*16th World Conference on Earthquake Engineering, 16WCEE 2017*

*Santiago Chile, January 9th to 13th 2017*

*3772 (Abstract ID)*

*Registration Code: S-O1461973736*

# ANALYTICAL STUDY OF RESIDENTIAL MASONRY CHIMNEYS DURING RECENT CALIFORNIA EARTHQUAKES

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## *Abstract*

The City of Napa in California had chimneys damaged from the 2000 Yountville (M5.0) and 2014 South Napa (M6.0) earthquakes. Presented is an analytical study of chimneys to assess key factors affecting behavior. Ground motions recorded at two locations in Napa were used as input to a simplified nonlinear house-chimney computer model, and the responses interpreted. It was found that the house had significant influence on its associated chimney behavior. For unreinforced chimneys, performance was influenced by the masonry flexural tensile strength, and this factor can be quite variable thus leading to erratic damage patterns. If chimneys were properly steel reinforced according to minimum building code requirements, they should have survived the subject earthquakes mostly unscathed suggesting that quality of construction (or lack thereof) was another key factor. Based on the results, suggestions for the formulation of improved chimney fragility functions are offered that are the subject of ongoing research.

*Keywords: masonry chimneys, earthquake damage, analytical study*

## 1. Introduction

Reconnaissance surveys for moderate or greater earthquakes (≥ 5.0M) affecting urban communities often report damage to masonry chimneys. Many brick masonry chimneys on residential homes in the City of Napa, California (population 80,000) were damaged during recent earthquakes. Utilizing recorded motions nearby chimneys that were damaged offered the opportunity to analytically investigate chimney response to gain insights about the factors at work governing their behavior. Earthquake ground motions recorded at two locations in Napa were used as input to a simplified nonlinear house-chimney computer model, and the responses interpreted. Recommendations for analytical-based fragility functions are offered.

## 2. Background

Steel reinforcement (rebar) and lateral anchorage to the house affect the seismic ruggedness of brick masonry chimneys. New chimneys are required by the building code to be reinforced and anchored to the house at the floor, ceiling and roof levels. Older chimneys might not be reinforced and/or anchored depending on code requirements and level of code enforcement at time of their construction. The Uniform Building Code (UBC) served for many years as the basis of building codes adopted by jurisdictions in the western United States. The UBC had requirements for reinforcement and anchorage starting in the mid-1940s [1]. However, more stringent prescriptive provisions are found in later editions perhaps pursuant to observed unsatisfactory construction practices (1967 and 1970 UBC).

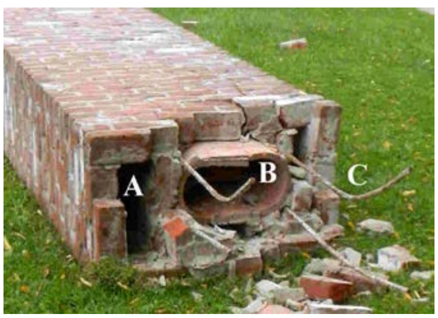
Masonry chimney damage has been reported after many earthquakes. The 1994 Northridge earthquake (M6.7) affecting the Los Angeles region is particularly noteworthy since it was centered in a large urban area. As many as 30,000 permits were issued by the City of Los Angeles for chimney work after the earthquake [2]. Due to the large number of damaged chimneys, the lessons learned from this event may apply in general to chimneys constructed in high seismic regions of the western U.S. including the City of Napa.

The Masonry Society surveyed masonry structures after the Northridge earthquake [3]. The survey found that poor construction practices and lack of adherence to building code provisions resulted in considerable damage to chimneys. These findings pointed to the need for more and better inspections, especially considering Los Angeles did not require chimney construction inspection prior to the Northridge earthquake. Key Northridge chimney damage observations follow.

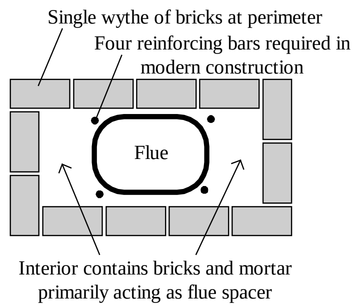
• Unreinforced chimneys performed poorly with damage being widespread. They appeared to fail primarily by collapse of sections (e.g., portion above roofline or above firebox), or detachment and collapse of the entire chimney.

• Reinforced chimney performance was variable. At locations close to the epicenter, some chimneys (presumably reinforced) appeared to have no damage, while other reinforced chimneys were damaged. Typical damage consisted of flexural failure of the cantilever portion above the roof, or overturning of the chimney as a rigid body.

• Construction in apparent violation of the building code seemed to be an important contributor to the failures of reinforced chimneys. Poor quality grout was common and many chimneys had construction debris at the bottom of grouted spaces. Many collapsed chimneys had vertical rebar, but lacked grout to bond the steel to the masonry. Other cases it appeared that vertical rebar had insufficient splice lengths so they pulled out. Still other cases appeared to lack lateral anchoring to the building, and/or anchorage to the foundation so that the chimney overturned. Figure 1 shows the internal features of a Napa chimney that failed during the 2014 South Napa earthquake.




(a)




(b)

Fig. 1 – Masonry chimney in Napa. (a) Tall reinforced chimney constructed in 1973 that failed at the roofline and collapsed during 2014 South Napa earthquake. Hollow cavity at A indicating chimney not grouted solid. Clay tile flue at B held in place by bricks and mortar not interconnected to perimeter wythe of bricks. Note how chimney failure plane went through the joint between two flue segments. Bent rebar at C that pulled out apparently due to insufficient bond strength (total four bars). (b) Section view depicting chimney internal features.

