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SHAKEOUT2008: TALL STEEL MOMENT FRAME BUILDING RESPONSE

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ABSTRACT

In 2008, there was a significant campaign undertaken in southern California to increase public awareness and readiness for the next large earthquake along the San Andreas fault that culminated in a large-scale earthquake response exercise. The USGS ShakeOut scenario was a key element to understanding the likely effects of such an event. In support of this effort, a study was conducted to assess the response of tall steel structures to a M7.8 scenario earthquake on the southern San Andreas Fault. Presented here are results for two structures. The first is a model of an 18-story steel moment frame building that experienced significant damage (fracture of moment-frame connections) during the 1994 Northridge earthquake. The second model is of a very similar building, but with a structural system redesigned according to a more modern code (UBC 97). Structural responses are generated using three-dimensional, non-linear, deteriorating finite element models, which are subjected to ground motions generated by the scenario earthquake at 784 points spaced at approximately 4 km throughout the San Fernando Valley, the San Gabriel Valley and the Los Angeles Basin. The kinematic source model includes large-scale features of the slip distribution, determined through community participation in two workshops and short length-scale random variations. The rupture initiates at Bombay Beach and ruptures to the northwest before ending at Lake Hughes, with a total length of just over 300 km and a peak slip of 12 m at depth. The resulting seismic waves are propagated using the SCEC community velocity model for southern California, resulting in ground velocities as large as 2 m/s and ground displacements as large as 1.5 m in the region considered in this study. The ground motions at the sites selected for this study are low-passed filtered with a corner period at 2 seconds. Results indicate a high probability of collapse or damage for the pre-1994 building in areas of southern California where many high-rise buildings are located. Performance of the redesigned buildings is substantially improved, but responses in urban areas are still large enough to indicate a high-probability of damage. The simulation results are also used to correlate the probability of building collapse with damage to the structural system.

Introduction

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In order to prepare for the next big earthquake on the San Andreas fault, the US Geological Survey (USGS) conducted a year-long "DARE TO PREPARE" campaign that culminated in the Great Southern California Shakeout Scenario in 2008, a large-scale earthquake response exercise. A magnitude 7.8 earthquake on the southern San Andreas Fault was chosen as the scenario event and a detailed, realistic source model for the event was generated (Hudnut et al. 2007) and used to create simulated ground motions at locations throughout Southern California (Graves et al. 2008). In support of this effort, we were charged by the USGS with developing a plausible realization of the response of tall steel buildings to the scenario event. Toward this end, we analyzed three steel moment frame buildings in the 20-story class, orienting them in two different directions, considering perfect and imperfect realizations of beam-to-column connection behavior, subjecting them to the simulated 3-component ground motions at each of 784 analysis sites in the San Fernando Valley, the San Gabriel Valley and the Los Angeles Basin spaced at approximately 4 km, as shown in Figure 1. We used the modeled building performance in these 12 cases (3 buildings x 2 orientations x 2 connection susceptibility realizations) to provide a qualitative picture of one plausible outcome in the event of the big one striking southern California.

