Seismic Performance of Modern Commercial Low-rise Reinforced Concrete Frame Buildings in Los Angeles County

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SUMMARY:
A seismic performance assessment for a given site in seismically active regions is often performed on behalf of a buyer of commercial buildings. Historically, one outcome of such assessments is a probable maximum loss (PML). To make such evaluation over a seismically active area, one may use common risk assessment software such HAZUS-MH or similar tools, which contains pre-evaluated vulnerability and hazard information. However, such software relies significantly on expert opinion making it insensitive to many details that do affect estimated loss.

In this research, we explore the basis for such performance assessment procedure using a set of modern commercial low-rise RC frame buildings representative of recent engineering practice in Los Angeles County. A simulation-based framework for assessing PML of low-rise reinforced concrete frame buildings were developed for two hazard levels (DBE-10% probability of exceedance in 50 years and MCE-2% probability of exceedance in 50 years) by taking the variability of site location into account. We also investigated the probability of collapse (PC) for each building throughout the region. The generated seismic performance metric (PML and PC) maps can inform the stakeholders for emergency planning and for earthquake hazard mitigation.

Keywords: Urban hazard mitigation, Probable maximum loss, Low-rise reinforced concrete frame building

1. INTRODUCTION

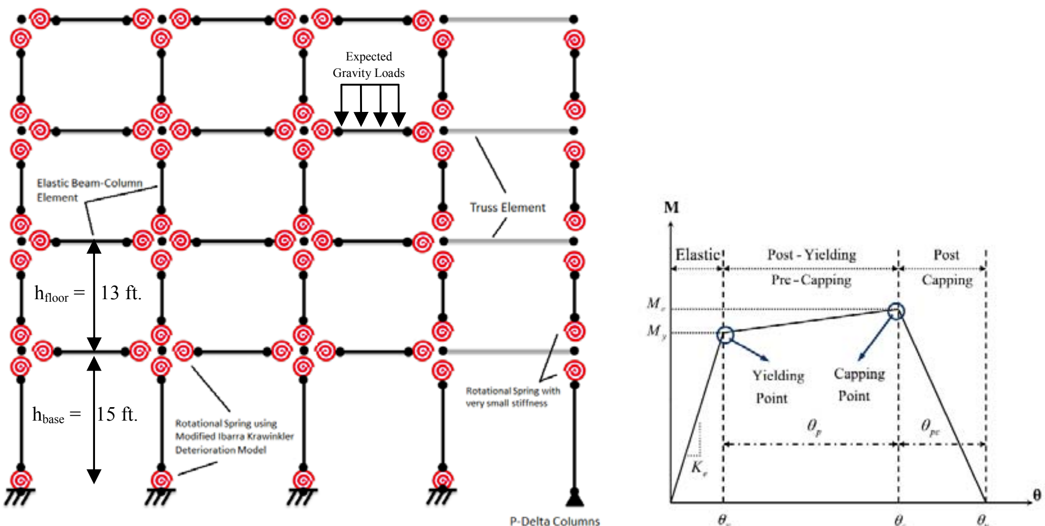
A fundamental goal of building code seismic designs is to guarantee the building occupants' life and safety under severe earthquakes. Even though having straightforward design methodologies, obtaining desired level of economical safety provided by current building codes is vague and could not be satisfactory unless utilizing advanced analytical methods. Advancements in nonlinear dynamic analyses and seismic performance-based earthquake engineering concepts have enhanced building seismic safety traditional assessments by quantifying the building collapse risks and potential losses during large earthquakes.

California like many other vulnerable areas around the world, has failed to identify and retrofit thousands of older buildings despite years of warnings from scientists and engineers that the structures are highly susceptible to collapse during a major earthquake. Experts estimate that between 25,000 to 30,000 concrete buildings were erected before building codes were strengthened in the mid-1970s, including some heavy clusters in downtown Los Angeles and along Wilshire and Hollywood boulevards. The devastating earthquakes in Japan and New Zealand have focused more attention on the vulnerability of such buildings (Lin and Allen 2011).

Once vulnerable buildings are identified, officials face a difficult question: Should people who work or live in them be told about the risks? This study assesses the seismic performance of modern RC special moment-resisting frame (SMRF) office buildings designed according to Performance-Based Plastic Design (PBPD) method and controlled according to current American standards, including ASCE 7-05 and ACI 318-08. The results presented in this paper are part of a larger study by authors to develop a seismic safety rating mechanism that utilizes the existing evaluation methodologies and translate the metrics from different aspects into an understandable format that will be easily realized by building owners and the general public.

