states that the tags have the following meanings:

- Red Tag: A red tag means the building structure has been severely damaged and it is no longer safe. Please do not enter the building.
- Yellow Tag:
    - A yellow tag means that the building has been moderately damaged. Use of the building is limited.
    - Residents may continue to live in a home that has a yellow tag, but they may have to avoid damaged areas.
    - Commercial properties may be entered for the purposes of clean-up and repair, but may not be opened to the public.
    - The yellow tag cannot be removed by the County until the damage has been repaired and passed inspection.
- Green Tag: A green tag means that either the building has not been affected or has slight damage. The building is structurally safe to enter.

The information for UNSAFE (red) and INSPECTED (green) placards is consistent with ATC-20-1 for both jurisdictions. However, for both jurisdictions, the information for the RESTRICTED USE (yellow) is not consistent with ATC-20-1. According to ATC-20-1, all restrictions on use should be noted on the placard and should indicate areas that are not safe for entry. The information for Napa County is more nuanced, but the prohibition of public access to a commercial building with a RESTRICTED USE placard may be overly restrictive. As noted above, the placard should identify the areas that are restricted and those that are not.

The City of Vallejo's website (http://vallejo.hosted.civiclive.com/cms/one.aspx?objectId=99620, accessed December 1, 2014) provided the following information:

- Red Tag: A red tag means the building structure has been damaged and is deemed unsafe to enter and occupy. Please do not enter your building.
- Yellow Tag: A yellow tag means "restricted use" or "limited access." Property owners can access, clean, remove possessions and abate imminent hazards in their building, and once clean can contact the City for re-inspection.
- Green Tag: A green tag means the building has not sustained structural damage or has minor damage. The building is structural safe to enter.

The information for UNSAFE (red) and INSPECTED (green) placards is consistent with ATC-20-1. The information for the RESTRICTED USE (yellow) is not. Again, all restrictions on entry and use should be noted on the placard.

11.5 Summary

In the aftermath of the 2014 South Napa earthquake, each jurisdiction responded to the need to evaluate the safety of buildings in the impacted region, using the resources they had available. In general, the process went smoothly, and buildings were evaluated in a timely manner. However, there were significant variations in evaluation and placarding procedures among the different jurisdiction. The definitions provided to the public by jurisdictions on the meaning of the RESTRICTED USE placard were inconsistent with the intent of the ATC-20-1 procedures, and varied significantly by jurisdiction.

In some cases, the placarding of the buildings was not clear, with current and superseded placards posted on the same entrance. This could present a potentially hazardous condition, because in several instances the building was initially posted INSPECTED (safe to occupy) and subsequently posted UNSAFE. Postings at different entrances of a building were not always consistent, and in some cases, a person entering through one entrance would be unaware of a hazardous condition in the building, because there was a different placard at the door they entered.

11.6 Recommendations

A number of areas for improvement in the postearthquake damage evaluation program were identified, including:

1. Use of the ATC-20-1 evaluation forms and procedures as written is strongly recommended. The evaluation procedures are the product of extensive development, testing, and experience and they should not be modified or abridged.
2. A document should be developed to provide guidance to local jurisdictions on how to provide effective management of the postearthquake safety evaluation process and incorporate common issues and lessons learned from those who have been directly involved in the process. This document could include discussion and recommendations on:
   - Appropriate use of evaluation forms, together with placards.
   - Best practices for assigning safety assessment program volunteers with varying degrees of experience and training. While ATC-20-1 evaluations can be conducted by anyone with proper training and registration in the State's program, using the limited resource of available experienced structural engineers for more complicated structures is recommended. ATC-20-1 recommends that Detailed Evaluations be conducted by structural engineers.
   - Quality assurance techniques should be outlined to provide consistent evaluation and placarding decisions and to reduce misuse of placards.
   - Recommendations on implementing language for local ordinances to formally authorize the evaluation and placarding process.
3. A document should be developed to provide guidance for communities in how to communicate the placard requirements to citizens. Common misunderstandings, particularly related to the RESTRICTED USE placard, should be discussed.
4. Placarding should be consistent for each building, and copies of the placard should be placed at all entrances. No building should have different placards at different entrances.
5. Only the most current placard should be posted. Superseded placards should be removed, and all entrances to the building should be posted with the most current placard.

6. RESTRICTED USE placards should never include provisions that permit the public to be exposed to hazardous conditions based on the time of day. Public use should be prohibited until the hazardous condition in that location is mitigated.
7. Postearthquake evaluation training should be updated to address the commonly observed issues as noted above. Building Occupancy Resumption Programs should be considered for supplementing limited city resources and facilitate postearthquake recovery.
8. It was observed that because there is no requirement to pay the salaries of volunteer evaluators (as opposed to building officials from other jurisdictions paid through mutual aid), initial requests were typically aimed at volunteer evaluators. Wider discussion of the appropriateness of this situation is worth discussing in the building community.

Chapter 12
Barricading of Unsafe Areas

Following the 2014 South Napa earthquake, barricades and fencing were installed around damaged buildings to protect pedestrians, traffic, and adjacent buildings. This chapter summarizes selected observations related to barricades and fencing and related guidelines and codes.

12.1 Available Resources

ATC-20-1, *Field Manual: Postearthquake Safety Evaluation of Buildings*, (ATC, 2005) provides limited guidance on barricades but does not provide details of design and location of barricades and fencing.

Following review of the 2010 and 2011 earthquakes in Christchurch, New Zealand, the development of guidelines for barricades and cordons was identified as a high priority. As a result, the volunteer members of California Building Officials (CALBO) developed the *Interim Guidance for Barricading, Cordoning, Emergency Evaluation and Stabilization of Buildings with Substantial Damage in Disasters* (CALBO, 2013). The *Guidelines* note that requirements in *California Building Code* Chapter 33, Safeguards During Construction, apply only to stable buildings under construction and not unstable, damaged buildings. The *Guidelines* recommend initial placement of soft barriers, such as fencing, at horizontal distances (H) up to 1.5 times the height (V) of the façade or structure at risk, termed a 1.5H:1V setback, "to allow for the possibility that falling items can bounce and shatter." Wide safe distances including block-long cordons are warranted until inspectors, Safety Assessment Program (SAP) evaluators, building owners, engineers, contractors, and other agents can evaluate, stabilize, or remove potential falling or collapse risks and erect hard, impact-resistant barriers to protect the public.

It is noted that the G*uidelines* are permissive and can be adapted to the unique circumstances of every damaged structure. For example, a 7-story 100-foot tall building with a damaged appendage at the second floor need not be barricaded to 150 feet out, whereas for a unreinforced masonry (URM) building with a damaged parapet at the roof level, the full setback of 1.5 times the building height would be appropriate. For some multi-story buildings, the setback distance may be larger than the width of the sidewalks and street in front of the building. In cases perpendicular to the street or for rear walls, there may be adjacent buildings abutting the damaged building with no separation. To achieve the required setback distance, the adjacent building would have to be cordoned off, as well. The *Guidelines* note that the locations and types of barriers, barricades, and cordons can be refined as situational awareness improves during recovery.

Another available resource is the *Safety Assessment Program Coordinator Student Manual* (Cal OES, 2013), which discusses issues related to cordoning and barricades.

12.2 Observations and Issues

Damage in the downtown business district of the City of Napa was substantial, and barricading and fencing were an important aspect of the recovery process. It appears that there were at times two conflicting priorities: (1) barricading around damaged buildings to protect the public from falling debris, especially in an aftershock, and (2) keeping the streets open so that owners and customers could still reach those businesses that were able to remain open. Given that the primary economy of the downtown Napa area is tourism, it is understandable that the City did not want to cause further economic hardship by closing off entire streets. However, safety must remain a top priority. It is fortunate that the number and size of aftershocks were much smaller than expected.

Based on field visits and discussions with others, several issues with barricades were identified.

12.2.1 Initial Barricade Distances

In some cases, it was observed that initial barricades were inadequately spaced from damaged buildings. Figure 12-1 shows an unreinforced masonry (URM) building with an UNSAFE placard and significantly damaged masonry façades, with a fence only several feet from the building (much less than the 1.5 times the vertical height), with the street allowed to remain in full operation.

Figure 12-2 shows a church with a gable wall separated from the roof diaphragm where the fence was set back some distance into the street, but less than the 1.5H:1V setback threshold.

There were some streets that were closed in downtown Napa due to the falling hazards from damaged buildings, but typically only the sidewalks and portions of the street were fenced off, usually at a distance of much less than 1.5 times the height of the falling hazard. Figure 12-3 shows a street that was partially closed due to damage of the adjacent building. Prior to removal of the damaged roof over the cupolas, the entire street had been closed.

There were also some adjacent buildings where access was restricted due to damage in the adjoining structure though the buildings were not cordoned off.

Figure 12-1 A building with an UNSAFE placard where the fence setback was much less than a 1.5H:1V setback distance.

Figure 12-2 Church with damaged gable façade where the fence setback was less than a 1.5H:1V set back distance.

Figure 12-3 Partial street closure. Prior to removal of the damaged roof cupola, the entire street was closed.

12.2.2 Transitions to Longer-Term Barricades

As time passed and damage could be better evaluated, the barricades and fencing strategies evolved. A common approach while repairs were being made was to install scaffolding in front of the damaged building façades. To limit the visual impact, plastic fencing was usually part of the installed scaffolding (Figure 12-4). Reportedly, scaffolding was not engineered and not anchored or attached to the building it was barricading. Although it is likely that the scaffolding and screening would stop small pieces of masonry and other cladding materials from falling outward, given the relative mass of many of the damaged structures with URM façades, it is not obvious that the scaffolding would be effective in preventing a failing wall from falling on the sidewalk or street.

In at least one case, a windstorm damaged a lightweight scaffolding system placed adjacent to a damaged URM side wall (Figures 12-5 and 12-6). The lightweight system was later replaced with a more robust scaffolding system (Turner, 2014).

To keep bystanders from breeching a fenced area, some of the longer-term barricades used smooth plywood walls of around eight feet in height that could not be easily scaled without a ladder.

Figure 12-4 Scaffolding used as a barricade on damaged URM building.