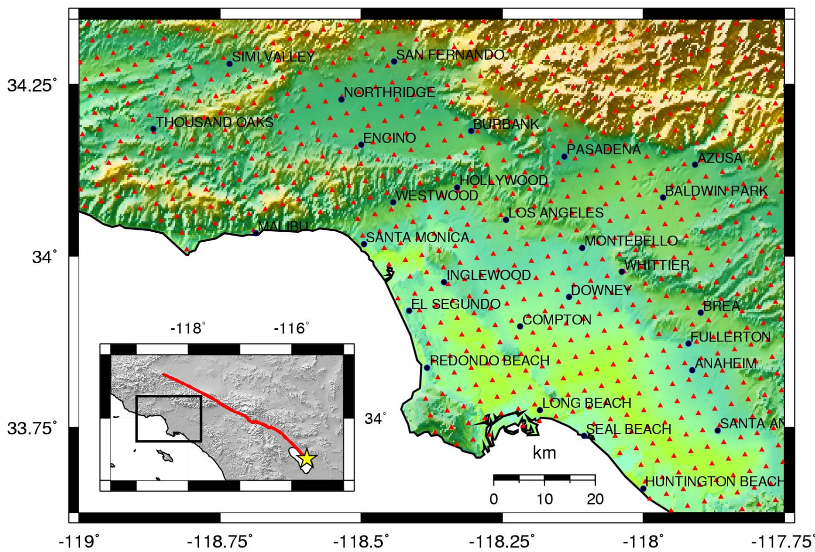


Figure 1. Geographical scope of study area: Triangles represent sites where building time-history analyses are performed. The inset shows the study area in relation to the rupture trace. The star represents the epicenter of the earthquake.

Scenario Earthquake

The scenario event is a magnitude 7.8 earthquake on the San Andreas fault with rupture initiating at Bombay Beach and propagating northwest a distance of roughly 304 km, terminating at Lake Hughes near Palmdale, as shown in the inset of Figure 1. The source model developed by Hudnut et al. (2007) is based on a wide variety of observations and constraints. Using this source model, Graves et al. (2008) has simulated 3-component seismic waveforms on a uniform grid covering southern California. The SCEC Community Velocity Model (Magistrale et al. 1996; Magistrale et al. 2000; Kohler et al. 2003), which allows for the modeling of the basin response down to a shortest period of approximately 2s, was used for the ground motion simulations. Figures 2(a) and 2(b) show the peak velocities of the simulated waveforms in the east-west and north-south directions, respectively. Peak velocities are in the range of 0-100 cm/s in the San Fernando valley, and 60-180 cm/s in the Los Angeles basin. Corresponding peak displacement ranges are 0-100 cm and 50-150 cm.

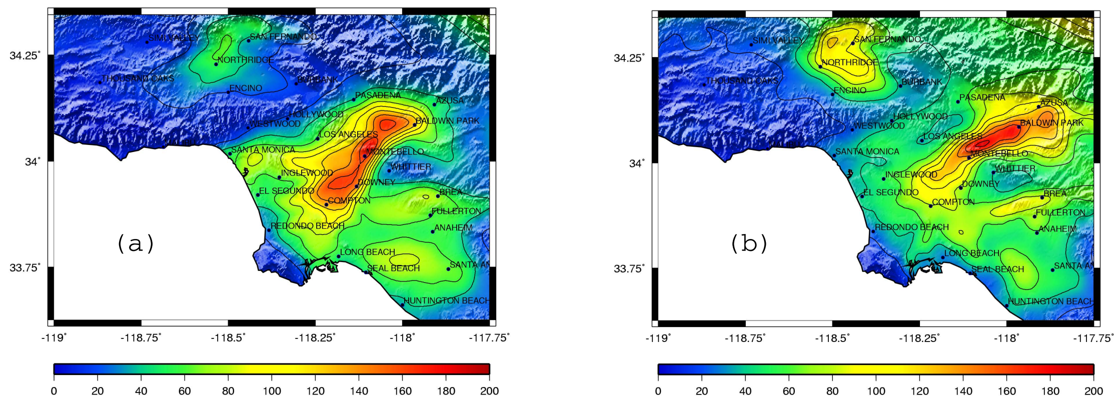
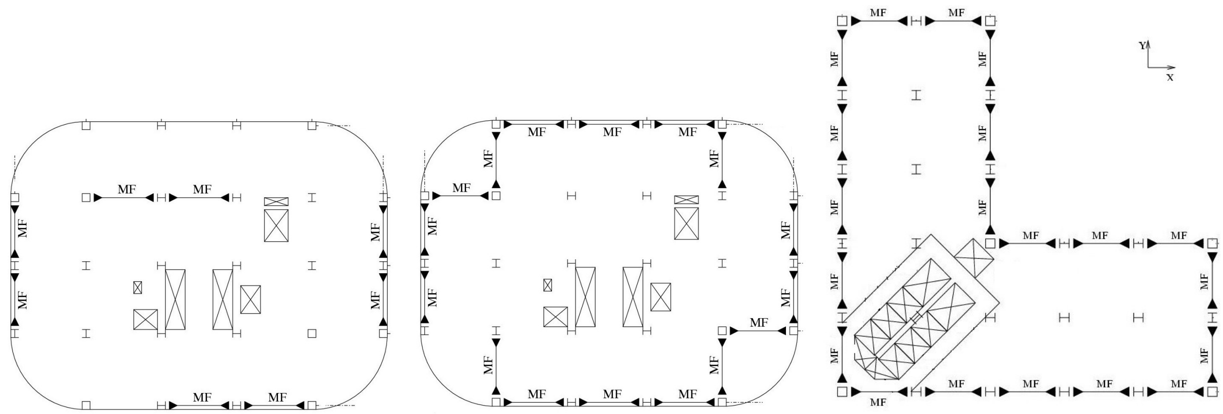