1. REPRESENTATIVE SET OF BUILDINGS

In order to predict the collapse capacity and seismic performance of regular structures, simple mathematical models of structural systems denoted as ‘‘generic structures’’ are utilized in this study. A two-dimensional four-story three-bay nonlinear numerical frame model is created for each generic RC SMF by using the OpenSEES structural analysis platform (OpenSEES 2009), as illustrated in Fig. 1a. It is assumed that flexural nonlinear behavior is concentrated at the ends of beams and columns based on modified Ibarra-Krawinkler deterioration model (Ibarra et al. 2005, Lignos and Krawinkler 2009, 2010) (Fig. 1b), and also none of structural components are shear critical, i.e., shear failure is not modeled in generic structures. Rayleigh damping corresponding to 5% of critical damping in the first and third modes is applied. Destabilizing P-Δ effects due to gravity loads are accounted for by applying gravity loads on a leaning column in the analysis model.

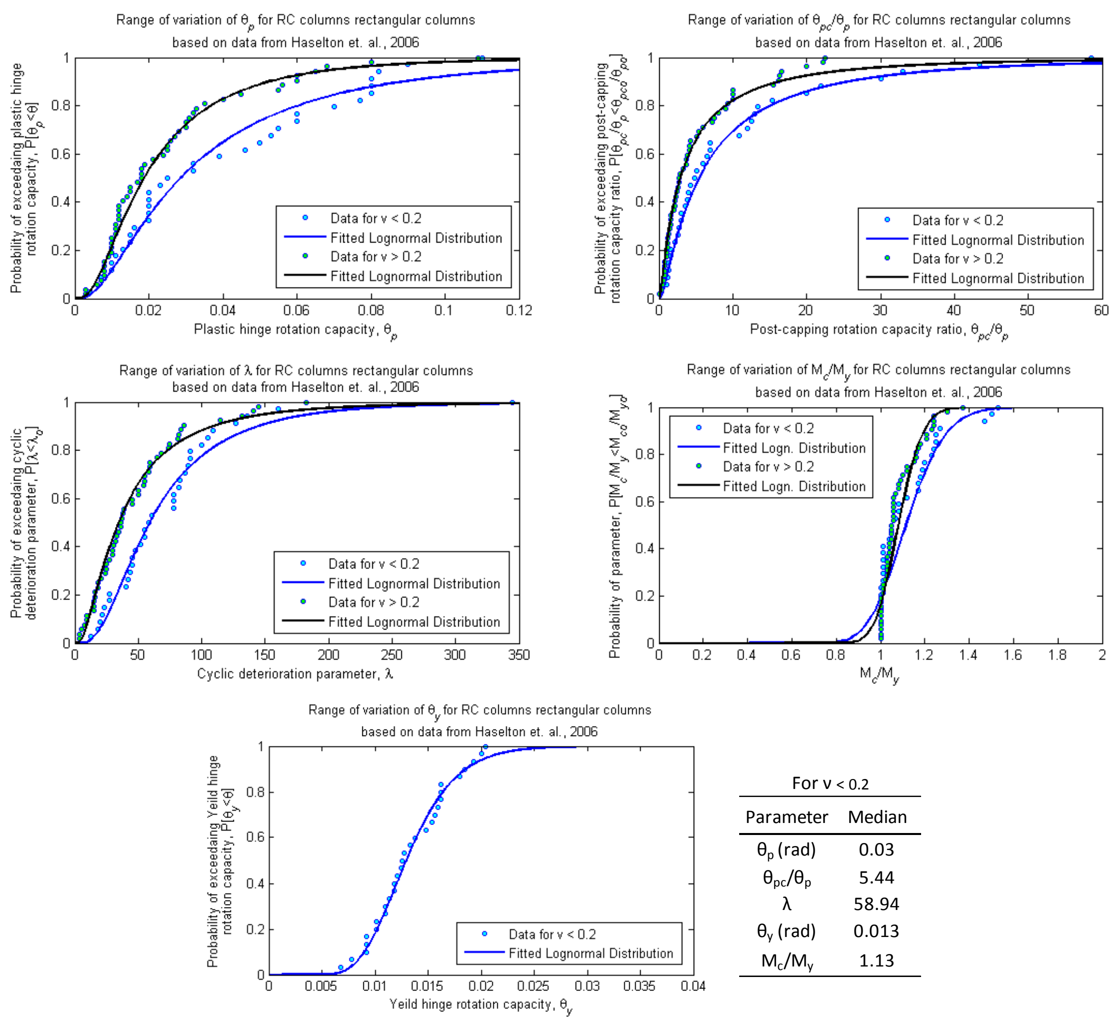


(a) (b)

