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Seismic performance-based design optimization considering direct economic loss and direct social loss

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ABSTRACT

Seismic performance-based design of a steel structure is performed using a multi-objective optimization that considers both direct economic and social losses. Specified performance objectives are considered as constraints and their variance over the obtained Pareto front is investigated. Optimization objectives are selected as the lifetime cost calculated from the initial construction cost and expected annual loss associated with seismic direct economic losses, and direct social loss parameter defined as expected annual social loss. Inelastic time history analysis is used to evaluate structural response under different levels of earthquake hazard to obtain engineering demand parameters. To illustrate the seismic performance-based design procedure, calculations are presented and compared for a sample steel structure located in Los Angeles, CA and Memphis, TN.

1. Introduction

The objective of seismic loss evaluation is to estimate and address the risks associated with having structures located in regions with high seismicity. Earthquake hazard impacts communities in various ways; from economic to social. Considering the impacts of economic and social losses should be an essential component of the structural design and decision making process. This study applies performance-based design (PBD) for structures and implements multi-objective optimization to minimize the potential losses associated with probable earthquake events. Seismic performance-based design (SPBD) is a process of designing new structures or upgrade existing structures to meet specified performance objectives for probable future earthquakes. Performance objectives are defined to quantify the building's behavior in seismic events in terms that would be meaningful and useful to all decision-makers [1]. PBD addresses performances at the system level in terms of risk of collapse, fatalities, repair costs and loss of function [2]. Seismic risk assessment combines hazard analysis with the relationship between intensity measures and seismic loss. Expected annual loss (EAL) is used as the seismic risk measure and is calculated in four major steps: probabilistic seismic hazard analysis, probabilistic seismic demand analysis, probabilistic capacity analysis, and probabilistic loss analysis [3–7]. The results of these four procedures are aggregated using the total probability theorem based on the framework presented by PEER (Pacific Earthquake Engineering Research Center) [6]. The evaluation procedure is a time-based assessment that considers different possible intensities of ground motion that might be experienced by building over a specific period of time [1]. In the PEER framework, losses due to structural performance are quantified by casualties, economic losses and, downtime (temporary loss of functionality) [8,9]. Economic losses as a measure of building performance have been considered in several studies [8,10–12]. In this study, to reflect different aspects of the seismic loss, two types of loss are considered in the calculations: direct economic loss and direct social loss. Direct economic loss expresses the probabilistic economic loss in probable future earthquakes as a percentage of the building replacement cost (%BRC). Direct social loss estimates the probabilistic casualty loss associated with an earthquake event. A multi-objective optimization is implemented to minimize the combination of the present value of the total economic cost (PC_t^T), considering initial cost and seismic economic loss for a lifetime period of structure, and expected annual social loss (EASL). The optimization is applied to the design of an example steel structure that resides in two different geographical regions: Memphis, TN located in Central United States and Los Angeles, CA, located in Western United States.

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Nomenclature

EAL	expected annual loss
EASL	expected annual social loss
TC	penalized value of the PC_t^T
PC_t^T	present value of the total economic cost
SL	penalized value of the EASL
φ	penalty function
CL_{CP}	confidence levels for collapse prevention
CL_{IO}	confidence levels for immediate occupancy
c_i	ith constraint
C_i	scaled ith constraint
DV	decision variable
DM	damage measure
EDP	engineering demand parameter
IM	intensity measure
L_c	direct economic loss for each component
L	direct economic loss
$RC_{DMi,c}$	repair cost for each component c
λ	annual rate of exceedance for each intensity measure
$\Delta\lambda_i$	change in annual rate of exceedance associated with dividing the hazard curve into m different segments
$SL_{outdoor}$	social loss associated with outdoor injuries
m	number of hazard levels considered
SL_{indoor}	social loss associated with indoor injuries
CSL_j	casualty severity level j
α	comprehensive cost for CSL_1($/person)
N_o	number of occupants in building
t	lifetime period
C^I	initial cost
PL_t^S	present value of the seismic direct economic loss
EN_{OI}	expected number of occupants injured or killed in an event
W	weight of the frame
ρ	cost per unit weight of the frame
i_r	discount rate
BRC	building replacement cost
λ_{CL}	confidence parameter
γ	demand variability factor
γ_a	analysis uncertainty factor
D	calculated demand on a structure
C	median estimate of the capacity of the structure
ϕ	uncertainty in the prediction of structural capacity
K_x	standard gaussian variant
β_{UT}	uncertainty measure
CL	confidence level