12.2.3 Exiting into Cordoned Areas

In some instances, buildings with RESTRICTED USE placards had certain exits below damaged masonry gable walls. Yellow caution tape was used around the area (at far less than the 1.5H:1V distance) and the door had a RESTRICTED USE placard on the outside. However, there was no warning sign on the interior of the door, and staff was observed taking breaks within the cordoned area using the exit. Since the building remained in use as a restaurant, diners and staff in an emergency might use this exit and walk directly into a potential falling hazard zone from the damaged gable above.

Figure 12-5 Scaffolding used as a barricade damaged in a windstorm (photo by Fred Turner).

Figure 12-6 Replacement scaffolding following the windstorm (photo by Fred Turner).

12.2.4 Protecting Adjacent Buildings

In one instance, an unretrofitted URM building was severely damaged and posted UNSAFE as a result of damage to brick walls. There were four shorter undamaged buildings immediately adjacent to it that were also posted UNSAFE due to risk of collapsing masonry walls striking their rooftops (Refer to Section 4.4.1). In order to allow occupancy in the adjacent buildings, the City of Napa required barriers to be constructed over the adjacent rooftops to prevent any falling bricks from penetrating the roofs and posing a safety risk to building occupants (Figure 12-7). With the protective barriers in place, the adjacent buildings were reposted INSPECTED and were opened for business.

12.3 Summary

In the period immediately following the earthquake, damage evaluations are performed rapidly, and because they are based on limited information, they are expected to be conservative. Local officials have to decide between two contending priorities: to protect the public from the hazards posed by damaged buildings while minimizing the impact of the earthquake on homes and businesses, by allowing the public access to undamaged structures as soon as possible.

The general guidelines for barricading and fencing of damage buildings that were available at the time of the South Napa earthquake are permissive, and substantial variations in the fencing and barricading distances were observed following the earthquake. For buildings with parapet and façade damage,

Figure 12-7 Protective barrier constructed over the rooftops of buildings to protect from falling brick.

which were fairly widespread in the downtown area, the recommended setbacks for the fences were rarely enforced. No hard, impact-resistant barriers were installed. The setbacks were typically determined by an assessment of the falling hazard risk posed by the structure, or by other criteria, such as the desire to maintain or reopen rigths of way, minimize disruption, and speed recovery. In some instances, exit paths for buildings passed under damaged façades, even though from the exterior, the area was cordoned off. So it appeared that the *Guidelines* were not fully implemented and the public was not uniformly protected from falling hazards in aftershocks. Furthermore, trained SAP Coordinators were not involved with ensuring that barricades recommended by SAP personnel were effectively erected.

12.4 Recommendations

The experience with barricading damaged buildings in the South Napa earthquake suggests several areas for possible improvement and further study, including:

1. The effectiveness of scaffolding and other types of barricades in providing life safety protection against various types of falling hazards is not well understood. A research project is recommended to compare the effectiveness of various forms of scaffolding and barricading against different falling hazards. This could include smaller and larger masonry

elements from different heights. Guidance should be provided to assist engineers with design of barriers to protect against damage posed by falling masonry.

2. Further development of consensus guidelines, such as *Interim Guidance for Barricading, Cordoning, Emergency Evaluation and Stabilization of Buildings with Substantial Damage in Disasters*, into a formal document is recommended and should involve various stakeholders and professionals with relevant expertise. This work should include discussion about protection not just of the public way, but also adjacent buildings in potential danger from damaged structures.

3. The local authority having jurisdiction should establish criteria for the placement of barricades and fencing around damaged buildings that allows for some flexibility based on easily identifiable conditions. This would establish baseline requirements that could be followed consistently following an earthquake.

4. Where occupancy is granted for a structure that has damaged elements posing as falling hazards, there should be follow-up site visits by the local building or fire authorities to confirm that limited access and barricading requirements are being followed.

Chapter 13
Summary and Conclusions

The August 24, 2014 South Napa earthquake struck early in the morning, which greatly reduced the potential for serious casualties. Had the earthquake struck 12 hours earlier during a downtown street festival, this would have been a far different story. No building sustained complete collapse, but a number of older structures experienced partial collapse or serious structural damage. Falling masonry from damaged buildings in downtown Napa would have posed a serious threat to life.

Damage to nonstructural components was by far the greatest contributor to property damage. Secondary damage due to failures of water piping and fire sprinkler systems was serious, often resulting in extensive losses to buildings that otherwise suffered only minor damage. Although of only moderately strong intensity, the South Napa earthquake provides a significant opportunity for study of building performance and the effectiveness of seismic hazard mitigation efforts.

Performance data were collected and analyzed systematically for buildings located in the vicinity of strong-motion recording instrument USGS NCSN Station N016 at Main Street in Napa. This is an effective means of rapidly collecting building performance data and should be implemented after every significant earthquake.

Buildings constructed to recent codes generally performed well structurally, although many buildings suffered some nonstructural damage. Older structures not of unreinforced masonry (URM) construction performed well structurally, although known vulnerabilities did in some cases result in significant damage and loss of use.

Buildings of URM construction make up 40% of the buildings surveyed within 1,000 feet of instrument N016, and over two-thirds of the URM buildings had been seismically retrofitted prior to the earthquake. Based on the performance of these buildings, the URM risk mitigation efforts in Napa were successful in reducing damage and protecting life safety. Unretrofitted URM structures continue to pose significant risks to the public and efforts should be made to reduce the risks posed by these buildings.

Although the hospital nearest the epicenter remained open and functional, it did highlight some of the challenges associated with sustaining hospital operations following an earthquake. Both new and older hospital buildings experienced some nonstructural damage which, in a larger earthquake with longer duration, could have impacted the continuity of service.

Schools performed well structurally, but they experienced nonstructural damage that could have been life threatening had the earthquake occurred during school hours. The earthquake highlighted the danger posed by pendant light fixtures, unrestrained bookcases, storage units, and similar components that overturned in the earthquake, often striking tables and desks. These items are generally not covered by the building code, and a special effort will be needed to reduce the hazards posed by these items. Classrooms should be examined on an annual basis to ensure that furnishings and contents are properly anchored and braced. Such an inspection program could be tied to the annual ShakeOut drill that almost all schools now perform. All schools, not just public schools, should implement measures to enhance seismic safety.

The overwhelming majority of residences affected by the South Napa earthquake suffered little damage. The performance of single-family residences is consistent with the prior observations that light-frame residential construction generally performs well, provided that known hazardous conditions are either not present or have been mitigated. Continued efforts to increase the awareness of the public of the hazards presented by masonry chimneys and unbraced cripple walls are needed. The fault rupture passed through some residential neighborhoods, damaging homes that otherwise would be expected to perform well, and the afterslip observed in the days after the earthquake continues to be an issue.

Manufactured homes with earthquake bracing systems appeared to perform no better than those lacking these systems. The performance objectives of these earthquake bracing systems should be studied to improve performance.

Approximately 50 wineries were exposed to significant seismic ground shaking in the South Napa earthquake. Direct loss of wine and damage to the production equipment varied by location, but was relatively low for the Napa Valley as a whole. The timing of the earthquake relative to the grape harvest contributed to the low losses. Much of the wine loss that did occur was attributed to collapse of wine barrel stacks. Given the hazard posed by collapsing barrel stacks, criteria for unanchored barrel storage should be established. The exposure of the public and winery guests to the wine barrel stacks should also be strictly limited.

Nonstructural components and systems in most buildings surveyed were damaged to some extent, in some cases resulting in loss of building use for an extended period. Of concern was serious damage to exterior curtain walls in modern structures, an indication that building code requirements for accommodation of story drift are not being effectively applied. Pressurized piping system failures, especially fire sprinkler systems, caused significant water damage even though the actual number of piping failures was relatively small.

The process of evaluating and posting buildings went smoothly following the earthquake. There were some significant variations in evaluation and placarding procedures among the different jurisdictions, especially with regard to the meaning of the RESTRICTED USE placard. The definitions provided to the public by jurisdictions should be consistent with the intent of the procedures set forth in ATC-20-1, *Field Manual: Postearthquake Safety Evaluation of Buildings*, (ATC, 2005). Use of the ATC-20-1 evaluation forms and procedures as written is strongly recommended.

Once damaged buildings were identified, local officials took steps to protect the public from potential falling hazards posed by these buildings, balancing the need to protect the public from falling hazards while minimizing the impact of the earthquake on undamaged homes and businesses. Additional research is recommended to establish the effectiveness of various forms of scaffolding and barricading against different falling hazards, and provide guidance for engineers who design barriers to protect against damage posed by falling masonry.

Every significant earthquake provides opportunities to assess the performance of buildings, reevaluate the effectiveness of earthquake risk mitigation efforts, and review the effectiveness of post-disaster response procedures. The 2014 South Napa earthquake demonstrated the vulnerability of URM buildings to damage in moderate ground shaking. It also highlighted that even in areas of high seismic risk where there is elevated awareness of that risk in the design and construction communities, moderate earthquakes cause costly and disruptive damage. The losses in modern structures were largely due to avoidable nonstructural damage. Attention to detailing for story drifts, installation of piping systems to avoid impact with other components, and contingency plans to allow for timely shut-down of damaged piping systems would have mitigated much of the observed damage, and none of these actions would have materially added to the cost of construction. Continued efforts are needed to inform and educate building owners and the design and construction communities on cost-effective ways to reduce earthquake damage and improve the resiliency of our communities.

Appendix A
Survey Forms and Instructions

This appendix presents the forms used in the 1,000 foot survey around Station N016. The forms are based on the survey forms presented in ATC-38 report, *Database on the Performance of Structures near Strong-Motion Recordings: 1994 Northridge, California, Earthquake*, (ATC, 2000).

Based on the feedback received from initial investigations on the field, the ATC-38 forms were modified to account for new knowledge and specific issues related to this event. In addition, some portions of the instructions were clarified to improve the consistency of the data collected.

A.1 Modifications to Survey Form and Instructions

A General Damage Classification for nonstructural components was added on the page 1 of the form. In addition, two new levels were introduced to the General Damage Classifications (to apply to both structural and nonstructural components):

- An intermediate General Damage Classification was added between the "insignificant" and "moderate" levels. This new damage classification, "minor," represents a state where structural or nonstructural damage have occurred but can be repaired relatively easily, without significant disruption to the occupants. Examples include repairs to equipment anchorage and supports, repair of minor cracking in concrete or masonry, or replacement of loosened anchors. It also includes damage to URM parapets, where the work does not require removal and reconstruction, and restrictions on occupants are limited to restricted access in the immediate vicinity of the work.
- A "collapse" classification was added to describe partial or complete loss of gravity support.