Figure 1. Schematic concentrated plasticity OpenSees model of 4-story RC-SMRF structural analysis model (a), Backbone curve definition based on typical modified Ibarra-Krawinkler deterioration model where K_e is the initial stiffness, M_y is the yield moment, M_c/M_y is the capping moment ratio, θ_p is the plastic hinge rotation capacity, and θ_{pc}/θ_p is the post-capping rotation capacity ratio (b)

As it can be seen in Fig. 1(a), the generic moment-resisting frames are 3-bay 4-story frames with typical story heights h_{floor} = 13 ft, h_{base} = 15 ft at the base and each bay width equal to 30 ft. Three different values for the first mode period, T_1, are defined for the generic frames as a function of number of stories. These values are 0.10N, 0.15N, and 0.20N, representing different lateral stiffness values of a moment frame with a given number of stories, N. In generic frames, the floor mass is the same at all floor levels, and the story stiffness varies along the height of each generic frame such that a straight line deflected shape is obtained when the ASCE 7-05 lateral load pattern is applied to the frames. It is assumed that stiffness and strength of all structural elements are proportional, and the variation of beam and column strength along the height of generic frame is identical to the variation of stiffness, which has been tuned to the design lateral load pattern (Zareian et al. 2007).

The strong-column weak-beam design philosophy is considered in design of generic structures. Based on ACI 318-08, the ratio of $M_{pbeam-neg}/M_{pbeam-pos}$ in beam designs of a RC-SMRF should not exceed 2.0. Therefore, it is assumed to be 2.1. The design base shear for each frame is determined for two level performance criteria: 1) a 2% maximum Interstory Drift Ratio (IDR), (θ_u) for a ground motion hazard with 10% probability of exceedance in 50 years (DBE); 2) a 3% maximum IDR, (θ_u) for 2% probability of exceedance in 50 years (MCE). The yield drift ratio (θ_y) of 0.5% is used as a lower bound for typical of RC moment frames. The analytical models used for plastic hinge locations in structural components of generic frames include both monotonic and cyclic strength and stiffness deterioration. The backbone curve for stiffness and strength of generic frames is illustrated in Fig. 1b. Cyclic deterioration of strength and stiffness is based on a reference hysteretic energy dissipation capacity, $E_t = \lambda M_y \theta_p$ where λ is a parameter that is estimated using experimental results (Zareian and Krawinkler 2009, Lignos and Krawinkler 2009). For generic frames plastic hinge rotation capacity, θ_p, post capping rotation capacity ratio, θ_{pc}/θ_p , and cyclic deterioration parameter, λ, of beams and columns are set to median values 0.03, 5.44, and 58.94, respectively. The range of deterioration parameters used in this research are depicted in Fig. 2 and is obtained based on reinforced concrete components developed by Berry et al. (2004) and calibrated by Haselton et al. (2006).



For v < 0.2

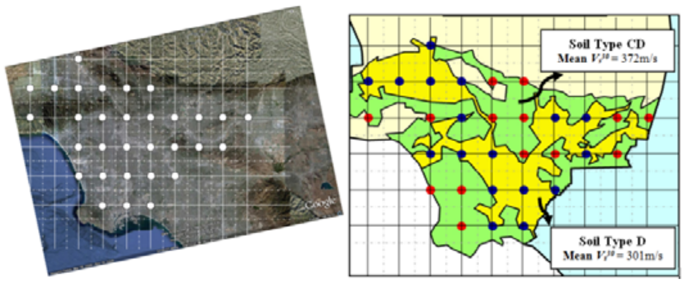
Parameter	Median
θ_p (rad)	0.03
θ_{pc}/θ_p	5.44
λ	58.94
θ_y (rad)	0.013
M_c/M_y	1.13

Figure 2. Range of variation and fitted distributions for component parameters M_c/M_y, θ_p, θ_{pc}/θ_p and λ utilized in modified Ibarra-Krawinkler deterioration model for two axial load intensities ν, based on data from Haselton et. Al. 2006 and selected median values for the three cases