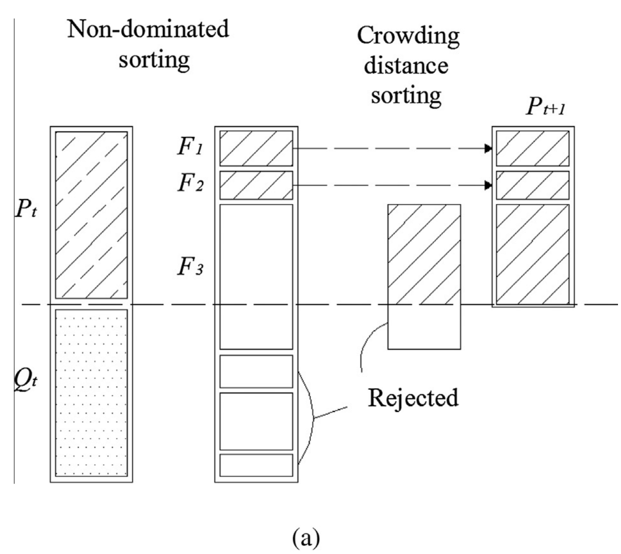
2. Optimization problem definition

A multi-objective genetic algorithm using an elitist non-dominated sorting strategy [13] is implemented to perform the optimization. In order to preserve the diversity of the solutions in the Pareto front, a crowding distance methodology is used. The steps of the implemented optimization method are:

- *Step 1:* Randomly generate a population P_n (size N).
- *Step 2:* Compute a fitness value for each parent individual in the population based on a non-dominated sorting. Fitness is assigned to individuals based on the number of solutions they dominate. An individual dominates another solution when it excels that solution in both objectives.
- *Step 3:* Generate a new child population Q_n (size N) based on general GA methodology (roulette wheel selection, uniform crossover, and mutation).
- *Step 4:* Develop a new population P_{n+1} from the parent and child populations (size 2N) by grouping individuals into subsets of different fronts F_i based on the non-dominated sorting procedure. The next generation (size N) is populated with members for the first front F_1 (the most dominate front). If the new generation is not fully populated from the F_1 front pool, members are taken from the second front F_2, and so on, until the new generation P_{n+1} is fully populated. If there are fewer unfilled positions in the new generation than there are members in a front group, a crowding distance sorting strategy is applied where individuals with larger crowding distances (the distance between the individuals immediately before and after the individual j located on the Pareto front, as shown in Fig. 1b) are chosen to fill out the parent population.
- *Step 5:* If the maximum number of generations has not been met, repeat steps 2–4.

Fig. 1 is the graphical explanation of non-dominated sorting genetic algorithm (NSGA-II) implemented.

Optimization objectives are defined as the lifetime cost of the structure or the present value of the total economic cost PC_t^T and direct social loss $EASL$. Therefore, the optimization problem would be



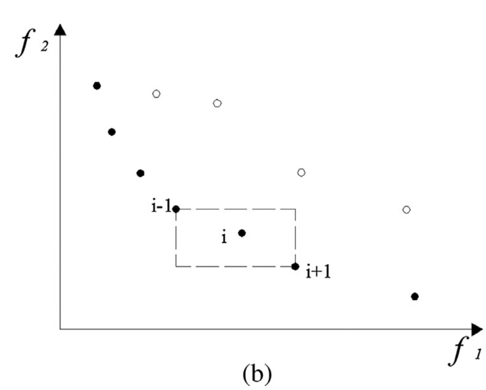


Fig. 1. Multi-objective optimization algorithm (a) NSGA-II procedure, and (b) crowding distance calculation [13].