## 2. Napa Chimney Damage

The 2000 Yountville (M5.0) and 2014 South Napa (M6.0) earthquakes damaged chimneys in Napa [4, 5, 6, 7]. Chimneys in two areas in close proximity to earthquake recording stations in Napa were considered. As shown in Figure 2, these are identified as the Fire Station Area and Main Street Area. The City of Napa permit database was queried to find building permits issued for work on chimneys, and then site visits were conducted to assess particular chimneys. Figure 3 shows response spectra from the recorded motions.

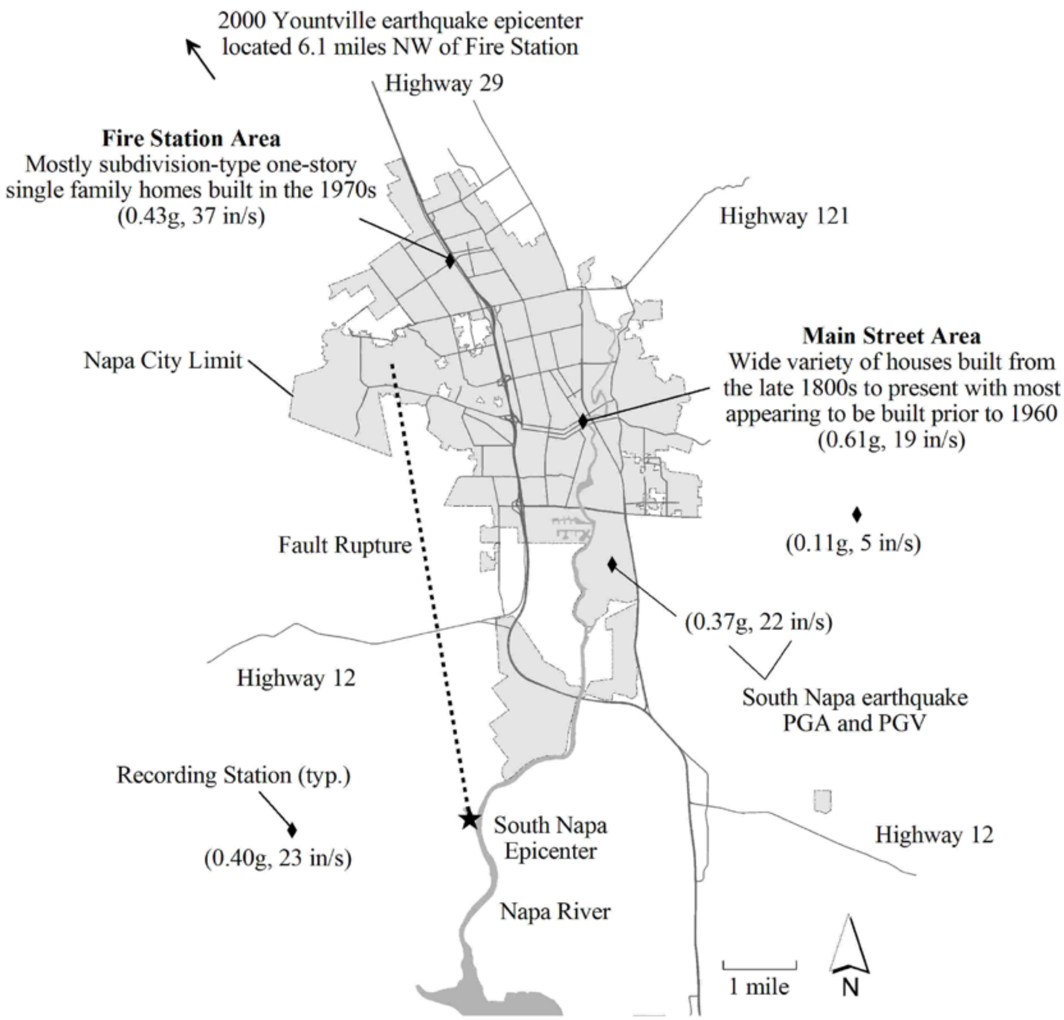



Fig. 2 – City of Napa map showing areas considered for chimney analyses. Also shown is the epicenter from 2014 South Napa earthquake, peak ground accelerations (PGAs) and peak ground velocities (PGVs).