2. SITE, SEISMIC HAZARD, AND GROUND MOTION CHARACTERIZATION

Having a large number of active faults potentially capable of generating earthquakes with magnitude 7 or greater events at Southern California put this area among the most seismically active area in the Western states over the last 20 years (Fialko 2010). The City of Los Angeles is located on the east edge of the Pacific Plate, within the wide transform boundary zone with the North American Plate and near the big bend in the San Andreas fault. The city has experienced and mitigated the effects of earthquakes on the San Andreas and local faults. Owing to the choice of design parameters and seismic hazard level, the buildings are representative of high-seismic regions of Los Angeles. The designs are for a site in downtown Los Angeles. Seismic design is based on the mapped hazard for a Los Angeles site with S_S = 1.5 g and S_1 = 0.9 g and typical soil site class D.

In order to take the variability of site location into account, we assumed the buildings to be located anywhere in the study area, so we performed a grid line over the area and based on 31 representative points of our tentative building locations in Los Angeles (Fig. 3a). The corresponding soil type (C/CD) for each selected point is considered in ground motion selection and modification (Fig. 3b&c). Once we perform seismic hazard disaggregation through utilizing USGS 2008 NSHMP PSHA Interactive Disaggregation (available at: https://geohazards.usgs.gov/deaggint/2008/), recent methodology of ground motion selection and modification proposed by Jayaram et al. (2010) is utilized for each point.



Soil Types	118.600	118.500	118.400	118.300	118.200	118.100	118.000	117.900	117.800	117.700
34.300			D							
34.200	D	D	D	D	CD	CD				
34.100	CD		CD	D	CD	CD	D	D	CD	CD
34.000			D	D	D	CD	CD	D	CD	
33.900			CD	CD	D	D	D			
33.800				CD	D	D				

(a) (b) (c)

Figure 3. Representative site locations (picture is taken from Google Map) (a), Soil type and corresponding V_s^{30} values for the interested locations (b) and latitude/longitude of the selected locations vs. soil type (c).

3. INCREMENTAL DYNAMIC ANALYSIS

In order to determine the collapse capacity of generic moment resisting frames, incremental dynamic time-history analyses (IDA) have been utilized using a set of 44 "Far-Field" pre-defined ground motions in FEMA P695 (FEMA 2009). The database include 22 ground motion record pairs from sites located greater than or equal to 10 km from fault rupture.

Incremental dynamic time-history analyses of generic structures have been performed by OpenSEES structural analysis platform (OpenSEES 2009) that incorporates analytical models that can capture monotonic and cyclic deterioration of structural components. The results obtained from incremental dynamic analyses of one of the 4-story moment-resisting frames subjected to the set of 44 ground motions are illustrated in Fig. 4. Figure 5 shows the collapse fragility curves of all aforementioned 4-story moment-resisting frames.

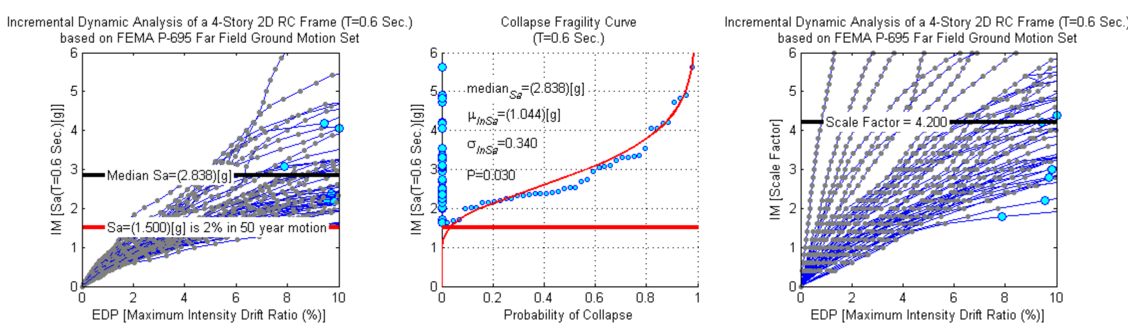


Figure 4. Incremental Dynamic Analysis of a 4-Story 2D RC Frame (T=0.6 Sec.) based on FEMA P-695 Far Field Ground Motion Set