Table A-1 provides the revised General Damage Classifications.

A new level, "0," was added to ATC-13 Damage State definitions for conditions where the percent damage to the structure or nonstructural components cannot be determined. Table A-2 provides the ATC-13 Damage State definitions used by the investigator teams on the field.

Table A-1 General Damage Classifications

Code	Description
(N) None	No damage is visible, either structural or nonstructural.
(I) Insignificant	Damage requires no more than cosmetic repair. No structural repairs are necessary. For nonstructural elements this would include spackling partition cracks, picking up spilled contents, putting back fallen ceiling tiles, and repositioning equipment and furnishings.
(m) Minor	Minor repairable structural or nonstructural damage has occurred. Repairs can be made without significant disruption to occupants. This damage state includes cracked or dislodged masonry requiring repair.
(M) Moderate	Repairable structural damage has occurred. The existing elements can be repaired in place, without substantial demolition or replacement or elements. For nonstructural elements this would include minor replacement of damaged partitions, ceilings, contents, or equipment.
(H) Heavy	Damage is so extensive that repair of elements is either not feasible or requires major demolition or replacement. Includes URM buildings that require partial or complete reconstruction of damaged masonry walls. For nonstructural elements this would include major or complete replacement of damaged partitions, ceilings, contents, or equipment.
(C) Collapse	Partial or complete loss of gravity support with collapse.

Table A-2 ATC-13 Damage State Definitions

Damage State	Percent Damage (damaged value/replacement value)
0 Unknown	Unknown
1 None	0%
2 Slight	0% - 1%
3 Light	1% - 10%
4 Moderate	10% - 30%
5 Heavy	30% - 60%
6 Major	60% - 100%
7 Destroyed	100%

Additional modifications included the following:

- Checkboxes were added for identifying the building as a candidate for further study of FEMA methodologies FEMA P-58, *Seismic performance Assessment of Buildings,* (FEMA, 2012b), FEMA E-74, *Reducing the Risks of Nonstructural Earthquake Damage*, (FEMA, 2012a), and FEMA P-154, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, (FEMA, 2015), or as a retrofitted unreinforced masonry building at the top of page 1 of the form.

- Irregularities were described in terms consistent with FEMA P-154 on page 2 of the form.
- Opportunities for sketching building plans and elevations were expanded on page 2 of the form.
- Parapets and major appendages were added to the scope of nonstructural systems and components examined on page 2 of the form.

The six-page form and the instructions are provided in Figures A-1 and A-2, respectively.

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Candidate for Further Study FEMA P-58 (PACT): ☐ FEMA E-74 (Nonstructural): ☐
FEMA P-154 (RVS): ☐ Retrofitted URM: ☐

ATC-38 POSTEARTHQUAKE BUILDING PERFORMANCE ASSESSMENT FORM

Note: DO NOT LEAVE ANY BLANK SPACES!
Indicate Unknown (UNK), Not Applicable (NA), or None if necessary.

Building Site Information [1]

Inspector(s):	Date:	Bldg. ID#:	Page _1_ of _6_
Address:		Building Name:	
Type of Survey: ☐ Exterior Only ☐ Exterior and Interior		Recording Station ID:	
Existing Posting Placard: ☐ Red ☐ Yellow ☐ Green ☐ None		Photo ID#s:	
Building Owner/Manager Contact – Name:		Phone:	
Civil/Structural Engineer for Repair – Name:		Phone:	
General Damage Classification (Structural): ☐ None (N) ☐ Insignificant (I) ☐ Minor (m) ☐ Moderate (M) ☐ Heavy (H) ☐ Collapse (C) General Damage Classification (Nonstructural): ☐ None (N) ☐ Insignificant (I) ☐ Minor (m) ☐ Moderate (M) ☐ Heavy (H) ☐ Collapse (C)			

[Note: For "M" or "H" classification, fill out Detailed Damage Description Section on page 5]

Building Construction Data [2]

Construction Date:	Design Date:	Sloped Site: ☐ Yes ☐ No
Number of Stories Above Ground:		Number of Basement Levels:
Number of Living Units:	Foundation Type:	Soil Type:
Plan Width (ft):	Plan Length (ft):	Approximate Building Area (sq.ft.):
Occupancy Type (see Glossary):	Occupied Prior to Earthquake: ☐ Yes ☐ No ☐ UNK	
Notes:		

Model Building Type [3]

Predominant Model Building Type (see Glossary):	Seismic Retrofit: ☐ Yes ☐ No ☐ UNK
Describe Building if More Than One Model Building Type Present:	
Describe Retrofit if Present:	
Additions? If yes, describe building type, date of construction:	

Figure A-1 Postearthquake Building Performance Assessment Form (page 1 of 6).

Performance Modifiers [4] (refer to FEMA P-154 for pounding and irregularity criteria)	Bldg. ID#:	Page _2_ of _6_

Discontinuous Columns: ☐ Yes ☐ No ☐ UNK ☐ NA	Facade Setbacks: ☐ Yes ☐ No ☐ UNK ☐ NA
Pounding Potential: ☐ Yes ☐ No ☐ UNK ☐ NA	Seismic Expansion Joints: ☐ Yes ☐ No ☐ UNK ☐ NA
Open Front Plan: ☐ Yes ☐ No ☐ UNK ☐ NA	Other Torsional Imbalance: ☐ Yes ☐ No ☐ UNK ☐ NA
Plan Irregularities: ☐ Yes ☐ No ☐ UNK ☐ NA	Deterioration of Structure: ☐ Yes ☐ No ☐ UNK ☐ NA
Previous Earthquake Damage: ☐ Yes ☐ No ☐ UNK ☐ NA	
Describe Other Vertical Irregularities:	
Describe Other Plan Irregularities:	
Describe Other Pre-Earthquake Building Conditions:	

Sketch of Building [5]

Plan sketch provided on separate paper: ☐ Yes ☐ No
Elevation sketch provided on separate paper: ☐ Yes ☐ No

Nonstructural Elements [6]

Exterior Cladding/Glazing Code (see Glossary):
Partitions Code (see Glossary):
Ceilings Code (see Glossary):
Fire Protection: ☐ Yes ☐ No ☐ UNK ☐ NA
Elevators: ☐ Yes ☐ No ☐ UNK ☐ NA
Chimneys: ☐ Yes ☐ No ☐ UNK ☐ NA
Parapets: ☐ Yes ☐ No ☐ UNK ☐ NA
Major Appendages: ☐ Yes ☐ No ☐ UNK ☐ NA
Standard Plumbing, Electrical, Lighting, HVAC: ☐ Yes ☐ No ☐ UNK ☐ NA
Describe Major Fixed Equipment:
Describe Unusual Contents:

Figure A-1 Postearthquake Building Performance Assessment Form (page 2 of 6).

General Damage [7]	Bldg. ID#:	Page _3_ of _6_

General Damage Classification (repeated from Section [1] on page 1): ☐ None (N) ☐ Insignificant (I) ☐ Minor (m) ☐ Moderate (M) ☐ Heavy (H) ☐ Collapse (C)	
[Note: See Glossary for ATC-13 Damage State Definitions]	
ATC-13 Damage State, Structural:	ATC-13 Damage State, Nonstructural:
ATC-13 Damage State, Equipment:	ATC-13 Damage State, Contents:
Percent of Floor Area Collapsed: _____% ☐UNK ☐NA	
Building off Foundation: ☐Yes ☐No ☐UNK ☐NA	Story out of Plumb: ☐Yes ☐No ☐UNK ☐NA
Damage to Structural Members: ☐Yes ☐No ☐UNK ☐NA	Hazmat: ☐Yes ☐No ☐UNK ☐NA
Parapet Damage: ☐Yes ☐No ☐UNK ☐NA	Chimney Damage: ☐Yes ☐No ☐UNK ☐NA
Exterior Non-building Damage: ☐Yes ☐No ☐UNK ☐NA	Pounding Damage: ☐Yes ☐No ☐UNK ☐NA
Comments about General Damage:	

Nonstructural Damage [8]

Cladding Separation or Damage: _____% of wall area ☐ UNK ☐ NA
Partitions Damage: ☐ None (N) ☐ Insignificant (I) ☐ Minor (m) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Windows Damage: _____% of windows ☐UNK ☐NA
Lights and Ceilings Damage: ☐None (N) ☐Insignificant (I) ☐Minor (m) ☐Moderate (M) ☐Heavy (H) ☐UNK ☐NA
Buildings Contents Damage: ☐None (N) ☐Insignificant (I) ☐Minor (m) ☐Moderate (M) ☐Heavy (H) ☐UNK ☐NA
Comments about Nonstructural Damage:

Injuries or Fatalities [9]

No. of Minor Injuries: _____ ☐UNK	No. of Major Injuries: _____ ☐UNK	No. of Fatalities: _____ ☐UNK
Comments about Injuries or Fatalities:		

Figure A-1 Postearthquake Building Performance Assessment Form (page 3 of 6).

Functionality [10]	Bldg. ID#:	Page _4_ of _6_
Percent Usable Space Immediately: _____% ☐ UNK	Percent Usable Space in 1-3 Days: _____% ☐ UNK	
Percent Usable Space within 1 Week: _____% ☐ UNK	Percent Usable Space within 1 Mo.: _____% ☐ UNK	
Percent Usable Space in 1-6 Months: _____% ☐ UNK	Time Until Full Occupancy: _____ ☐ UNK ☐ NA	
Comments about Functionality:		

Geotechnical Failures [11]

Lateral Ground Movement: ☐ Yes ☐ No ☐ UNK ☐ NA	Buckled Sidewalks: ☐ Yes ☐ No ☐ UNK ☐ NA
Ground Settlement: ☐ Yes ☐ No ☐ UNK ☐ NA	Liquefaction Indicators: ☐ Yes ☐ No ☐ UNK ☐ NA
Separation Between Building and Ground: ☐ Yes ☐ No ☐ UNK ☐ NA	
Landslide: ☐ Yes ☐ No ☐ UNK ☐ NA	
Comments about Geotechnical Features:	

Additional Comments

Additional Comments Pertaining to Any Section of Survey Form (use additional pages if necessary):

Figure A-1 Postearthquake Building Performance Assessment Form (page 4 of 6).

