Chapter 7

Discussion and Conclusions

7.1 Discussion

Experience from historic earthquakes provides evidence for seismic building response. Some classes of buildings are known to be unsafe because they have performed poorly in the past. Unreinforced masonry buildings crumbled in the 1906 San Francisco earthquake. The 1971 San Fernando earthquake exposed the problem of non-ductile concrete frames. Long-period resonance in the sedimentary basin under Mexico City significantly damaged many, and collapsed some, mid-rise buildings in 1985. The 1994 Northridge earthquake demonstrated the widespread problem of steel moment-resisting frames with brittle welds. These and other experiences motivated immediate study and eventual changes to the building code.

Computational and experimental models allow engineers to study building behavior from a physical understanding. Models are an efficient way to study various building systems in the absence of documented real-world behavior. An engineer can propose several designs and test each to provide a client with the best of all considered designs. Researchers generate new models to explain observed building behavior or predict future building response to seismic ground motions. Structural modeling provides an explanation for observed building behavior and helps to anticipate future problems.

7.1.1 Simulations as Proxies for Experience

This thesis anticipates what might be learned about steel moment frames in the next large earthquake near a major urban center. Simulations provide this next large earthquake in the form of ground motions on regional simulation domains. By applying these tens of thousands of ground motions to models of steel moment frames, this thesis predicts what may happen to this class of buildings in future earthquakes. This study can only account for known building behavior; it cannot anticipate unknown problems. However, based on the current best understanding of steel moment frame physics and ground motions from large earthquakes, this study provides important lessons before the next devastating earthquake.

To make accurate predictions of future building behavior, simulated ground motions should be consistent with recorded ground motions. The simulated ground motions used in this thesis have time histories consistent with recorded ones. However, there are some inconsistencies between the peak ground velocities (PGVs) of these simulated ground motions and the PGVs of ground motion prediction equations (GMPEs). For near-source sites in earthquakes of magnitudes between 6.3 and 6.8, the PGV of the simulated ground motions does not saturate as predicted by GMPEs. Also, for distant sites in earthquakes of magnitudes 7.7 and 7.8, the variance of PGV from simulated ground motions is larger than that defined in GMPEs. These inconsistencies do not affect the building response prediction equations developed in this thesis because I disregard magnitude and distance information for the simulated ground motions in the building response prediction models. Resolving these differences in PGV as a function of magnitude and distance between simulated and recorded ground motions may be of interest to the seismological community.

7.1.2 "Lessons Learned"

Fracture-prone welds significantly degrade a building's lateral force-resisting capacity. In the same ground motion, a steel MRF with brittle welds is 2–8 times more likely to collapse than the same building with sound welds. If the buildings remain standing, the peak inter-story drift ratio (IDR) of the building with brittle welds is likely 1.7 times the peak IDR of the perfect weld building. The design considerations of building height and the combination of strength and stiffness do not affect building response as significantly as the state of the welded connections. The results and predictive models in this thesis quantify the problem of brittle welds. A building owner contemplating a retrofit can better estimate the benefits of improved building performance due to fixing brittle welds.

This thesis also considers the seismic responses of shorter and taller steel moment frames. Properly designed and constructed short (as represented by six stories) and tall (as represented by twenty stories) buildings are approximately equally likely to collapse in the same strong ground motion. If both buildings with sound welds remain standing, the six-story building develops a peak IDR 1.3–1.6 times the peak IDR in a twenty-story building, depending on the stiffness and strength of the design. However, twenty-story buildings with brittle welds are up to five times more likely to collapse than six-story buildings with brittle welds, depending on the ground motion. If both buildings with brittle welds remain standing, the peak IDR of the six-story building is 1.2–1.4 times that of the twenty-story building, depending on the stiffness and strength of the design.

Taking a more philosophical point of view, consider brittle welds not as a known problem but rather as exemplary of an unknown problem. The problem of brittle welds was not widely known before the 1994 Northridge earthquake. If this study had been performed in 1994, there would have been no consideration of the brittle weld problem. The conclusion at the time would have been unqualified: shorter and taller buildings are approximately equally likely to collapse, and if they remain standing, six-story buildings develop larger peak IDRs. Today there is almost certainly an unknown problem with tall buildings. Considering the experience of historic earthquakes, it is inevitable that there are presently unknown problems. Newer systems are more likely to have unknown problems simply because previously unknown problems of older systems became known problems through experience. Taller buildings are less robust against these unknown problems; they are inherently more unstable than shorter buildings, so a design or construction flaw is more likely to catastrophically weaken a taller building compared to a shorter building. A taller building is riskier than a shorter building in an uncertain world.

The stiffness and strength of a building design affects building response, albeit less significantly than do weld state or building height. A more flexible, lower-strength design is 1–4 times more likely to collapse in strong ground motions than a stiffer, higher-strength design, depending on the ground motion. If both designs remain standing in a ground motions, the stiffer, higher-strength design develops a smaller peak IDR than does the more flexible, lower-strength design, as expected.

A great earthquake is literally and figuratively an order of magnitude larger than a large earthquake. The areal extent of large building responses to the simulated earthquakes in this thesis is qualitatively different for large and great earthquakes. Compare the building responses in the 1989 Loma Prieta (Figure 3.4) and 1906 San Francisco (Figure 3.6) earthquakes, for example. Inelastic building responses dominate the simulation domain in the three magnitude 7.8 simulations, whereas the area of inelastic building response in the magnitude 6.9 simulations is much smaller. Specifics of the assumed slip model and rupture propagation alter the ground motions, and thus the building responses, at a particular site. However, the regional extent of large building responses remains similar for similar magnitude simulations.