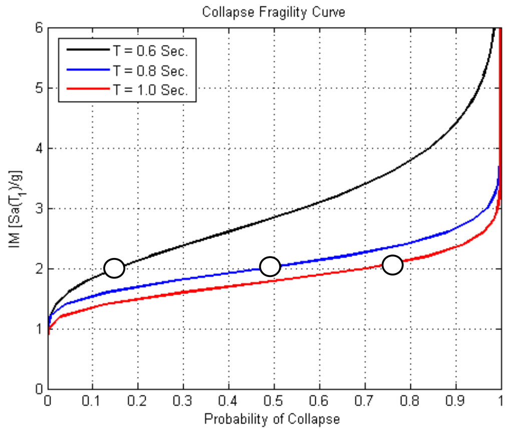


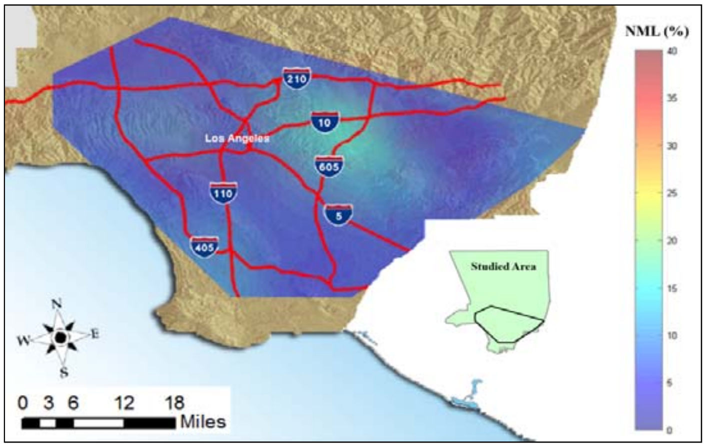
Figure 5. Collapse fragility curves of all 4-Story 2D RC Frames based on IDA of the buildings under FEMA P-695 Far Field Ground Motion Set

It can be inferred from Fig. 5 that the stiffer the building, the less probability of collapse it has under the same level of IM compared to flexible buildings which rationally have higher first mode of vibration.

4. SEISMIC RISK ASSESSMENT

Seismic risk enters into several important real-estate decision-making processes: purchase of investment property, performance-based design of new structures, seismic rehabilitation of existing buildings, and decisions regarding the purchase of earthquake insurance, for example (Porter et al. 2004). The real-estate market is not wholly without forces to influence seismic-risk mitigation. The due-diligence study typically includes an engineering assessment of the condition of the property, which itself typically includes an estimate of the earthquake probable maximum loss (PML) and median loss (ML). PML is by far the dominant earthquake risk parameter in financial decisions. PML is the 90^{th} percentile of loss given the occurrence of what building codes until recently called the design basis earthquake, or DBE - an event producing a shaking intensity with 10% exceedance probability in 50 years and ML is the 50^{th} percentile respectively. We can assume PML an upper-bound loss given the 500-year earthquake. Commercial lenders often use PML to help decide whether to underwrite a mortgage. It is common, for example, for a commercial lender to refuse to underwrite a mortgage if the PML exceeds 20% to 30% of the replacement cost of the building (normalized PML greater than 20% to 30%), unless the buyer purchases earthquake insurance - a costly requirement that often causes the investor to decide against bidding (Porter et al. 2004).

After performing nonlinear time history analyses of the generic frames under the selected ground motions for two hazard levels, DBE and MCE, by taking the variability of site location, we utilized a simulation-based framework for assessing PML of our low-rise reinforced concrete frame buildings. We also investigated the probability of collapse (PC) for each building throughout the region. The generated seismic performance metric (PML and PC) maps can inform the stakeholders for emergency planning and for earthquake hazard mitigation. For this purpose, we applied the Pacific Earthquake Engineering Research Center (PEER) loss estimation framework (Miranda and Aslani 2002) by means of Performance Assessment Calculation Tool (PACT) (ATC 2011) for the RC-SMRF for Engineering Demand Parameters (EDPs) from nonlinear time history analyses regarding each selected station. The Monte Carlo simulation was utilized in PACT to generate realizations of each random variable, which were inputted into a simulation model. The median and mean of Collapse Capacity in terms of $Sa(T_1)$ and the associated aleatory dispersion for each building estimated from IDA. The generic buildings were designed assuming office occupancy. The regular office building components are used to quantify the non-structural components of the building - the exterior closure, interior finishes, and selected mechanical, electrical, and plumbing features that would most likely account for most of the repair cost. Loss analysis results from other PBEE studies (e.g., Porter et al. 2002) have suggested that the building components for this facility that would contribute the most to repair cost are its structural members, drywall partitions and interior paint. Thus, we tried to focus on these specific building components in our loss estimation. The normalized ML and PML plots over the selected area of Los Angeles County are shown in Fig. 6&7.