Figure 2. Peak ground velocities (cm/s) for simulated ground motions in the (a) east-west and (b) north-south directions.

Description of Modeled Buildings

Structural models of three buildings are subjected to ground motions at the 784 analysis sites. Building 1 is based on an existing 18-story office building located within five miles of the epicenter of the 1994 Northridge earthquake. It was designed according to the lateral force requirements of the 1982 UBC and construction was completed in 1986-87. The lateral force-resisting system consists of two-bay welded steel moment-frames, as shown in Figure 3(a). The location of the north frame one bay inside of the perimeter gives rise to some torsional eccentricity. Many moment-frame beam-column connections in the building fractured during the Northridge earthquake, and the building has been extensively investigated since then by many engineering research groups (SAC 1995). Building 2 is similar to Building 1, but the lateral force-resisting system has been redesigned according to the 1997 UBC. The new building has been designed for larger earthquake forces and greater redundancy in the lateral force-resisting system and the torsional eccentricity seen in Building 1 has been eliminated. Building 2 has 8 bays of moment-frames in each direction, as shown in Figure 3(b). Building 3 is L-shaped in plan, as shown in Figure 3(c). The UBC classifies such buildings as irregular and stipulates that they be designed for lateral forces that are approximately 10% larger than those prescribed for regular buildings. Detailed floor plans, beam and column sizes, and the gravity, wind and seismic loading criteria are given in Krishnan et al. (2006c) for Buildings 1 and 2 and in Krishnan (2003a, 2007) for Building 3.



(a) (b) (c)

Figure 3. Typical floor plans are shown for the three buildings modeled: (a) Building 1, an existing 18-story building designed according to the 1982 UBC; (b) Building 2, a redesigned version of Building 1 conforming to the 1997 UBC; and (c) Building 3, a 19-story L-shaped building designed according to the 1997 UBC. Bays marked "MF" indicate moment frames.

Finite-Element Analysis

Nonlinear damage analyses of the structures are performed using the program FRAME3D (Krishnan 2003b, 2009a). FRAME3D (http://virtualshaker.caltech.edu) incorporates geometric nonlinearity, which enables the modeling of the global stability of the building, accounting for P-Δ effects accurately. Beams are modeled using segmented elastofiber elements, with nonlinear end segments that are subdivided in the cross-section into a number of fibers, and an interior elastic segment; column elements include an additional nonlinear segment in the middle to enable modeling of column buckling (Krishnan 2009b). Beam-to-column joints are modeled in three dimensions using panel zone elements. These elements have been shown to simulate damage accurately and efficiently (Krishnan and Hall 2006a, 2006b). Material nonlinearity resulting in flexural yielding, strain-hardening, and ultimately rupturing of steel at the ends of beams and columns, and shear yielding in the joints (panel-zones) is included. The fracture mode of failure is also included, and used here to consider the effect of fracture-susceptible connections on overall building response. There is great uncertainty in the performance of the beam-to-column connections in welded steel moment frame buildings as evidenced in the 1994 Northridge earthquake. Two models are considered for each building, one with perfect connections, and the other with susceptible connections. Specifications (FEMA 2000a) developed by the Federal Emergency Management Agency (FEMA) for moment-frame construction following the Northridge earthquake should result in superior connection performance, and hence, the connections in the buildings designed according to UBC97 are assumed to be less vulnerable to fracture than for the older (pre- 1994) Building 1.