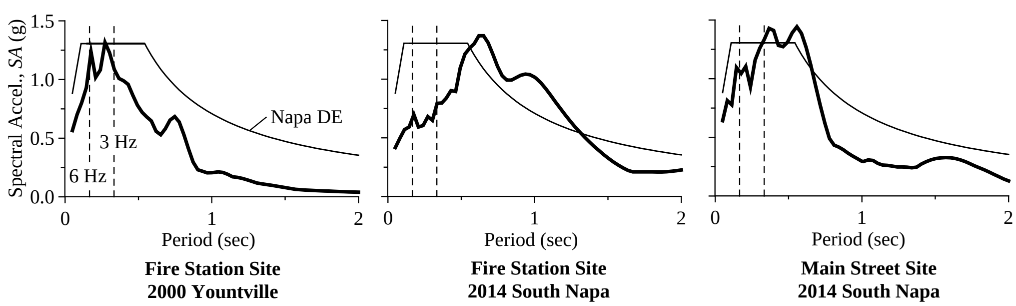
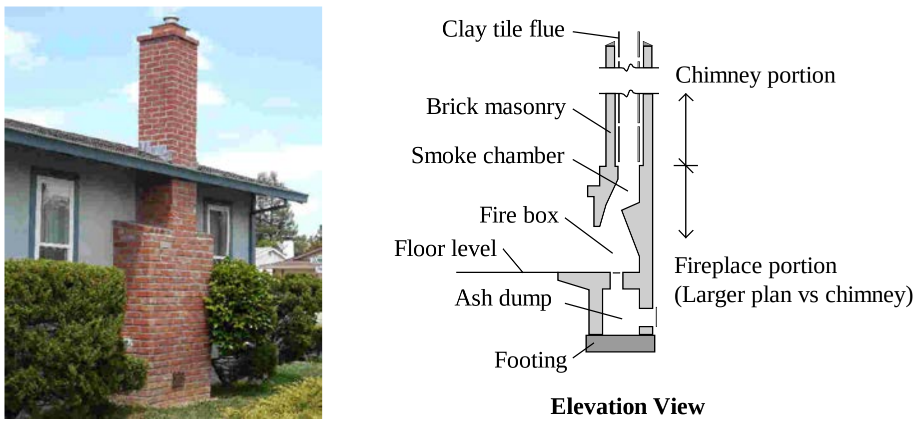



Fig. 3 – Response spectra of NS components from 2000 Yountville and 2014 South Napa earthquakes (5% damping). Also shown is the ASCE 7-10 design earthquake (DE) for a stiff soil type (class D) typically referenced by building codes. The Main Street recording station was installed in 2012 so no recordings of the 2000 Yountville earthquake were available.

## 2.1 Fire Station Area

This area has many classic U.S. subdivision-type tract homes build circa 1970, and therefore they should have had chimneys reinforced and anchored as required by the building code. Figure 4 depicts chimney features common to many homes in the area. This area experienced significant peak ground motions during the

Yountville (0.51g, 16 in/s) and South Napa (0.43g, 37 in/s) earthquakes. Building permit records indicated that chimneys were damaged in both events. Noteworthy was a cluster of homes located on three blocks within 500 feet southwest of the fire station having 17 chimneys damaged in the Yountville earthquake, but the same area had virtually no damage from the South Napa earthquake (Figure 5). Analyses presented below suggests a likely reason for the discrepancy was poor construction quality and/or lack of adherence to building code provisions at the time of original construction with better repair/replacement construction quality after the Yountville earthquake.




(a) (b)

Fig. 4 – Typical brick masonry fireplace and chimney associated with homes near the Fire Station recording station. (a) Chimney damaged and rebuilt after Yountville earthquake that survived the South Napa earthquake unscathed (home built in 1973), and (b) Section view depicting typical features.


Fig. 5 – Aerial view of Fire Station area depicting homes on three streets having chimneys damaged by the Yountville earthquake (denoted by circles). Investigation shortly after the South Napa earthquake found only one damaged chimney (denoted by “1” in the image).

## 2.2 Main Street Area

This area is the original city center and has many of the oldest structures in Napa. It contains an eclectic mix of buildings and houses with the majority appearing to be built prior to 1960 with some being as old as from the 1880s. Building permit records indicated that numerous chimneys were damaged in both the Yountville and South Napa earthquakes. Because of their age, it is likely that most of the masonry chimneys were not reinforced at the time of the original construction. The South Napa earthquake recorded peak ground acceleration (PGA) and velocity (PGV) respectively of 0.61g and 19 in/s in this area. Figure 6 shows several photos of one home that suffered chimney collapse during the earthquake. Figure 7 shows examples of other chimneys in the area. Noteworthy is the home in Figure 7a that had its chimney rebuilt after damage from the Yountville earthquake and it was unscathed by the South Napa earthquake.