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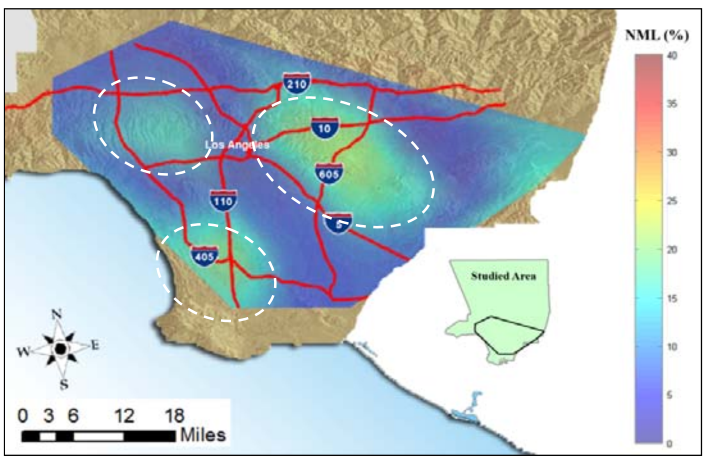

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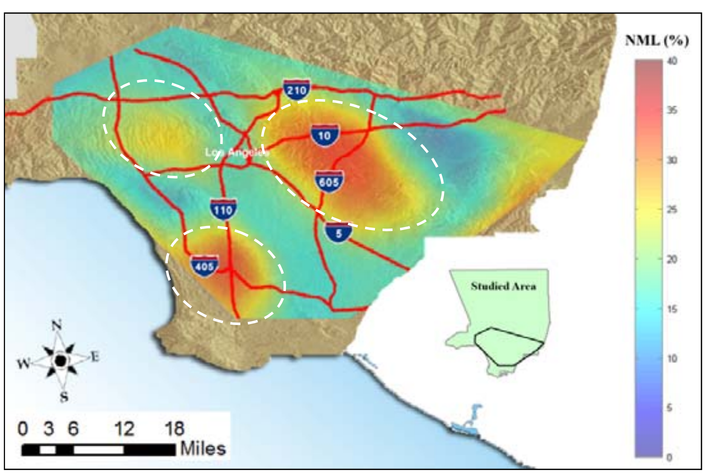
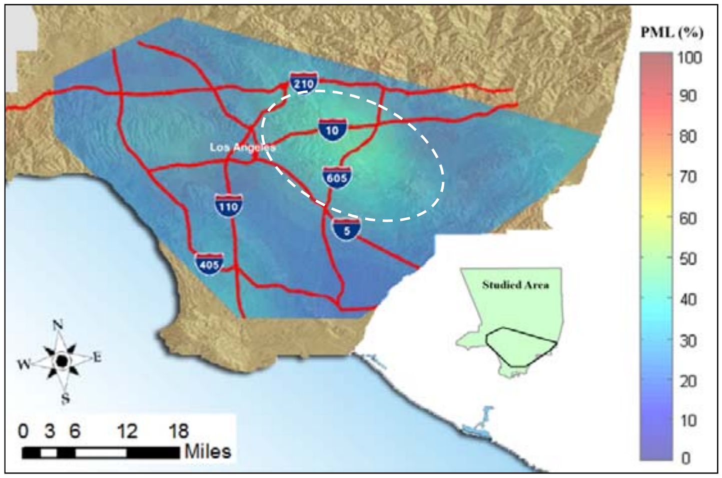


(c)

Figure 6. Normalized ML plot for generic RC-SMRF with (T_1=0.6 Sec.) (a), with (T_1=0.8 Sec.) (b) and with (T_1=1.0 Sec.) (c) over the selected area of Los Angeles County