Building Performance

At each site, analyses were performed using FRAME3D for the three building models, with perfect and fracture-susceptible connections and in two different orientations (with the xaxis in Figure 3 oriented in the east-west direction and rotated 90 degrees for Buildings 1 and 2 and 45 degrees for Building 3) for a total of 9408 3-D nonlinear time-history analyses. In each case, detailed structural damage as well as the displacements and interstory drifts are calculated. To assess the performance of these buildings, we use the performance levels defined by FEMA 356 (FEMA 2000b): Immediate Occupancy (IO), where very limited structural damage has occurred; Life Safety (LS), a damage state that includes damage to structural components but retains a finite margin against collapse; and Collapse Prevention (CP), a damage state at which the structure continues to support gravity loads but retains no margin against collapse. For existing buildings, the interstory drift limits for the IO, LS, and CP performance levels specified by FEMA are 0.007, 0.025, and 0.05, respectively. In addition to these criteria, we assume that the buildings will be red-tagged (RT) if the peak interstory drift ratios exceed 0.05. If the peak interstory drift ratio exceeds 0.075 we assume that there is a great likelihood that the building has collapsed (CO). Maps of peak interstory drift ratios for the base orientation for the three buildings assuming fracture-susceptible connections are shown in Figures 4(a), 4(c) and 4(e). Corresponding maps assuming perfect connections are shown in Figures 4(b), 4(d) and 4(f). The color-coding on the maps follows the previously-described performance criteria. Results for building performance are summarized in Table 1. Building 1 exhibits the worst performance with the susceptible connection model collapsing at 18.3% of the analysis sites and being red-tagged at 11.7% of the sites. The L-shaped building 3 performs the best with the percentage of collapsed and red-tagged instances being 10.3% and 6.4%, respectively. The performance of Building 2 is only slightly worse than Building 3. If we assume that the beam-to-column connections are perfect, then there is a significant drop in the number of collapsed and red-tagged buildings. In the rotated orientation, performance is slightly worse for Buildings 1 and 2 and slightly better for Building 3, as shown in Table 1. However, the spatial contours of building performance in the corresponding peak interstory drift maps are not significantly altered from those shown in Figures 4(a)-4(f).

Table 1. Building performance in base and rotated orientations, with susceptible and perfect beam-to-column connections. Numbers indicate the percentage of analysis sites at which performance can be categorized as: (a) immediately occupiable (IO); (b) life-safe (LS); (c) collapse-prevention (CP); (d) red-tagged (RT); or (e) collapsed (CO).

Model	**Orientation**	**Connections**	**Performance Level**				
			IO	**LS**	**CP**	**RT**	**CO**
Building 1 (1982 UBC)	Base	Susceptible	5.2	28.3	36.5	11.7	18.3
		Perfect	5.4	29.7	46.0	11.9	7.0
	Rotated	Susceptible	4.8	29.7	33.8	7.5	24.2
		Perfect	4.9	31.0	42.2	10.7	11.3
Building 2 (1997 UBC)	Base	Susceptible	8.5	36.4	35.5	9.8	9.8
		Perfect	8.5	37.2	42.0	7.7	4.7
	Rotated	Susceptible	7.7	36.0	36.0	8.2	12.1
		Perfect	7.7	37.4	41.2	10.0	3.8
Building 3 (1997 UBC) (L-shaped)	Base	Susceptible	8.2	42.4	39.0	6.6	3.9
		Perfect	8.2	42.8	40.9	6.6	1.5
	Rotated	Susceptible	9.9	45.5	34.2	4.6	5.7
		Perfect	9.9	46.0	35.9	5.5	2.7