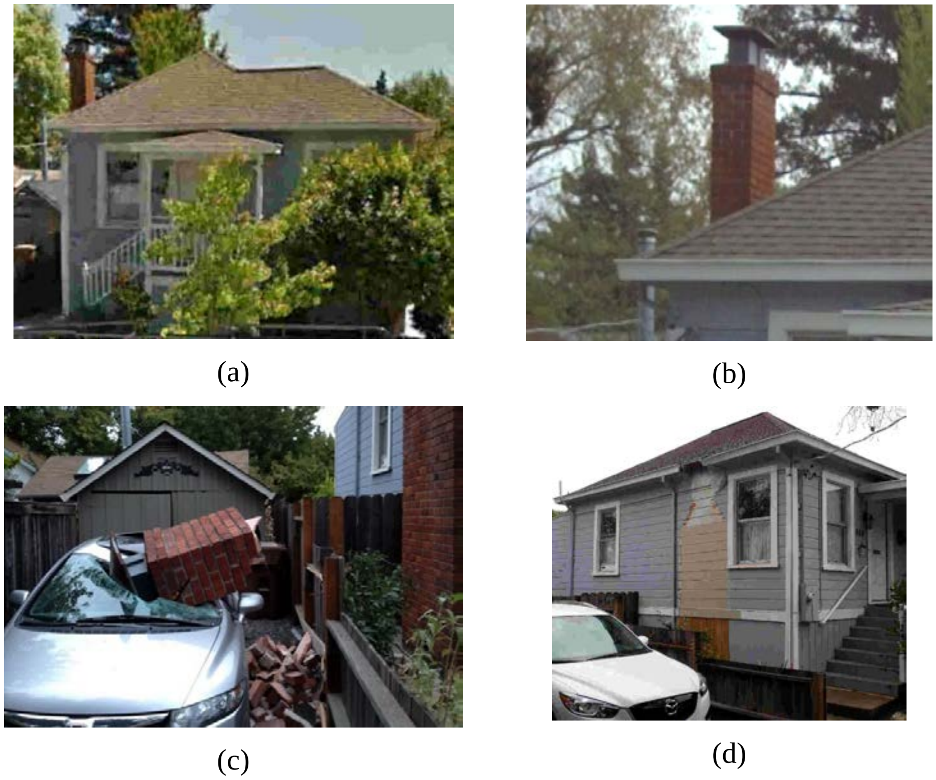



Fig. 6 – Chimney damage for home located southwest of downtown Napa (home built in 1902). (a) House and chimney before earthquake, (b) Close-up view of chimney portion above roofline, (c) Damage to auto from collapse of upper portion of chimney during South Napa earthquake (house at right), and (d) Photo taken several months after earthquake showing house after removal of fireplace and chimney.


Fig. 7 – Examples of chimneys located within 3,000 feet southwest of the Main Street recording station. (a) Chimney rebuilt after damage from Yountville earthquake, and was unscathed by the South Napa earthquake. This home, built in 1922, is next door to the home that suffered chimney collapse during the South Napa earthquake (visible in lower right; also shown in Figure 6). (b) Chimney damaged by South Napa earthquake and repaired. Note fresh bricks and mortar at upper portion of chimney. Home built in 1940.

## 3. Earthquake Response Simulations

This section presents computer simulations of chimney response to gain insights about factors affecting behavior. The scenario considered was that of unreinforced (plain) masonry chimney having flexural cracking and then rocking at the house roofline elevation. This could be the case where an exterior chimney is anchored to the home at the roofline by strapping and/or roof flashing. Or it could be where the chimney is located within the roof footprint so it is laterally supported on all sides at the point it extends through the roof. These situations are very common, but other failure modes do exist (e.g., failure just above the smoke chamber when the chimney is located at the house exterior with insufficient anchorage to the house). Solution was for chimney lateral displacement as an indicator of chimney flexural cracking, rocking, and toppling. A custom computer program was created to solve the equations of motion using the technique by Newmark [8].

### 3.2 Simplified Computer Model

The upper portion of the chimney above the roofline was modeled as a generalized single-degree-of-freedom (SDOF) system by a linear shape function describing the lateral displacement (Figure 8). A nonlinear lateral force-deformation relationship was formulated based on traditional strength-of-materials engineering principles. The chimney was assumed linear-elastic until it cracked after which it reverted to rocking behavior (Figure 9). The ultimate lateral forces were taken as the ultimate moments at the crack plane divided by the distance to the chimney mid-height (Figure 10). When linear-elastic, 3% damping was assumed, and when cracked, energy was dissipated only when rocking thru zero displacement based on the energy loss relation for rocking of rigid blocks by Housner [9]. The generalized mass and seismic loading was formulated by integration of the shape function and mass distribution over the height using standard textbook techniques [10]. The seismic input was the absolute accelerations as computed at the house roofline (location of the chimney crack plane).

The house was also modeled as a generalized linear-elastic SDOF system by a linear shape function describing the lateral displacement (i.e., the house acts like a shear-building having a straight-line lateral deflected shape). Post-earthquake reconnaissance surveys often observe damaged chimneys on homes having no other apparent damage, thus indicating the peak house drifts must have been small (say < 1%) so idealizing the house as a linear-elastic SDOF was deemed reasonable. It was assumed that the mass of the house was much greater than that of the chimney so the absolute accelerations at the house roofline elevation ($a_h$ in Figure 11) were input to the SDOF chimney model (no other house-chimney interaction).

The model is planar thus lacking bi-directional and vertical earthquake input likely making it somewhat un-conservative (analysis results likely over-state chimney ruggedness). Nevertheless, the model had a useful role in coherently examining several key aspects affecting chimney seismic performance.

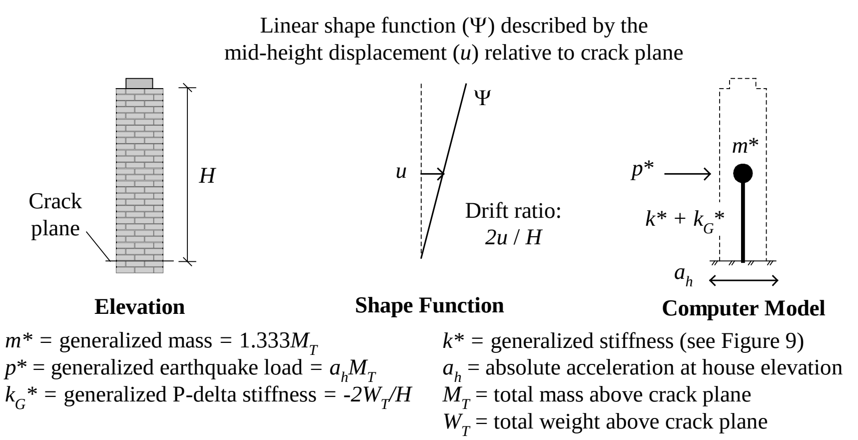



Fig. 8 – Computer model idealization of chimney.