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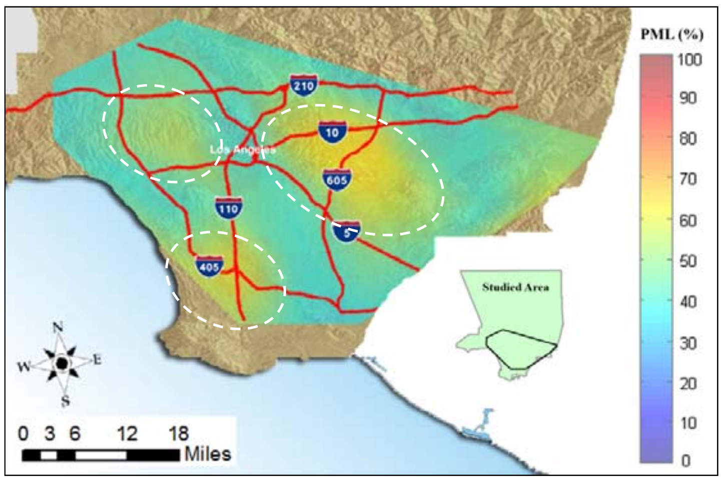

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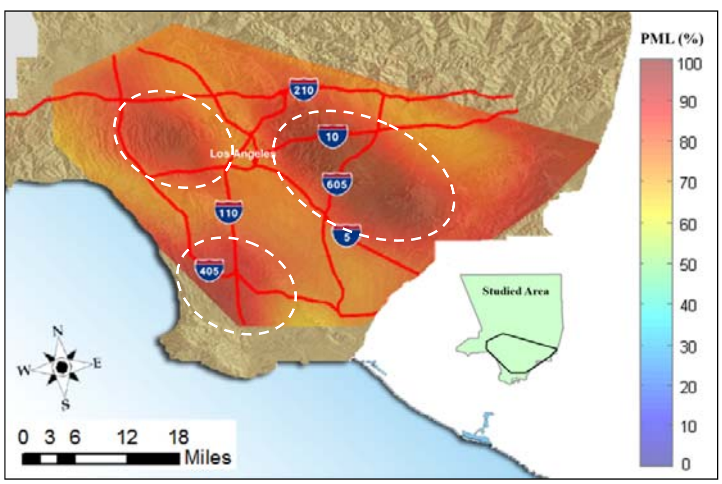
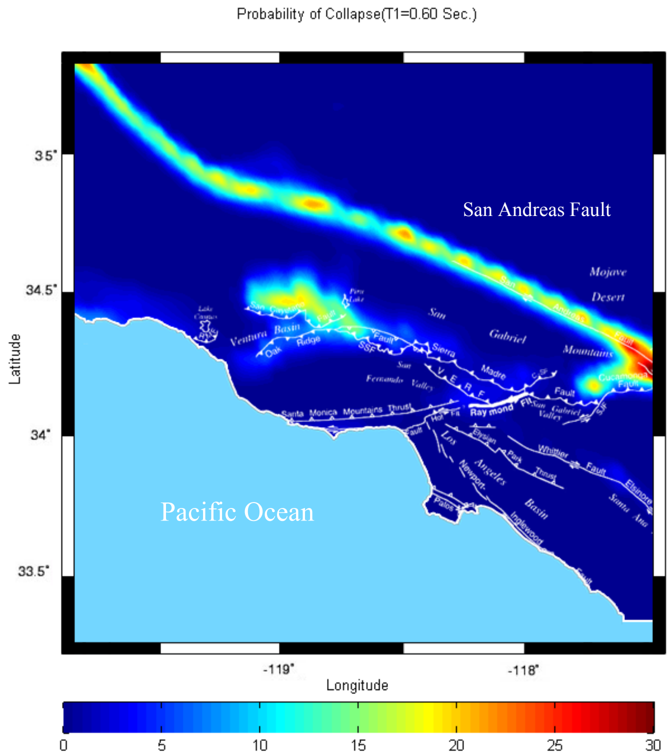
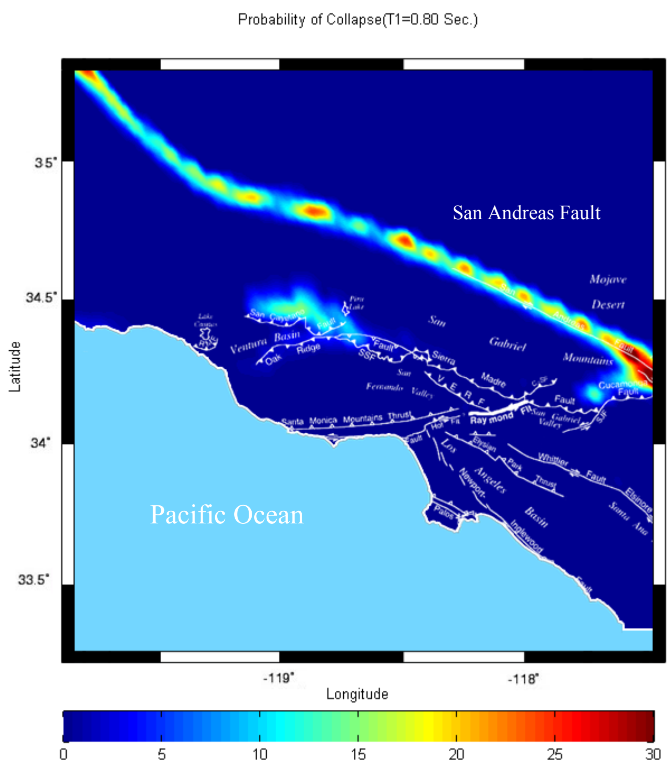