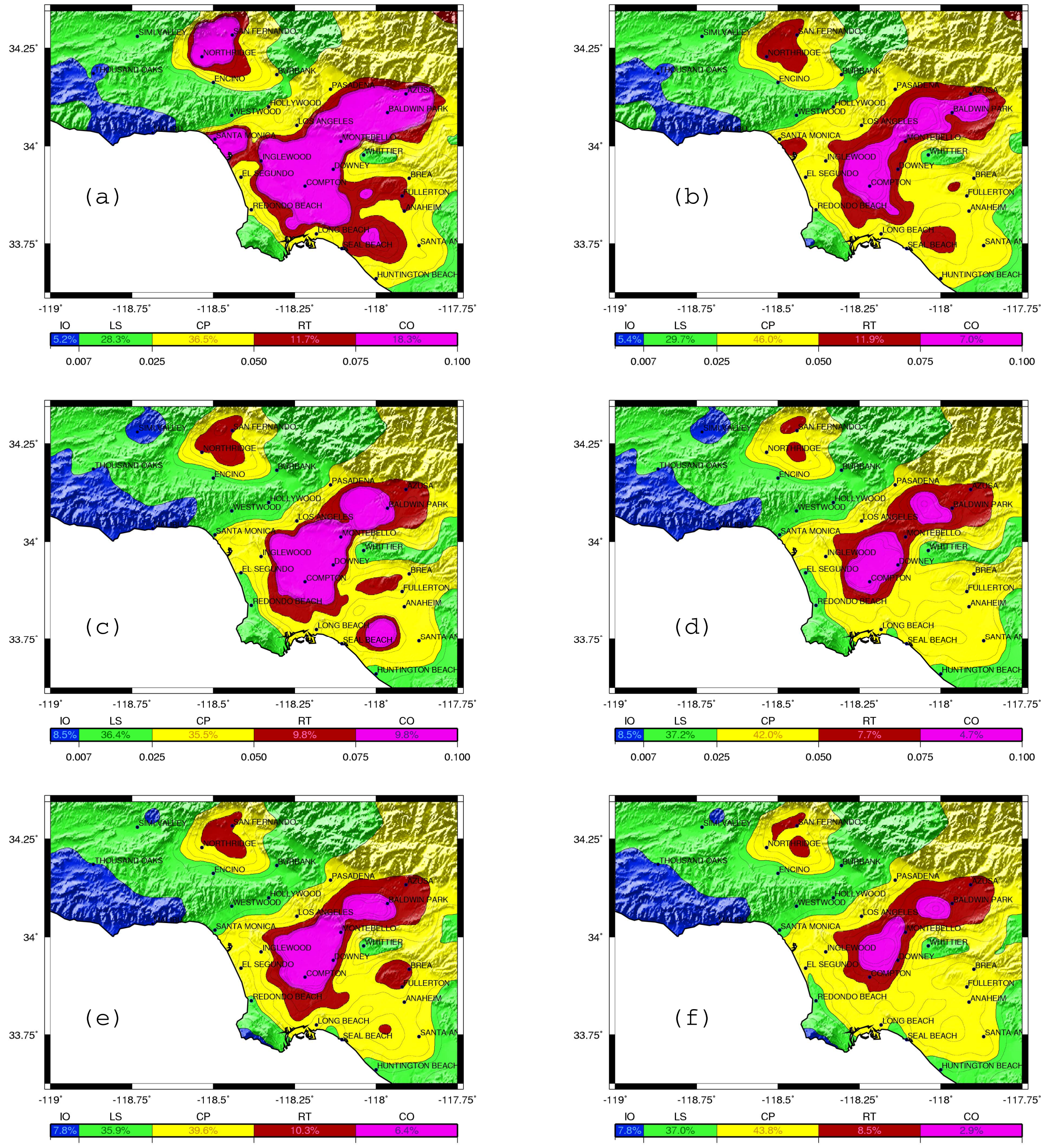


Figure 4. Maps of peak interstory drift for Building 1 with (a) susceptible and (b) perfect connections, Building 2 with (c) susceptible and (d) perfect connections, and Building 3 with (e) susceptible and (f) perfect connections. Color-coding corresponds to performance classification: Immediate Occupancy (IO); Life-Safety (LS); Collapse Prevention (CP); Red-Tagged (RT); Collapse (CO).