DETAILED DAMAGE DESCRIPTION

Bldg. ID#:	Page _5_ of _6_

Vertical Elements

Racking of Main Walls: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Racking of Cripple Walls: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Buckling, Crippling, Tearing of Steel Beams, Columns, or Braces: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Spalling or Cracking of Concrete Columns or Beams: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Column Crushing Due to Overturning or Discontinuous Lateral Resisting Elements: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Shear Cracking in Columns: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Cracked Shear Walls: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Percentage of Shear Walls with Cracks: _____% ☐ UNK ☐ NA
Rocking of Shear Walls: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Damage to Shear Wall Boundary Elements: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Damage to Shear Wall Coupling Beams: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
/ % of Tiltup Wall Panels Leaning or Fallen Out: _____ / _____% ☐ UNK ☐ NA
Infill Walls Damaged or Fallen Out: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA

Horizontal Elements

Roof Collapse: _____ % of Diaphragm ☐ UNK ☐ NA	Floor Collapse: _____ % of Diaphragm ☐ UNK ☐ NA
Loss of Vertical Roof Support: _____ % of Roof Area Affected ☐ UNK ☐ NA	
Tearing of Diaphragms at Other Points of High Stress: _____ % of Diaphragm ☐ UNK ☐ NA	
Damage at Re-entrant Corners: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA	
Damage to Collectors at Walls: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA	
Cross Grain Bending Damage at Roof-to-Wall Connections: _____ % of Connection Length ☐ UNK ☐ NA	

Figure A-1 Postearthquake Building Performance Assessment Form (page 5 of 6).

DETAILED DAMAGE DESCRIPTION (Continued)

Bldg. ID#:	Page _6_ of _6_

Connections

Girder-Column Connection Damage Including Panel Zones: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Column Splice Damage: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Damage to Brace Connections: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Damage to Column-to-Foundation Connections: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Damage to Connections of Precast Elements that are Part of the Lateral Force Resisting System: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA

Foundations

Foundations Cracked or Otherwise Damaged: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Slabs-on-Grade Cracked or Otherwise Damaged: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA

Equipment and Systems

Electrical Equipment Damage Including Backup Generators: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Damage to Boilers, Chillers, Tanks, etc.: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
HVAC Damage (Fans, Ducts) : ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Damage to Water and Sprinkler Lines and Fire Pumps: ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA
Elevator Equipment Damage (Car and Counterweight Rails, Cars, Penthouse Equipment): ☐ None (N) ☐ Insignificant (I) ☐ Moderate (M) ☐ Heavy (H) ☐ UNK ☐ NA

Additional Comments (use additional pages if necessary:

Figure A-1 Postearthquake Building Performance Assessment Form (page 6 of 6).

ATC-38 POSTEARTHQUAKE BUILDING PERFORMANCE ASSESSMENT FORM

SURVEYOR INSTRUCTIONS

This form should be filled out as completely as possible by the surveyor(s). Do not leave blank spaces; use "UNK" for "Unknown", "NA" for "Not Applicable", or "None" when appropriate. Talk with the owner to obtain as much information as possible. Assure him/her that detailed name and address information will not be released to the public. Photos should be taken of each exterior building elevation, and of any locations where significant damage is visible. For each strong motion site, obtain or sketch a map of the block or blocks surveyed to identify the locations of each building relative to the strong motion instrument. Distances from the buildings to the instrument should be determined wherever possible.

The ATC-38 Postearthquake Building Assessment Form includes 11 sections as listed below. Refer to the *Glossary of Terms and Codes* for classifications and codes that should be used on the form. The form is intended to be self-explanatory; however, some clarifying comments are included here for each of the 11 sections. In all cases, write down as much information as possible, and state any assumptions you need to make about the building and/or its performance. Too much or repeated information is always better than incomplete information.

1. **Building Site Information**. For Building ID#, use the following notation: station owner, last 3 digits of station number, initials of surveyor, and sequential number. (For example: CDMG386-ER-01.) Be sure to include the Building ID number on each page and indicate the number of pages. For Photo ID#s, make sure to note the number(s) on the film roll that were taken of the given building. When the film is developed, write the same numbers on the back of each photo so they will be matched to the proper building.

2. **Building Construction Data**. If possible, indicate design date and construction date by year, not decade.

3. **Model Building Type**. If the building has different model building types in different directions or on different floors, describe in the space provided.

4. **Performance Modifiers**. In this section, describe any other vertical or plan irregularities that are not listed on the form, including unusual pre-earthquake building conditions.

5. **Plan Sketch of Building**. Provide a sketch of the building footprint. Annotate the sketch as appropriate. Note on the sketch the assumed east-west and north-south directions if they are used in other sections of the form, and include a north arrow. Surveyors should carry a compass.

6. **Nonstructural Elements**. Refer to the *Glossary* for codes to be used for cladding and partition types.

7. **General Damage**. This section should be descriptive as well as quantitative. Indicate the General Damage Classification that corresponds to the worst damage to any specific element. (This should be the same General Damage Classification as that checked in Section 1.) Estimate the ATC-13 damage state as defined in the *Glossary* for each building area as shown (for residences, consider chimneys and veneer to be nonstructural and water heaters to be equipment). In the space provided for comments, include possible reasons for damage if appropriate. For buildings with General Damage Classification of "M" or "H", fill out the 2-page *Detailed Damage Description* as described below.

Figure A-2 Surveyor instructions (page 1 of 4).

8. **Nonstructural Damage**. Indicate damage to partitions, lights, ceilings, and contents in terms of General Damage Classification as defined in the *Glossary*.

9. **Injuries or Fatalities**. Include comments where appropriate, such as unusual reasons for casualties.

10. **Functionality.** Indicate percentage of space that can be used for the building's original pre-earthquake function for the various time periods listed, as well as the amount of time needed to restore the building to its full pre-earthquake functionality. In the comments section, include any reasons for closure and note if the building can only be accessed for clean-up.

11. **Geotechnical Failures**. In this section, describe any other geotechnical failures or unusual features that are not listed on the form.

After the 11 main sections of the form, space is provided for additional comments pertaining to any section of the form. Attach additional sheets if necessary, making sure to label each sheet with the Building ID number. For buildings with General Damage Classification of "M" or "H", fill out the 2-page *Detailed Damage Description* as briefly described below.

Detailed Damage Description. This part of the form should be filled out as completely as possible for any buildings with General Damage Classification of "M" or "H". It includes sections for Vertical Elements, Horizontal Elements, Connections, Foundations, and Equipment and Systems. In each case the damage should be described in terms of the General Damage Classification defined in the *Glossary*. Make sure to use "NA" or "UNK" as appropriate. Use the notes section to include additional information about the building and the damage, such as differences by direction or floor level in damage or model building type. The notes section may also be used to indicate the location (i.e., ground floor or top story) of extensive damage to equipment and systems. Add extra pages if necessary, making sure to label each one with the Building ID number.

Figure A-2 Surveyor instructions (page 2 of 4).

ATC-38 GLOSSARY OF TERMS AND CODES

General Damage Classification:

Code	Description
N	No damage is visible, either structural or nonstructural.
I	Damage requires no more than cosmetic repair. No structural repairs are necessary. For nonstructural elements this would include spackling partition cracks, picking up spilled contents, putting back fallen ceiling tiles, and repositioning equipment and furnishings.
m	Minor repairable structural or nonstructural damage has occurred. Repairs can be made without significant disruption to occupants. This damage state includes cracked or dislodged masonry requiring repair.
M	Repairable structural damage has occurred. The existing elements can be repaired in place, without substantial demolition or replacement or elements. For nonstructural elements this would include minor replacement of damaged partitions, ceilings, contents, or equipment.
H	Damage is so extensive that repair of elements is either not feasible or requires major demolition or replacement. Includes URM buildings that require partial or complete reconstruction of damaged masonry walls. For nonstructural elements this would include major or complete replacement of damaged partitions, ceilings, contents, or equipment.
C	Partial or complete loss of gravity support with collapse.

Occupancy Type:

Occupancy Type	Code
Apartment	A
Auto Repair	AR
Church	C
Dwelling	D
Data Center	DC
Garage	G

Gas Station	GS
Government	GV
Hospital	H
Hotel	HL
Manufacturing	M
Office	O
Restaurant	R

Retail	RS
School	S
Theater	T
Utility	U
Warehouse	W
Other	OTH
Unknown	UNK

Model Building Type:

Framing System	Reference Codes and Diaphragm Types
Steel Moment Frame	S1 - Stiff Diaphragms; S1A - Flexible Diaphragms
Steel Braced Frame	S2 - Stiff Diaphragms ; S2A - Flexible Diaphragms
Steel Light Frame	S3
Steel Frame w/ Concrete Shear Walls	S4 - Stiff Diaphragms; S4A - Flexible Diaphragms
Steel Frame w/ Infill Masonry Shear Walls	S5 - Stiff Diaphragms; S5A - Flexible Diaphragms
Concrete Moment Frame	C1 - Stiff Diaphragms; C1A - Flexible Diaphragms
Concrete Shear Wall Building	C2 - Stiff Diaphragms; C2A - Flexible Diaphragms
Concrete Frame w/ Infill Masonry Shear Walls	C3 - Stiff Diaphragms ; C3A - Flexible Diaphragms
Reinforced Masonry Bearing Wall	RM1 - Flexible Diaphragms; RM2 - Stiff Diaphragms
Unreinforced Masonry Bearing Wall	URM - Flexible Diaphragm; URMA - Stiff Diaphragm
Precast/Tiltup Concrete Shear Walls	PC1 - Flexible Diaphragms; PC1A - Stiff Diaphragms
Precast Concrete Frame w/ Conc. Shear Walls	PC2
Wood Light Frame	W1
Commercial or Long-Span Wood Frame	W2
Mobile Home/School Portable	MH
Multi-unit, multi-story residential	W1A

Figure A-2 Surveyor instructions (page 3 of 4).