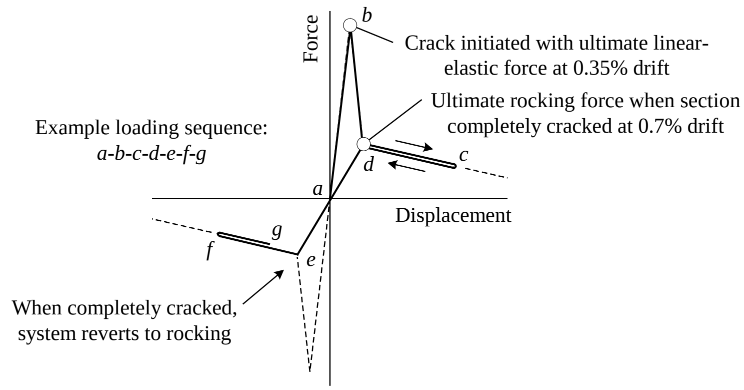



Fig. 9 – Chimney lateral force-deformation relation describing generalized chimney spring ($k^* + k_G^*$).

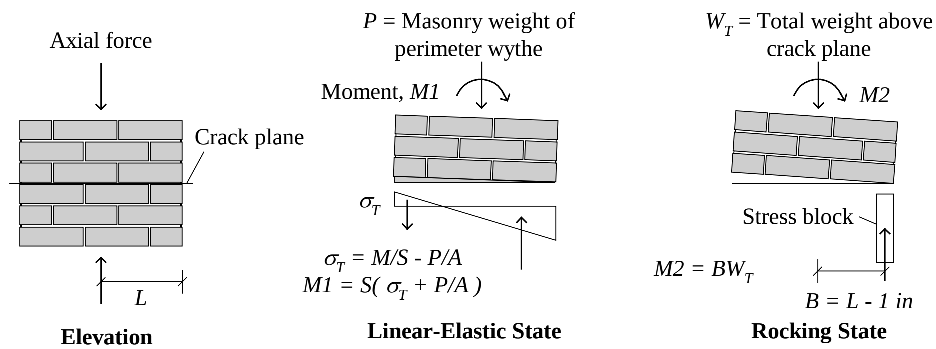



Fig. 10 – Chimney moments at crack plane.

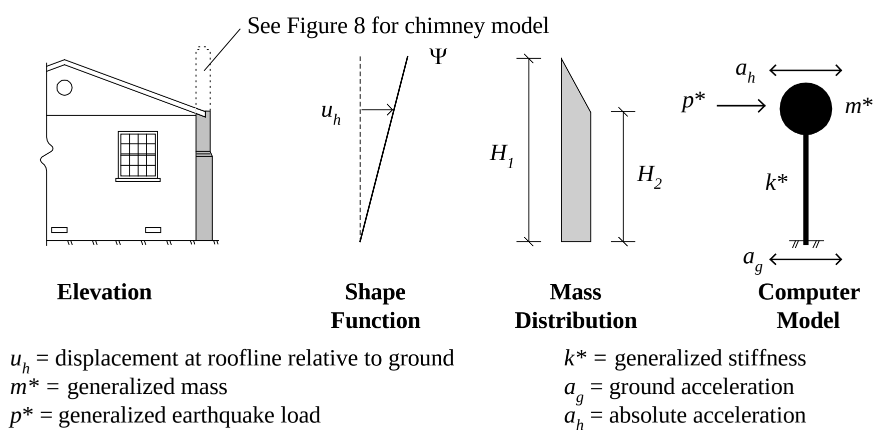



Fig. 11 – Computer model idealization of house. This model was used to generate the absolute acceleration time history at the roof level ($a_h$) that was input to the chimney model (Figure 8).

## 3.3 Analysis Approach

Incremental dynamic analyses (IDA) were performed on models representing chimneys in the two areas. IDA involves a series of analyses with the earthquake acceleration records scaled to have progressively increasing intensities. The resulting IDA graphs of peak chimney drift (defined in Figure 8) versus earthquake scale factor provide insight as to the chimney behavior over a range of shaking intensities. Two parameters were varied: masonry ultimate flexural tensile stress and the natural frequency of the house.

Three values of masonry ultimate flexural tensile stress were considered: 10 psi, 45 psi, and 60 psi (1 MPa = 145 psi). The in-situ ultimate flexural tensile strength can vary considerably due to the influence of factors such as the surface roughness of the bricks, mortar composition and moisture content; as well as the workmanship of the bricklayer. ASCE 41-13 [11] suggests default values of 45 psi for a lower bound and 60 psi for an expected value (ASCE 41-13 Tables 11-1 and 11-2(a)). However, older masonry (circa 1900) constructed with archaic lime mortars have much less strength.

Three house conditions were used: a rigid house and flexible houses having fundamental frequencies of 6 Hz and 3 Hz (5% damping). Homes consisting of wood construction have fundamental frequencies that can vary considerably. For example, Kharrazi and Ventura [12] report frequencies ranging from 0.85 Hz (non-engineered) to 5.6 Hz (engineered with stucco) from forced vibration tests of woodframe residential construction. The ASCE 7-10 [13] design period equation 12.8-7 yields a frequency of 6.9 Hz for woodframe buildings having heights of 14 feet. The CUREE-Caltech Woodframe project equation has a frequency of 7.3 Hz for buildings having heights of 14 feet [14]. The FEMA/NIBS earthquake loss estimation methodology (HAZUS computer program) assumes an effective frequency of 2.9 Hz for so-called W1 type woodframe buildings having heights of 14 feet [15].

## 3.4 Fire Station Area Analyses

The house-chimney arrangement shown in Figure 4 is used as the case study. The chimney was assumed to extend five feet (1 meter = 3.28 feet) above the roofline with plan dimensions of 21 by 21 inches (1 inch = 2.54 cm). The distance from the ground level to the roofline (chimney crack plane) was assumed as 9 feet. The model ultimate strengths are in Table 1. Note that masonry even with a relatively low ultimate flexural tensile stress (10 psi) has an ultimate lateral strength (610 lbs) that is 1.5 times that (400 lbs) for the chimney in a rocking state.