(c)

Figure 7. Normalized PML plot for generic RC-SMRF with (T_1=0.6 Sec.) (a), with (T_1=0.8 Sec.) (b) and with (T_1=1.0 Sec.) (c) over the selected area of Los Angeles County

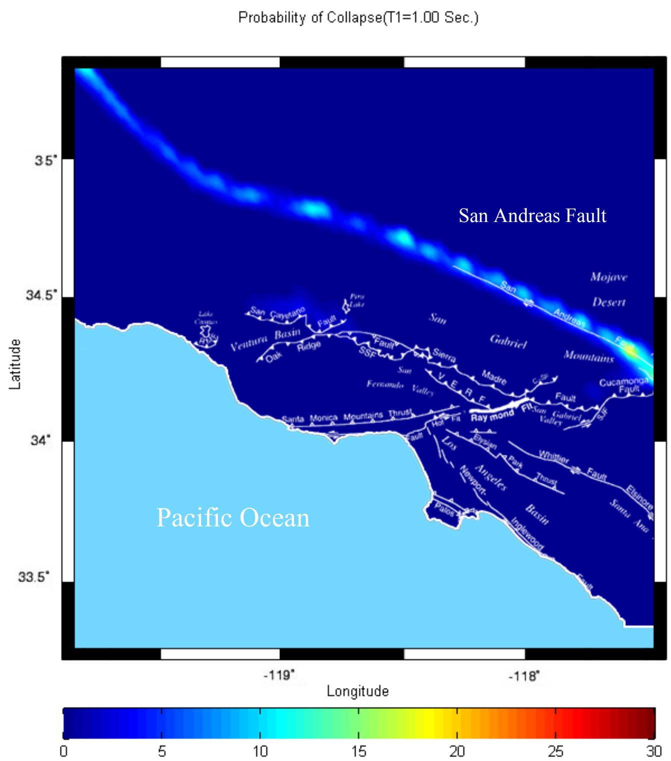


(a)



(b)

Figure 8 - continued – Probability of collapse of generic RC-SMRF with (T_1=0.6 Sec.) (a), with (T_1=0.8 Sec.) (b) and with (T_1=1.0 Sec.) (c) over the southern California compared with most active fault lines in the area



(c)

Figure 8 - continued – Probability of collapse of generic RC-SMRF with (T_1=0.6 Sec.) (a), with (T_1=0.8 Sec.) (b) and with (T_1=1.0 Sec.) (c) over the southern California compared with most active fault lines in the area

5. CONCLUSIONS

Through case studies of several modern reinforced-concrete moment-frame buildings, we have shown that normalized probabilistic maximum repair costs of office buildings which fall inside of the structural range of our study, would be potentially followed by lenders refusal to underwrite a mortgage of a building and costly earthquake insurance would be required after.

In order to have a better building condition, it is inferred from NML and PML maps that having a stiffer office building could effectively control the level of possible loss and consequently satisfy the owner regarding the economical safety of his property, but unfortunately make the building more collapse vulnerable in this area. Fig. 8 shows by having more structural stiffness, we have high probability of collapse in the same area compared to other buildings with more flexibility.

Therefore, finding the optimum structural flexibility of a building which provides us the acceptable level of probability of collapse accompanying a satisfactory level of probable maximum loss would be cost effective. That level of structural flexibility would be considered as initial conceptual design flexibility compared to what we currently have in the design guidelines.

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