ATC-38 GLOSSARY OF TERMS AND CODES (continued)

Exterior Cladding/Glazing Codes:

Cladding/Glazing Type	Code
Stucco	S
Wood Product	W
Curtain Wall	C
Brick	B
Glass	G
Concrete	O
Metal	M
Exposed Structure	E
Window Wall	I
Pre-cast Panels	P
PC Fascia	F
Stone	N
Marble	R
URM	U
Masonry	Y
Ceramic Tiles	T

Partitions Codes:

Partition Type	Code
Gypsum Board	G
Plaster	P
Wood Lath	W
URM	U
Metal	M
Concrete	C
Brick	B
Marble	R
Masonry	Y

Ceilings Codes:

Ceiling Type	Code
Gypsum Board – nailed directly to framing	G
Gypsum Board - suspended	H
Suspended Tile	S
Lath and plaster – attached directly to framing	L
Lath and plaster - suspended	P
Exposed Slab	E
Metal	M
Wood	W
Glued Tiles	T
Suspended acoustic T-Bar	A

ATC-13 Damage State Definitions:

Damage State	Percent Damage (damaged value ÷ replacement value)
0 Unknown	Unknown
1 None	0%
2 Slight	0% - 1%
3 Light	1% - 10%
4 Moderate	10% - 30%
5 Heavy	30% - 60%
6 Major	60% - 100%
7 Destroyed	100%

Figure A-2 Surveyor instructions (page 4 of 4).

Appendix B
Recovery Advisories

This appendix presents two recovery advisories being prepared by the Applied Technology Council for the Federal Emergency Management Agency in response to the 2014 South Napa earthquake.

B.1 Earthquake Strengthening of Cripple Walls in Wood-Frame Dwellings

South Napa Earthquake Recovery Advisory FEMA DR-4193-RA2, *Earthquake Strengthening of Cripple Walls in Wood-Frame Dwellings*, was under development by the Applied Technology Council at the time this report was being finalized, thus a brief summary is provided here. The document will be available on the FEMA website (http://www.fema.gov/earthquake-publications) when completed.

B.1.1 Purpose and Background

The South Napa earthquake and past earthquakes have damaged many cripple walls in residential structures, causing significant repair costs to the homeowners. The repair cost for failed cripple walls is significantly higher than the cost of strengthening cripple walls before an earthquake.

Several existing documents provide prescriptive approaches to the strengthening of cripple walls up to four feet tall:

- Chapter A3 of the *International Existing Building Code* (ICC, 2012b) provides provisions and details for engineers.
- *Standard Plan A: Residential Seismic Strengthening Plan* (developed in 2008, available at http://seaonc.org/free-publications, last accessed March 12, 2015) provides a prescriptive seismic strengthening plan for cripple wall bracing and foundation sill plate anchorage of light wood-framed residential structures.
- *Seismic Retrofit for Residential Wood Frame Cripple Walls and Sill Plate Anchorage* (developed by Simson Strong-Tie in 2012, available at http://www.strongtie.com/literature/f-plans.html, last accessed March 12, 2015) is based on the 2012 *International Existing Building Code* and calls out the specific retrofit solutions for reinforcing a home's cripple wall and foundation connection.

- *Standard Earthquake Home Retrofit Plan Set* (prepared by the City of Seattle under Project Impact in 2012, available at http://www.seattle.gov/dpd/static/get_file/Earthquake%20Home%20Retrofit%20Planset_DPDD017407_LatestReleased.pdf, last accessed March 12, 2015) provides plan details and reference sheets.

Where a dwelling meets the limitations set for applicability and where approved by the local building official, a homeowner or contractor may use one of the prescriptive plan sets to strengthen cripple walls up to four feet tall without a detailed design by an engineer.

B.1.2 Scope of Document

This Recovery Advisory currently under development briefly describes the issues involved in the retrofit of cripple walls in wood residential structures and will include a prescriptive plan set for the earthquake strengthening of cripple walls and the anchorage of sill plates in one- or two-story tall, one- or two-family residential structures with cripple walls up to seven feet in height.

The plan set is generally consistent with the seismic strengthening provisions of the 2012 *International Existing Building Code* and the 2013 *California Existing Building Code* (California Building Standards Commission, 2013b). The design in the plan set is based on the following assumed seismic design parameters: S_{DS} equal to 1.56g and S_{D1} equal to 0.97 g; Seismic Design Category D; Site Class D; Seismic Importance Factor, I, equal to 1.0; Response Modification Factor, R, equal to 6.5; and Design Base Shear, V, equal to 0.13W, where W is the seismic weight of the structure (using allowable stress design). The value of the Design Base Shear will include a 0.75 reduction factor commonly applied in design for existing structures, as specified in Section A301.3 of the 2012 *International Existing Building Code*. Given this design basis, the plan set is expected to be applicable in many areas of high seismicity in the United States, not just in the vicinity of Napa, California.

The plan set includes the following sheets: Technical notes; construction data and earthquake strengthening schedule; layout plan; and details of sill plate to concrete foundation connections, floor framing to sill plate connections, cripple wall to floor framing connections, tie-down installation, plywood braced-panel installation, foundation replacement, and panel notching and top-plate splices. The plan set will also provide example foundation plans with example strengthening of cripple walls.

B.2 Repair of Earthquake-Damaged Masonry Fireplace Chimneys

South Napa Earthquake Recovery Advisory FEMA DR-4193-RA1, *Repair of Earthquake-Damaged Masonry Fireplace Chimneys*, recommends best practices for reconstruction of earthquake-damaged masonry chimneys in one- and two-family dwellings to minimize risk of damage in a future earthquake. The information provided is advisory and building permits are required when conducting the work described.

FEMA DR-4193-RA1 is provided in its entirety in the following 11 pages.

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Repair of Earthquake-Damaged Masonry Fireplace Chimneys

SOUTH NAPA EARTHQUAKE RECOVERY ADVISORY *FEMA DR-4193-RA1*

Purpose and Intended Audience

The August 24, 2014 South Napa Earthquake has again served as a reminder that masonry chimneys in wood-frame dwellings are extremely vulnerable to earthquake shaking. FEMA assessment teams observed over 100 brick masonry chimneys damaged in the South Napa Earthquake (Figures 1 and 2). Older, unreinforced masonry chimneys with degraded mortar are most vulnerable, but even masonry chimneys constructed according to modern standards are susceptible to significant damage. Collapses of previously damaged and reconstructed chimneys were also observed.

Figure 1: Chimney damaged in the South Napa Earthquake. Photo Credit: Exponent.

This Recovery Advisory recommends best practices for reconstruction of earthquake-damaged masonry chimneys in one- and two-family dwellings to minimize risk of damage in future earthquakes. Owners of dwellings that might have a historic designation, and owners of multi-family dwellings, should consult with the building department regarding applicable requirements.

Information in this advisory is intended to be used by homeowners to compare and contrast options for reconstruction, and by contractors to understand details and applicable building code requirements associated with implementation of these options. ***Note that building permits are always required when performing the work described in this advisory.***

Key Issues:

1. Damaged chimneys reconstructed to match pre-earthquake conditions will remain vulnerable to damage in future earthquakes.
2. Repair of older, unreinforced masonry chimneys to meet modern seismic performance standards is generally considered to be infeasible, and such chimneys may remain vulnerable to collapse in future earthquakes.
3. Best practices for reducing future potential for damage involve partial or complete removal of the masonry and reconstruction with metal flues or fireplace inserts and light-frame construction.

This Recovery Advisory Addresses:

- Capping of the chimney at the roof level (Alternative A)
- Reconstruction of the chimney from the top of the firebox up, either maintaining the use of the existing masonry fireplace (Alternative B), or installing a fireplace insert (Alternative C), or
- Full reconstruction of the firebox and chimney (Alternative D).

Figure 2: Chimney damaged in the South Napa Earthquake. Photo Credit: Janiele Maffei, CEA

Best Practices for Minimizing Future Risk

Good – Capping of Chimney at Roof Level (Alternative A)

For a single story dwelling, if all damage occurred at or above the roof level, the chimney can be permanently removed down to the roof level. This is only possible when use of the fireplace will be discontinued. This is the least costly of the alternatives, but also provides a lesser level of hazard mitigation.

Exterior appearance: The firebox and chimney will remain unchanged up to the roof level. The upper portion of the chimney will be removed.

Interior appearance: Fireplace and mantel will remain, but the fireplace can no longer be used and will need to be closed off.

Environmental: Fireplace will no longer burn combustible materials.

Better – Reconstruction from Top of Firebox, Maintaining Existing Fireplace (Alternative B)

Allows continued use of an undamaged masonry firebox in combination with a new metal flue and light-weight chimney enclosure.

Exterior appearance: The firebox at the bottom of the chimney will remain exposed brick. The reconstructed portion of the chimney is often finished with siding or stucco, but can also be finished with adhered brick veneer to preserve the original appearance.

Interior appearance: Remains unchanged.

Environmental: Fireplace can remain fuel-burning (note that the owner could also choose to convert to a more environmentally friendly gas-burning fireplace).

Better – Reconstruction from Top of Firebox, Using Fireplace Insert (Alternative C)

Allows continued use of an undamaged masonry firebox in combination with a new chimney. In addition, a factory-built fireplace insert is installed inside of the fireplace.

Exterior appearance: The firebox at the bottom of the chimney will remain exposed brick. The reconstructed portion of the chimney is often finished with siding or stucco, but can also be finished with adhered brick veneer to preserve the original appearance.

Interior appearance: Fireplace and mantel will remain, but a fireplace insert will be visible inside of the original masonry fireplace.

Environmental: Fireplace can remain fuel-burning. Fireplace inserts can be more energy efficient at producing heat in the home, can reduce emissions through more complete combustion of solid fuels, or can be converted to more environmentally friendly gas-burning.

Best – Full Reconstruction of Firebox and Chimney (Alternative D)

Involves replacement of the entire firebox and chimney with light-frame construction above the top of the foundation. This is necessary if earthquake damage extends below the shoulder of the firebox. It is also appropriate where complete removal of fireplace masonry is preferred.

Exterior appearance: The entire height of the firebox and chimney is reconstructed and is often finished with siding or stucco, but can also be finished with adhered brick veneer to preserve the original appearance. None of the original brick masonry construction remains.

Interior appearance: Fireplace and mantel will be removed and replaced with factory-built fireplace unit. This can provide an opportunity to change or modernize the interior appearance or enhance the use of the fireplace.

Environmental: Fireplace can remain fuel-burning. Factory-built fireplace units can be more energy efficient at producing heat in the home, can reduce emissions through more complete combustion of solid fuels, or can be more environmentally friendly gas-burning.