Table 1 – Ultimate strengths for case study chimneys.

| | Fire Station Area Case Study | | Main Street Area Case Study | |
|---|---|---|---|---|
| Property | Ultimate Lateral Force (lbs) | Ratio | Ultimate Lateral Force (lbs) | Ratio |
| Rocking | 400 | 1.0 | 340 | 1.0 |
| 10 psi | 610 | 1.5 | 490 | 1.4 |
| 45 psi | 2,100 | 5.3 | 1,700 | 5.0 |
| 60 psi | 2,800 | 7.0 | 2,200 | 6.5 |

Rocking = rocking state ultimate force; 10 psi = linear-elastic state ultimate force with masonry having 10 psi ultimate flexural tensile stress (typ.); and Ratio = ratio to rocking state force. 1 lb = 4.45 N

Analyses were performed using the North-South (NS) acceleration components recorded during the Yountville and South Napa earthquakes. Sample time history response is shown in Figure 12. Note the dramatic change in the chimney response after it cracks by having relatively large amplitude low frequency oscillations of about 1 Hz. The rocking frequency depends on displacement amplitude, and energy loss (damping) occurs only when the chimney rocks (impacts) through zero displacement. The computer analysis suggests the cracked chimney could withstand several cycles of large amplitude oscillations and remain standing. However, it is doubtful that an actual masonry chimney could undergo many large amplitude cycles without toppling due to mortar crushing at the rocking pivot points, and twisting of chimney from 3-direction shaking.

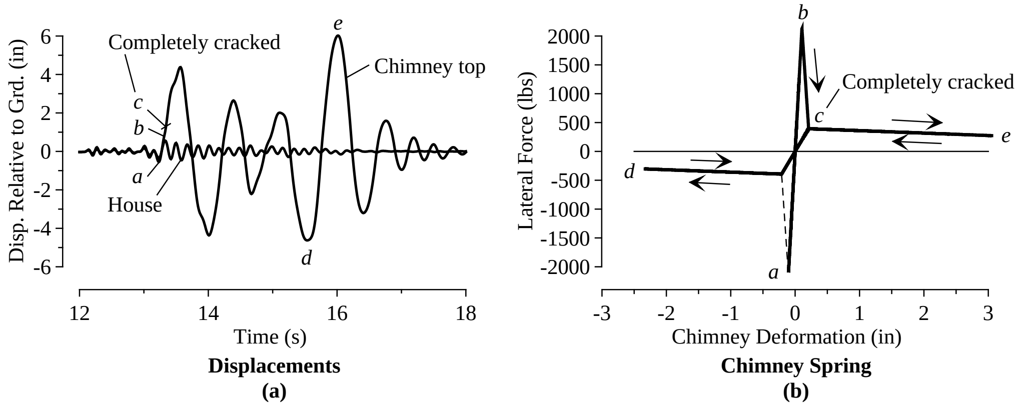



Fig. 12 – Sample analysis results for case of chimney with 45 psi masonry tensile strength attached to 6 Hz house subjected to 1.5 times the Yountville earthquake.

The IDA graphs tended to be irregular at larger drifts (say greater than 10%) when the chimney was in a rocking state after cracking (Figure 13). They also typically had an abrupt increase in peak drift for cases when the chimney cracked with this effect being more pronounced with increasing ultimate elastic strengths. Hence, chimney failure was taken here as when the drift exceeded 2%. The rationale was that beyond 2%, the response was sensitive to small changes in shaking intensity and other effects such as mortar crushing at the pivot point toe as well as out-of-plane effects (twisting from 3-direction shaking) might govern. ASCE 41-13 cites references indicating that twisting of masonry piers at their bases can influence performance when drifts are greater than 1.5% (C11.3.2.3.2).

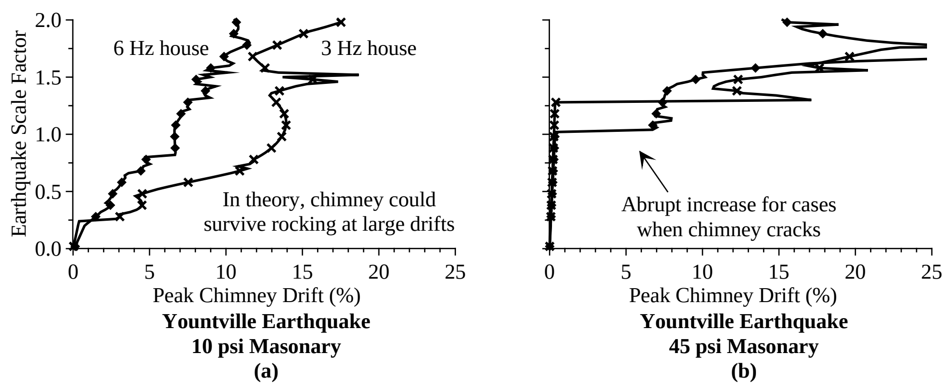



Fig. 13 – IDA graphs for chimney located in Fire House area. Chimney drift is defined in Figure 8.