7.1.3 Building Response Prediction Models

The building response prediction models developed in this thesis succinctly characterize the simulated steel moment frame response data. The prediction models quantify in a probabilistic sense the seismic response as collapse, total structural loss, and if repairable, the peak inter-story drift ratio. As observed in the simulation data, the presence of brittle welds increases the probability of collapse; buildings with brittle welds tend to collapse in smaller ground motions. Applying the prediction models where there is little available data, the shape of the collapse separating contours are different for the stiffer, higher-strength designs versus the more flexible, lower-strength designs. For ground motions with small peak ground displacement (PGD) and large peak ground velocity (PGV), the stiffer designs are less likely to collapse, whereas for ground motions with large PGD and small PGV, the more flexible designs are less likely to collapse. At a given intensity measure, a six-story building is less likely to collapse than a twenty-story building. Also, for most ground motions, the more flexible designs with either building height and perfect welds are less likely to be a total structural loss than the other considered buildings.

Different intensity measures are more appropriate for predicting different building responses. Pseudo-spectral acceleration (PSA) unquestionably best predicts the peak inter-story drift ratio if the building is not a total structural loss in a particular ground motion. Not surprisingly, for elastic and mildly inelastic building response, PSA accurately characterizes the response. To predict total structural loss, however, PGD and PGV do as well as PSA, and to predict collapse, PGD and PGV are superior to PSA. As the building deformations become more severe, the character of the building changes, and the information from a spectral quantity no longer applies to a damaged building. More broadband intensity measures, such as PGD and PGV, better characterize the ground motion for steel moment frame collapse and total structural loss.

The intended use of the collapse, total structural loss, and peak inter-story drift prediction models is in series: for a given ground motion, predict the probability of collapse and total structural loss; and then, assuming that the building is repairable, predict the peak inter-story drift ratio. In this way, these models predict the state of a steel moment frame building, explicitly acknowledging the possible collapse and total structural loss responses which are often overlooked. These prediction models have not been validated with the experience of steel moment frames in historic earthquakes. Data from 2,000 (six-story buildings) and 20,000 (twenty-story buildings) simulations of steel frame building response support these prediction models.

These building response predicition models can be used in conjunction with seismic hazard, damage, and loss analyses to probabilistically characterize the full performance of steel moment frames in seismically active areas. For example, they could be used for a simplified structural analysis in the modular procedure for performance-based earthquake engineering presented in Goulet et al. (2007). Also, a building owner contemplating the replacement of fracture-prone welds with sound welds can quantify the reduced probability of collapse or total structural loss of the building. At an initial design stage, engineers can compare the relative performance of different building heights and designs with different strength and stiffness combinations for the seismic hazard at the proposed site. These building response prediction models should not replace full nonlinear time history analyses of buildings when such detailed analyses are warranted.

7.2 Conclusions

- The presence of fracture-prone welds in a steel moment frame significantly degrades the seismic response. Steel moment frames with brittle welds collapse in weaker ground motions than do moment frames with sound welds. For the same ground motion, fixing brittle welds reduces the probability of collapse by a factor of 2–6 and reduces the median peak inter-story drift ratio by a factor of 0.59.

- In general, properly designed and constructed shorter steel moment frames are slightly less likely to collapse than taller buildings in a given strong ground motion. If the buildings remain standing, then the six-story building develops larger peak inter-story drifts. Taller buildings are less robust to flaws such as brittle welds: a taller building with brittle welds is 2–4 times more likely to collapse compared to a shorter building with brittle welds.

- The stiffness and strength of a steel moment frame design affect seismic response less than welds state and building height do. A more flexible, lower-strength design is 1–4 times more likely to collapse compared to a stiffer, higher-strength design. As expected, if the designs remain standing, the median peak inter-story drift ratio in the stiffer design is 0.75 times that of the more flexible design.

- The experience of a large earthquake (magnitude between 7.0 and 7.5) does not predict the experience of a great earthquake (magnitude greater than 7.5).

- Building response prediction models can estimate probabilistically the likelihood of collapse and total structural loss, as well as peak inter-story drift ratio (assuming the building is repairable) of steel moment frames. I recommend Model 3 in Equation 6.7 to predict the collapse and total structural loss states. Model 1 in Equation 6.8 should be used to predict the peak inter-story drift ratio, assuming the building is repairable. These prediction models can be incorporated into broader studies of damage and loss for this class of buildings in seismically active areas.

- For steel moment frames, a combination of peak ground displacement and peak ground velocity predicts the likelihood of collapse better than pseudo-spectral acceleration. PSA predicts total structural loss equally well as PGD and PGV. Assuming the building is not a total structural loss, however, PSA predicts the peak inter-story drift ratio better than a combination of PGD and PGV.

- Future engineering studies would benefit from a resolution of the differences between simulated ground motions and ground motion prediction equations. For magnitudes between 6.3 and 6.8, this thesis shows that the long-period peak ground velocities of the simulated ground motions considered here are larger than those expected by ground motion prediction equations.