Other Methods of Repair or Reconstruction

While the solutions detailed in this advisory are recommended as best practices, other code-conforming approaches for repair or reconstruction are possible:

- **Installation of concrete inside the flue.** In cases where the fireplace will not be used in the future, lightly damaged chimneys can sometimes be repaired and strengthened through the installation of rebar into the flue (over the full height) and by filling the flue with concrete. This approach reduces the potential for some types of damage, but the increased weight and difficulty in adequately anchoring the heavy chimney to the dwelling raises questions about the validity of this approach and the overall benefit for reducing future risk.
- **Engineered solutions.** Engineered solutions for reconstruction of damaged chimneys are possible. An engineered approach has the advantage that the solution can be customized for the specific situation, and can include a number of wide-ranging options for reinforcing or reconstructing the chimney and firebox. An engineered approach may be appropriate or necessary in some dwellings.
- **Reconstruction in-kind.** Although reconstruction of chimneys to their pre-earthquake configuration may be permitted by building codes in some jurisdictions, this approach does little to reduce the potential for future damage or risk to life-safety, and is not recommended.

Recommended Methods of Hazard Mitigation

Whether or not your chimney has been damaged, now is a good time to be thinking about mitigating the hazard associated with masonry chimneys before the next earthquake. Although they are presented as alternatives for repair and reconstruction after damage in an earthquake, the solutions detailed in this advisory are also recommended as best practices for pre-earthquake hazard mitigation (retrofit) of chimneys. As such they can be applied equally to undamaged chimneys.

Where it is not possible to implement these best practices outlined in this advisory, other steps can be taken before the next earthquake to partially reduce the risk posed by masonry chimneys, such as:

- Minimizing time spent next to the chimney, both inside and outside the dwelling. This will minimize the risk of serious injury should the chimney collapse. In particular, sleeping adjacent to a masonry chimney should be avoided.
- Installing plywood in the attic space, either attached to the underside of the roof rafters or the top of the ceiling joists. Plywood in the attic space is intended to slow or even prevent brick debris from coming through the ceiling into the living space in the event of chimney failure. This approach is easy and relatively inexpensive; however, the effectiveness is likely to vary widely from residence to residence. It is important that plywood installed for this purpose does not become used as an attic storage area, as this can overload the ceiling and roof systems and result in failure.

Several methods are NOT recommended for pre-earthquake mitigation. These include the following:

- Masonry chimneys should not be retrofitted using steel braces that extend down to the roof surface. Such bracing is thought to have promoted chimney failure in past earthquakes, and must be carefully engineered.
- Simply restoring the outside surface of deteriorated mortar joints (repointing) or patching of isolated masonry cracks is not sufficient. While such actions are encouraged as part of regular chimney maintenance, they will do little to reduce the potential for damage in future earthquakes.

General Information for Construction

Building Codes

Reconstruction of damaged chimneys must be in conformance with adopted building code provisions and local ordinances. Reconstruction must use manufactured parts (metal flues, anchor plates, flue caps, fireplace inserts, and factory-built fireplaces) meeting applicable Underwriters Laboratory (UL) Standards (commonly referred to as UL listed). For manufactured parts, installation in accordance with the manufacturer's installation instructions is mandatory. In addition, installation instructions impose some requirements on the surrounding construction, such as required dimensions and clearances.

Other parts of the reconstruction are required to be in accordance with applicable provisions of the adopted building or residential code. Most states and local jurisdictions base their residential code on a recent edition of the International Residential Code (IRC) (ICC, 2015). In California, this results in the California Residential Code (CRC) (CBSC, 2013), which can be further amended by the local building official.

This recovery advisory does not attempt to provide an exhaustive list of applicable building codes. Some limited sections of interest are summarized below. Persons designing or performing repair work are advised to become familiar with applicable requirements.

Scope of Work

For dwellings in California, repair of damage does not trigger requirements for seismic strengthening beyond the scope of repair (California Building Code Section 3405.1.2 (HCD 1)) (CBSC, 2013). The only exception is where conditions exist that are judged by the building official to result in substandard building conditions, in which case additional work to correct the substandard conditions would be required.

Building Permit

A building permit is required for work described in this advisory and must be obtained prior to the start of construction work.

California Residential Code Sections of Interest

For foundation extension:	Minimum concrete strength of 2500psi per Section R402.2
	Minimum depth of foundation, 12 inches below grade Section R403.1.4
For bolting to concrete beam:	Sections R403.1.6 and R603.3.1
For cold-formed steel stud walls:	Section R603
For fireplace and chimney construction:	Chapter 10
For roof flashing and crickets:	Section R905
For chimney enclosure wall covering:	Chapter 7

Dwellings with a Historic Designation

Older dwellings may have a historic designation, indicating that they are deemed to be historically significant. This designation can be given at the Federal (National Register of Historic Places), State (State Register of Historic Places) or local level, and local building departments generally have a list of buildings with such designation. For historic dwellings, further consideration of repair methods that preserve the existing historic construction and appearance are appropriate. Historic designation may come with additional requirements such as historic preservation guidelines, and review of planned repairs by the local planning department and historic preservation board. Historic designation may also permit flexibility regarding reconstruction methods and materials, including reconstruction of damaged masonry chimneys using matching masonry. Where such reconstruction is used, an engineered solution is recommended and a structural engineer should develop the best approach for mitigating future earthquake hazard. Owners of dwellings that may have a historic designation should consult with the building department regarding applicable requirements.

Alternative A: Capping of Chimney at Roof Level

Overview

For single story dwellings, where the chimney is damaged at or above the roof level and it is not intended that the fireplace be used in the future, it is possible to remove the chimney down to the roof line and install a sheet metal cap over the chimney for weather protection.

This alternative provides the greatest hazard reduction for chimneys that extend a considerable distance above the roof line, and less benefit for short chimneys.

It is recommended that the chimney be removed to a distance of three to six inches above the adjacent roof. This will permit a sheet metal cap that extends several inches down each side of the chimney to be installed, providing a flashing. The cap should be secured to the masonry.

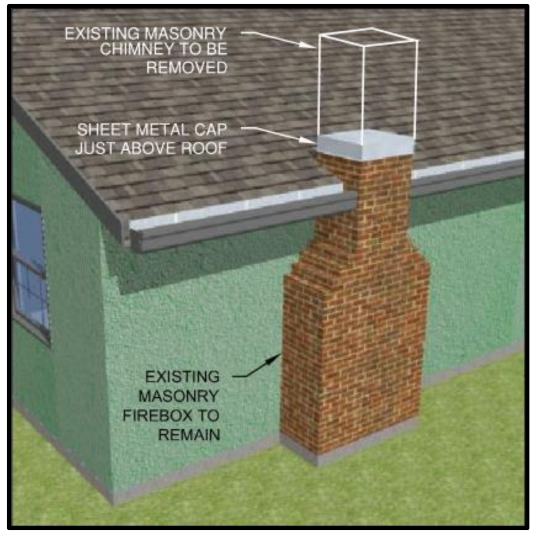

Figure 3: Dwelling with a chimney capped at the roof line.

The interior of the firebox will need to be closed off to prevent any possible use of the fireplace. For least impact on interior appearance, plywood can be installed across the fireplace opening, set back into the fireplace, and painted to match interior finishes.

Although capping eliminates the hazard associated with the most vulnerable portion of the chimney (above the roof), there is still risk of damage or collapse of the remaining masonry in future earthquakes.

Alternative B: Reconstruction from Top of Firebox Up, Maintaining Existing Fireplace

Overview

Alternative B permits continued use of an undamaged firebox in combination with a new chimney. Instead of using a conventional masonry chimney with a clay flue liner, the new chimney is constructed using a lightweight metal flue contained in a cold-formed steel stud chimney enclosure, as shown in Figure 4. This lightweight, flexible construction is much more resistant to damage in future earthquakes. Although the remaining masonry firebox could be damaged in a future earthquake, the firebox is much less vulnerable to damage than the original masonry chimney. As a result, the risk of collapse and the associated risk to life safety are greatly reduced when this reconstruction alternative is implemented.

Figure 4: Dwelling with chimney reconstructed from the firebox up.

Primary Components

The primary components of Alternative B construction are listed below, illustrated in Figure 5. Additional details of construction are provided in Figures 6 and 7.

1. **Masonry firebox.** *Inspect the firebox to verify that it is in good condition prior to start of repair work.*
2. **Existing framing.** *To remain as is except for roof blocking as detailed in Figure 7 and Item 7 below.*
3. **Masonry veneer.** *Verify support and anchorage of existing veneer where it occurs above and surrounding the fireplace.*

4. **Firebox to flue transition.** *The transition from the masonry firebox to the metal flue includes: anchorage to masonry, concrete bond beam, steel adapter cone, and anchor plate. This detail is critical to the safe performance of this reconstruction alternative. See Figure 6 for more information.*
5. **Cold-formed steel track.** *Anchor track to concrete beam per Figure 6.*
6. **Cold-formed steel stud wall.** *Provide full height studs.*
7. **Chimney connection to dwelling.** *Provide stud blocking and steel strap connection to existing dwelling framing at upper floor, ceiling, and roof framing.*
8. **Insulation.** *Provide insulation between studs at exterior walls of the chase, allowing for the proper clearances in accordance with the manufacturer's installation instructions.*
9. **Metal flue.** *Provide UL Standard 103 listed metal flue, installed in accordance with manufacturer's instructions. Provide as large a flue as can be installed in available space while still meeting minimum clear distances.*
10. **Flue cap.** *Install flue cap supplied by flue manufacturer as part of the flue assembly.*
11. **Fire blocking.** *Provide fire blocking between chimney chase and attic as required by the building code.*

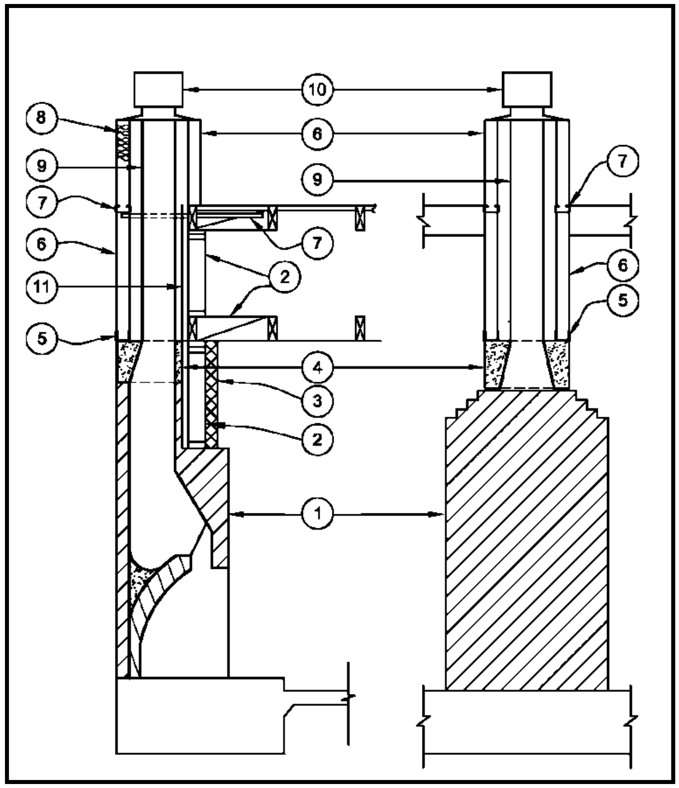

Figure 5: Components of a masonry firebox in combination with lightweight metal flue and chimney (Alternative B).