Conclusions

The location of tall buildings in the Los Angeles metropolitan area with 10 or more stories is shown in Figure 5. The size of circles shown in the figure is proportional to the number of stories. There are 489 buildings with 10-19 stories, 118 buildings with 20-29 stories, 28 buildings with 30-39 stories, 11 buildings with 40-49 stories, and 10 buildings with 50 or more stories. Many more are in the planning stages or under construction. It is clear that majority (607) are in the 10-30 story range. While a wide variety of structural systems are used in the buildings shown, we assume that approximately one-quarter of these buildings (150) utilize steel moment frames as the primary lateral force resisting system, similar to the buildings considered in this study. The buildings are clustered in small pockets that are aligned with the major freeways in the region. Most tall buildings have been built along Interstate freeway I-10 from Santa Monica to downtown Los Angeles, in the mid-Wilshire district along Wilshire Boulevard, and along State Highway 101 from Hollywood to Canoga Park in the San Fernando valley. In addition a few tall buildings are located along Interstate freeways, I-5 and I-405.

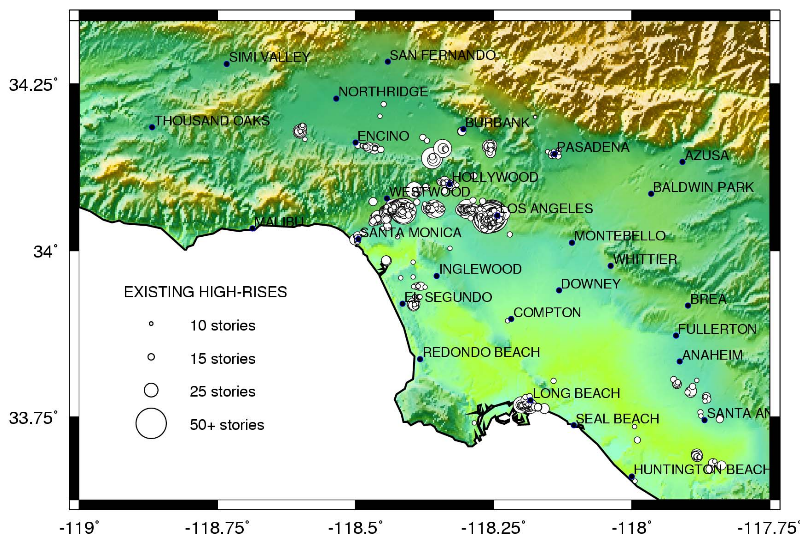


Figure 5. Distribution of tall buildings (10 stories or greater) in the Los Angeles metropolitan area as of mid-2007. Data source: Emporis.com by way of Keith Porter, University of Colorado at Boulder.

Figures 4(a) and 4(b) indicate that performance of the oldest design, Building 1, along the I-10, the Santa Ana-Anaheim corridor and Long Beach generally is classified as CP, with damage serious enough to cause loss of life, but without complete collapse. For Buildings 2 and 3, designed with UBC97, performance along much of the I-10 is improved to the LS damage state, though downtown Los Angeles remains classified as CP, as shown in Figures 4(c)-4(f). It is important to note that areas in the CP zone are within 10 km of the RT and CO zones. What this means is that given a different set of earthquake source parameters, it is entirely possible that at least some of these locations may end up in the red or pink zones indicating collapses or the need for red-tagging. As shown in Krishnan et al. (2006c) differences in the hypocenter location, slip distribution, rupture directivity, and the velocity model result in a dramatically different distribution of building damage. Bearing this in mind, we recommended that the ShakeOut drill be conducted assuming a damage scenario comprising of 5% of the estimated 150 steel moment frame structures in the 10-30 story range collapsing (8 collapses), 10% of the structures red-tagged (16 red-tagged buildings), 15% of the structures with damage serious enough to cause loss of life (24 buildings in the yellow zone with fatalities), and 20% of the structures with visible damage requiring building closure (32 buildings with visible damage and possible injuries).

Acknowledgments

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