Figure 14 shows the IDA graphs plotted out to 5% drift. Masonry flexural strength (Table 1) had a strong effect on chimney performance. For masonry having 10 psi strength, the chimney failed at both the Yountville and South Napa earthquake intensities (scale factor = 1.0), independent of the house frequency (Figures 14a and 14c), and vice versa for 45 psi strength masonry (Figures 14b and 14d). House flexibility also influenced performance. The IDA graphs with 6 and 3 Hz houses were below that from the rigid house case indicating that house flexibility reduced chimney ruggedness. This is because the vibration at the chimney crack plane is influenced by the house response. For example, the 3 Hz house had a peak absolute acceleration of 1.2g at the house roofline (at chimney crack plane) from the Yountville earthquake representing an amplification of 2.4 times the peak ground acceleration (0.51g).

Considering the age of the homes (built in the 1970s), the building code would have required the chimneys to have vertical steel reinforcement, and therefore an ultimate strength much greater than that for the 45 psi masonry case (by about eight times assuming the rebar develop full yield strength). Hence, they should have survived the subject earthquakes mostly unscathed suggesting that quality of construction (or lack thereof) was an important factor in the actual chimney damage from the Yountville earthquake. Chimneys rebuilt after the event had more stringent requirements (e.g., city inspection sign-off verifying construction including steel reinforcement) and this could explain why fewer were damaged during the South Napa earthquake (Figure 5).

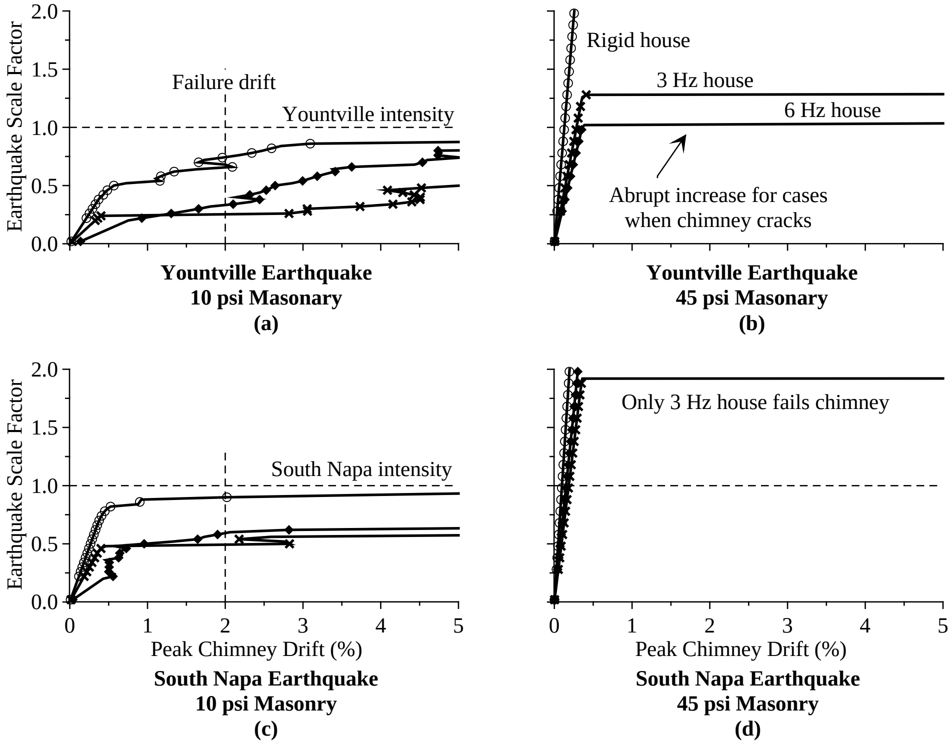



Fig. 14 – IDA graphs for chimney located in Fire Station Area.

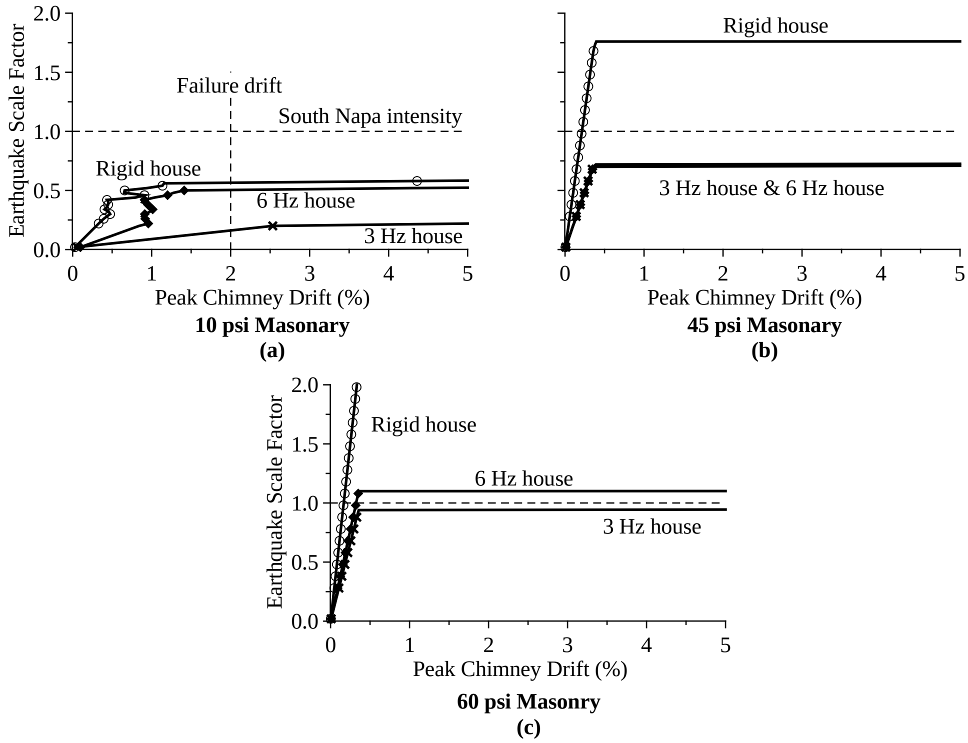



Fig. 15 – IDA graphs for chimney located in Main Street Area.