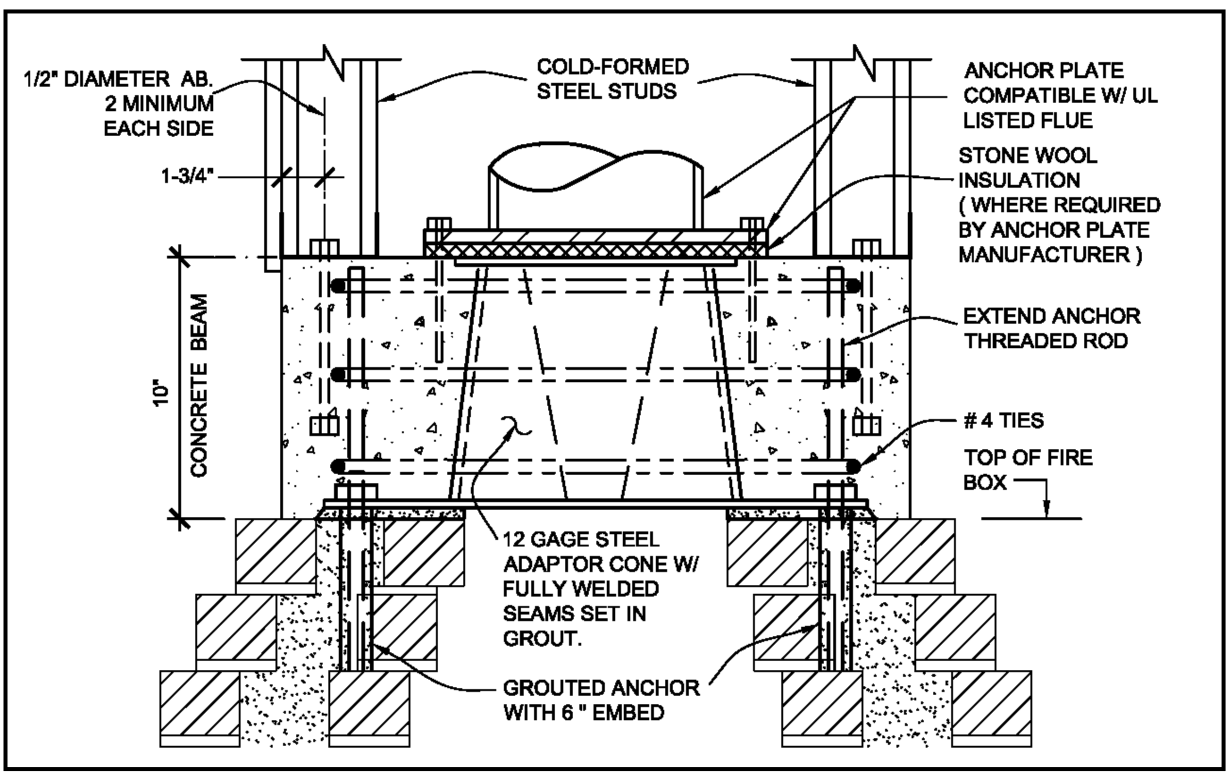

Figure 6: Example of detail at masonry firebox transition to metal flue and light-frame chimney (Alternative B, similar detail for Alternative C).

Requirements for Flue Transitions and Components

Figure 6 provides details for the transition from the masonry firebox to the metal flue and stud wall chimney chase. The components of this transition include the following:

- Fabricate a 12 gage (97 mil) minimum sheet steel adaptor cone as shown in Figure 6. The adaptor cone is to have minimum 12 gage (97 mil) sheet steel top and bottom plates as shown. This will likely need to be custom fabricated by a sheet metal shop for each chimney. All seams are to be fully welded. The adaptor cone is intended to provide a smooth-surfaced transition between the flue opening at the top of the firebox and the anchor plate and metal flue. The base plate geometry is to match the opening geometry at the top of the firebox, and the top plate geometry is to be coordinated with the anchor plate. The adaptor cone will also serve to minimize movement of heated gas through cracks that might form in the surrounding concrete beam. The adaptor cone is to be set in cementitious grout.
- Provide not less than four 1/2-inch diameter threaded rod anchors anchoring the adaptor cone base plate to the firebox masonry, as shown in Figure 6. Extend the threaded rods to one inch below the top of the concrete beam. Where the existing masonry is fully grouted at anchor locations, drill one-inch diameter holes six inches deep and set in high-strength grout. Where the existing masonry is not fully grouted, place threaded rods in cavity and grout the entire cavity.
- Place reinforcing steel (rebar) and construct a concrete beam around the adaptor cone, using the cone as the inside form. Maintain a minimum 1-1/2 inch clear distance between rebar and outside face of concrete.
- When required by anchor plate manufacturer, install stone wool (basalt) insulation board on top of the transition cone top plate as shown in Figure 6.
- Install fireplace anchor plate in accordance with the manufacturer’s installation instructions. UL Standard 103a provides information on anchor plates; however, UL does not currently certify (list) these anchor plates. The provider of the metal chimney should provide an anchor plate that is intended for use with the chimney, and should verify that it has been tested per UL103a.
- Enclose the new flue in a light-frame chimney enclosure constructed of not less than 18 gage (43 mil) by 3-1/2 inch deep galvanized steel studs at not more than 12 inches on center. Install fire stops per code requirements. Fasten the steel studs to the existing residence exterior wall and tie the chimney framing into the existing roof framing with not less than 18 gage by 1-1/4 inch wide steel straps with not less than four #8 screws to the steel construction and four 8d common nails to existing wood construction.

Light-Frame Chimney Bracing to Roof

IRC and CRC requirements for the height of the chimney require that the top of the chimney extend three feet above the roof and not less than two feet above the elevation of the roof or other construction within a ten foot radius, as illustrated in Figure 7. This often requires that the chimney extend a significant distance above the roof line. Where this occurs, it is necessary to provide bracing of the chimney down to the roof. Such bracing should be provided in the upper third of the chimney clear height above the roof (H), as shown in Figure 7. Chimney bracing may also be required in Alternative C.

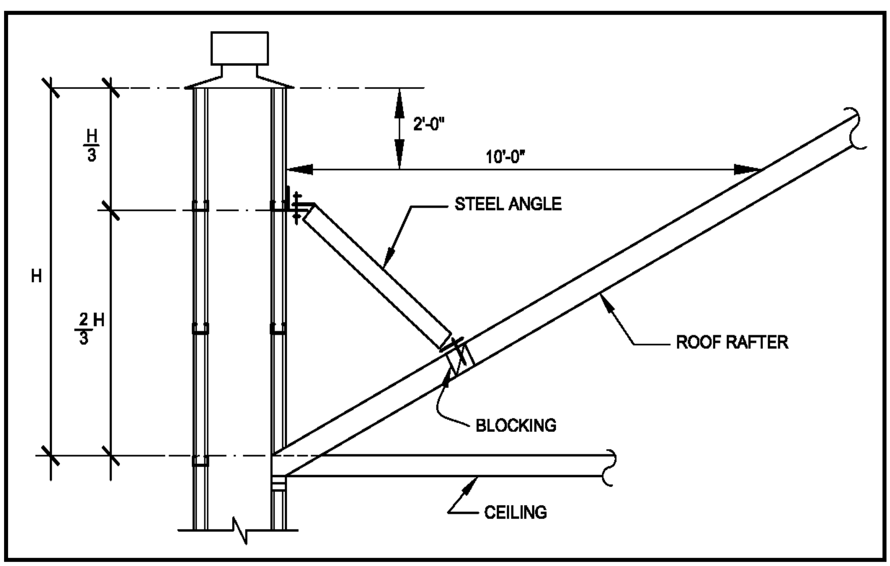

Figure 7: Bracing of light-frame chimney chase to roof (Alternatives B and C).

Alternative C: Reconstruction from the Top of Firebox, Using Fireplace Insert

Overview

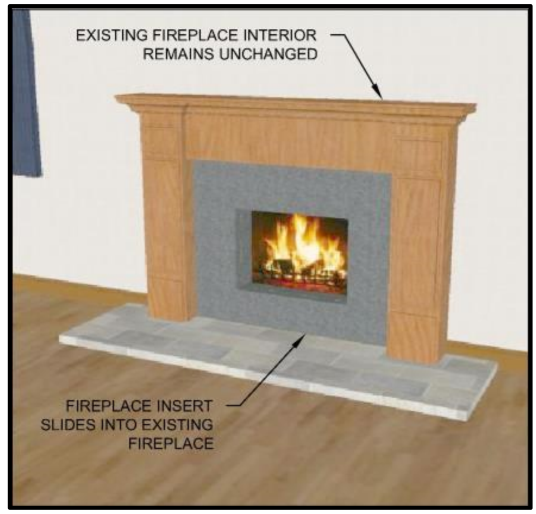

Figure 8: Fireplace insert

Alternative C permits continued use of an undamaged masonry firebox in combination with a new chimney and fireplace insert. For decades wood-frame dwellings included a conventional masonry chimney and fireplace. However, many homes are now constructed with prefabricated fireplaces, or have had factory units inserted into the original masonry firebox (Figure 8). While a factory insert will not make a fireplace more earthquake-resistant, it can be more energy efficient and more ecologically friendly. Thus a factory-built fireplace insert may be a viable option at the time of the repair.

A factory-built fireplace insert provides a firebox within a steel or cast iron shell. The insert occupies the space of the original masonry firebox and utilizes the existing chimney chase. Building codes require that these systems be UL listed, and fireplace inserts often come packaged with a metal chimney as a certified system. (Homeowners are cautioned against using a chimney or connections not approved by the fireplace manufacturer. If your home already has a fireplace insert that utilizes the original masonry chimney liner, you will need to find a metal flue that is UL listed for use with your fireplace insert.)

Primary Components

The primary components of the installation of a fireplace insert into an existing masonry fireplace are listed below and illustrated in Figure 9. Items not noted are similar to information shown in Figure 5.

1. **Bottom connection.** *Secure bottom of fireplace insert.*
2. **Fireplace insert.** *Provide UL Standard 127 listed fireplace insert.*
3. **Damper and draft stop**. *Remove as required to install flue.*
4. **Flexible flue transition from insert to chimney.** *Install transition supplied as part of UL listed flue.*
5. **Metal flue connections**. *Install flue connections supplied as part of UL listed assembly.*
6. **Firebox to flue transition.** *Provide transition similar to that shown in Figure 6.*
7. **Metal chimney**. *Provide UL listed metal chimney.*
8. **Cold-formed steel stud wall.** *Provide chimney enclosure as described for Alternative B.*

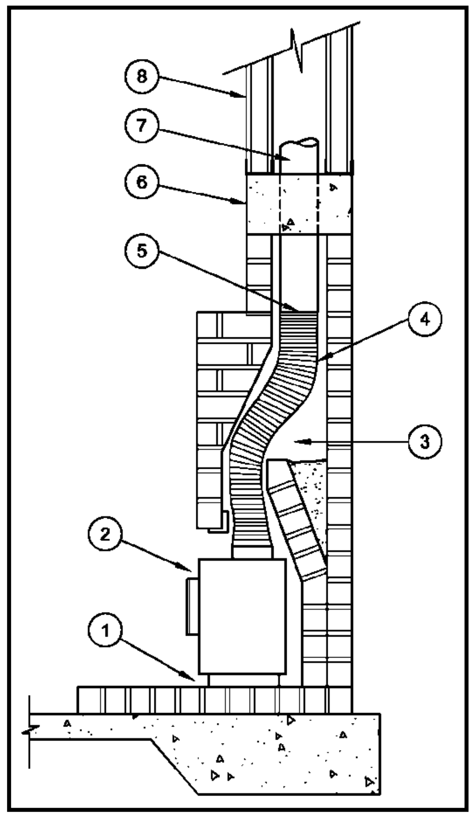

Figure 9: Components of a Factory-built Fireplace Insert (Alternative C).

Requirements for Fireplace Inserts

Installation of factory inserts must be done per the manufacturer's instructions and all applicable code requirements (consult your local building department). At a minimum, the following should be considered:

- Some homes have prefabricated chimneys that should not be equipped with an insert, or that greatly limit the inserts certified for such use. (It can be difficult to tell the difference between prefabricated and conventional masonry fireplaces without inspecting the inside of the firebox and flue connection.)

- Carefully check the dimensions to ensure the selected insert will fit properly. The size and location of the damper opening is particularly important. (If you are uncomfortable with this determination, enlist the help of a professional contractor.)
- The insert, chimney, and accessories must be certified per UL Standard 127 (listed and labeled as such). Your building official and contractor can assist with other local building code requirements.
- A hearth extension (UL 1618 listed) may be needed, particularly if the insert does not fit entirely within the firebox.
- The insert should be anchored to prevent shifting.
- Flue joints and connection to the fireplace must be per the manufacturer's instructions and UL standards.

Masonry Firebox Transition to Light-Frame Chimney

This method of reconstruction requires a transition from the firebox to the metal flue and stud wall chimney chase, similar to that shown in Figure 6 for Alternative B. Alternative C is different in that the metal flue runs continuous past the concrete bond beam. See installation instructions for required clear distance between bond beam and flue.

Light-Frame Chimney Bracing to Roof

This method of reconstruction is also subject to IRC and CRC chimney height requirements and chimney bracing may be required. See light-frame chimney bracing requirement in Figure 7 under Alternative B for more details.

Alternative D: Full Reconstruction of Firebox and Chimney

Overview

If earthquake damage extends below the shoulder of the firebox or if complete removal of all masonry is preferred, the entire firebox and chimney can be replaced from the top of foundation up. This involves installing a factory-built fireplace and metal flue inside a cold-formed steel stud chimney chase, as seen in Figure 10. This type of lighter and more flexible construction avoids many of the issues that have made masonry fireplaces and chimneys vulnerable to damage in past earthquakes. If preserving the architectural aesthetic of masonry is important, adhered masonry veneer can be used.

Figure 10: Replacement of entire firebox and chimney with light-frame construction.

Primary Components

The primary components of Alternative D construction are listed below and illustrated in Figure 11.

1. **Existing foundation.** *To remain.*
2. **Extension of existing foundation.** *Provide where required to meet dimensional requirements specified by the fireplace manufacturer. Where foundation extension is required, match the depth of the existing foundation, but not less than 12 inches below grade. Provide No. 4 rebar top and bottom of new concrete. Epoxy dowel to the existing footing at not more than 12 inches on center. See the applicable building code for additional requirements.*
3. **Non-combustible hearth extension.** *Provide hearth extension not less than 20 inches in depth.*
4. **Factory-built fireplace.** *Provide UL Standard 127 listed fireplace.*
5. **Cold-formed steel track.** *Anchor track to concrete foundation.*
6. **Cold-formed steel stud wall.** *Provide full height studs.*

7. **Existing framing.** *Framing may require modification to accommodate new fireplace opening. Use applicable building code provisions.*
8. **Metal flue.** *Provide UL listed metal flue supplied by fireplace manufacturer and to be installed in accordance with their installation instructions.*
9. **Stud blocking.** *Provide continuous blocking at 4'-0" maximum vertical spacing.*
10. **Insulation.** *Provide insulation between studs at exterior walls of the chase, allowing for the proper clearances in accordance with the manufacturer's installation instructions.*
11. **Chimney connection to dwelling.** *Provide stud blocking and steel strap connection to existing dwelling framing at upper floor, ceiling, and roof.*
12. **Existing framing.** *To remain.*
13. **Light-frame wall.** *From roof up.*
14. **Chimney cap.** *Provide framed chimney cap on chimney chase.*
15. **Flue cap.** *Install flue cap supplied by flue manufacturer.*
16. **Fire blocking.** *Provide fire blocking between chimney chase and attic.*

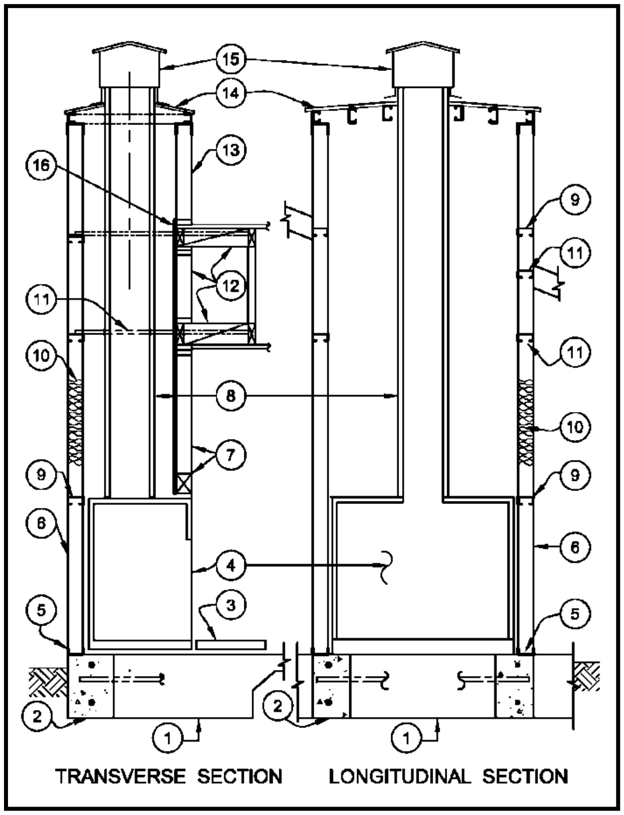

Figure 11: Components of a factory-built fireplace in light-frame chimney chase (Alternative D).

Requirements for Removal and Replacement of the Firebox

- Remove all existing masonry above the foundation.
- Fireboxes often extend a foot or more into the dwelling living space. Removal will need to include the interior portions of the masonry firebox. Mantels and shelves that are supported off of the firebox will likely need to be removed, but might be later reinstalled, supported off the dwelling wall.
- Masonry veneer is sometimes used as the interior finish for the wall around the fireplace. Where this is the case, this veneer will likely need to be removed to allow framing modifications and fireplace installation. If the veneer is to be reinstalled, details should conform to applicable provisions of IRC or CRC Chapter 7.
- Portions of the exterior wall, floor, ceiling, or roof of the dwelling could potentially be supported on the masonry firebox. Where this is the case, these elements of the dwelling will need to be re-supported. Modifications to dwelling framing should conform to applicable provisions of the IRC or CRC.
- Minimum dimensional requirements are provided in the manufacturer's installation instructions. These will determine the required depth and width of the chimney chase, as well as the size of the interior fireplace opening. It is possible that these dimensions will exceed current dimensions, requiring extension of the foundation and enlargement the framed wall opening. These modifications should be made in accordance with the IRC or CRC.
- Factory-built fireplace, flue and chimney cap should conform to UL Standard 127 and should be installed in accordance with the manufacturer's installation instructions.

Resources and other Useful Links

- Association of Bay Area Governments, **Training Materials for Seismic Retrofit of Wood-Frame Homes**. http://www.abag.ca.gov/bayarea/eqmaps/fixit/training.html

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- ICC, **2015 International Residential Code**, International Code Council, Country Club Hills, Illinois.

For more information, see the FEMA Building Science Frequently Asked Questions web site at http://www.fema.gov/frequently-asked-questions.

If you have any additional questions on FEMA Building Science Publications, contact the helpline at FEMA-Buildingsciencehelp@fema.dhs.gov or 1-866-927-2104.

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