### 3.5 Main Street Area Analyses

The house-chimney arrangement shown in Figure 6 is used as the case study. The chimney was assumed to extend five feet above the roofline with plan dimensions of 16.5 by 25.5 inches. The distance from the ground level to the roofline (chimney crack plane) was assumed as 12 feet. The model ultimate strengths are contained in Table 1. Figure 15 shows the IDA graphs using the NS acceleration component recorded during the South Napa earthquake. Like the Fire Station chimney case study, both the masonry strength and house flexibility influenced chimney performance. However, in this case the chimney would survive the South Napa earthquake (scale factor = 1.0) only if it were connected to a rigid house or perhaps had 60 psi masonry. Considering the age of the home (built in 1902), the chimney undoubtedly lacked steel reinforcement and was constructed with lime mortars having low strength. Hence, the behavior might be like the 10 psi masonry case that indicates chimney failure at shaking intensities less than about one-half that from the South Napa earthquake (Figure 15a). This agrees with the observed actual collapse (Figure 6).

The neighboring home had its chimney rebuilt including reinforcement (Figure 7a), and the chimney survived the South Napa earthquake unscathed. Its strength would be greater than the 60 psi masonry case and the IDA graphs indicate it would easily survive the earthquake which agrees with observation (Figure 15c).

## 4. Implications for Fragility Function Development

Existing masonry chimney fragility functions typically differentiate across few parameters such as height, and are based on a single demand parameter such as peak ground acceleration [2, 16]. Their basis is mostly empirical statistical analysis of observed earthquake damage combined with engineering judgment. The simple gross nature of such fragility functions is understandable considering the limitations of the data sets used in their formulation usually lacking detailed information about the chimneys and the shaking intensity particular sites. Computer simulations using simplified house-chimney models like those described here offers a practical alternative way to develop improved fragility functions that explicitly account for parameters such as chimney height, section dimensions, and masonry tensile strength. The functions also require development using suites of records instead of a few site-specific records used here. The functions could then be incorporated into regional earthquake loss studies to potentially better quantify damage to chimneys.

Figure 16 depict sample results from the authors' ongoing analytical studies. Figure 16a shows a damage function formulation. Chimney extensive damage was taken as when the drift exceeded 2%. The function was defined by the shaded envelope of 12 data points (3 masonry strengths x 2 house frequencies x 2 earthquake record suites). Chimneys having parameters that lie within the function (shaded envelope) are assumed to have a 50% chance of suffering extensive damage. Chimneys having parameters that lie below the function have less than a 50% chance of damage, and those that lie above have greater than a 50% chance. Figure 16b shows fragility curves derived from the damage function median values at 45 psi masonry tensile strength.

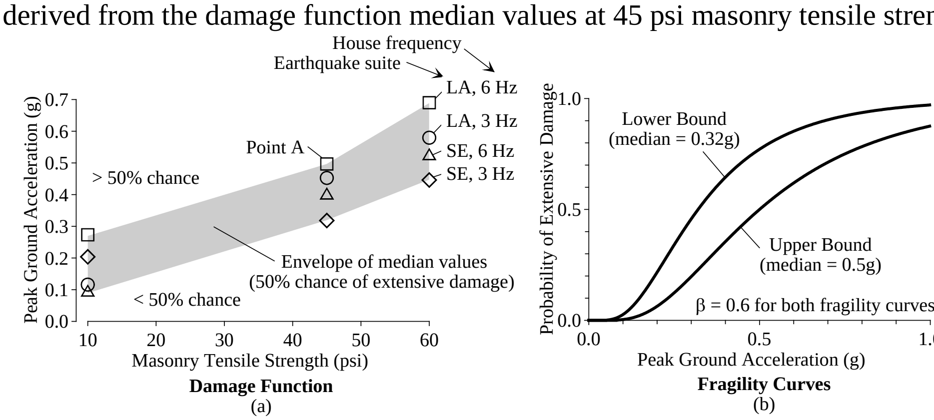



Fig. 16 – Damage function and fragility curves. (a) Illustration of damage function formulation as the envelope of analysis results (shaded area). Shown is the case of unreinforced chimneys having 5-foot heights above roof and 21-by-21 inch section. LA and SE suites refer to sets of records for characteristic earthquakes in Los Angeles and Seattle, respectively. (b) Fragility curves for 5-foot chimneys having 45 psi masonry strength.

## 5. Conclusion

The simplified house-chimney model presented here appears capable of capturing in a general way, the actual observed chimney response during recent earthquakes. The analysis results suggest that actual chimney damage can be quite variable due to the influence of many parameters including: flexural strength of masonry, natural frequency of house, and character of ground shaking. Improved analytical-based chimney fragility functions can be developed by including these parameters. This work is currently underway.

## 6. Acknowledgement

The authors are members of the Existing Buildings Committee (EBC) of the Structural Engineers Association of Northern California (SEAONC). The study herein was undertaken as a public service to better understand chimney behavior during earthquakes. The authors are most appreciative of the contribution of Kelly Cobeen (Wiss, Janney, Elstner Associates, Inc.) in providing interest and discussions about this study.

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