$$\begin{aligned} &\textit{Minimize}\ (TC, SL) \\ &\textit{Subjected to}\ :\ c_i \geqslant l_i\ (i = 1, 3) \end{aligned} \tag{1}$$

where TC and SL are the penalized values of PC_t^T and $EASL$, respectively; and c_i is the ith constraint that is applied on the optimization problem. The penalized values TC and SL are calculated as

$$TC = \varphi \times PC_t^T \tag{2}$$

$$SL = \varphi \times EASL \tag{3}$$

where φ is a penalty function. The constraints for the confidence levels for collapse prevention CL_{CP} and immediate occupancy CL_{IO} are

$$c_1 :\ CL_{CP} \geqslant CL_{CP,\min} \tag{4}$$

$$c_2 :\ CL_{IO} \geqslant CL_{IO,\min} \tag{5}$$

where $CL_{CP,\min}$ = 90% and $CL_{IO,\min}$ = 50%, as recommended by FEMA 350 [14]. The constraint for ensuring the AISC strong column-weak beam (SCWB) criteria of for seismic design, calculated for each connection in the frame is

$$c_3 :\ \frac{\sum M^*_{pc}}{\sum M^*_{pb}} \geqslant 1.0 \tag{6}$$

where M^*_{pc} is the modified flexural strength of the column and M^*_{pb} is the modified flexural strength of beam sections (neglecting the additional moment due to shear amplification from the location of the plastic hinge to the column centerline). Eq. (6) is calculated using the AISC Specifications Section E3 [15]. The penalty function φ is defined as

$$\varphi = \prod_{i=1}^{3} \varphi_i$$

$$\textit{where} \begin{cases} \varphi_i = \frac{l_i - c_i}{c_i} & \textit{if} \quad c_i < l_i \\ \varphi_i = 1 & \textit{if} \quad c_i \geqslant l_i \end{cases} \tag{7}$$

3. Calculation of EAL and EASL

Loss calculation procedure includes calculation of two loss parameters: expected annual loss (EAL) associated with direct economic loss and expected annual social loss (EASL) associated with direct social loss. Both of these parameters are calculated by aggregating probabilistic seismic hazard analysis (PSHA); probabilistic seismic demand analysis; probabilistic capacity analysis; and probabilistic loss analysis, using the total probability theorem.

The loss assessment framework developed by PEER center [6,16] calculates the mean annual occurrence rate of decision variable $\lambda[DV]$ as

$$\lambda[DV] = \int\int\int P[DV|DM]P[DM|EDP]P[EDP|IM]\lambda[IM]\ dDM\ dEDP\ dIM \tag{8}$$

where DV is the decision variable, DM is the damage measure, EDP is the engineering demand parameter, and IM is the intensity measure. The cumulative distribution function of the random variable X conditioned on random variable Y is $P[X|Y]$. The decision variable DV is direct economic loss for the EAL calculation and direct social loss for $EASL$ calculation.

PSHA, the first step in calculating the loss parameters, quantifies the uncertainties regarding the location, size, and resulting shaking intensity of possible future earthquakes at a given site [17]. PSHA is performed for Memphis using EZ-FRISK [18]. Synthetic ground motions are generated using the stochastic methods implemented in SMSIM [19]. A stochastic method for synthetic ground motion generation is used to address the need for ground motion records compatible with local seismic characteristics in regions with scarce recorded data. The ground motions are modified to match the uniform hazard response spectra for four hazard levels of earthquakes with 2, 5, 10, and 50 percent probability of exceedance (POE) in 50 years [20]. The SHAKE91 computer program is used to account for site effects using the Memphis site properties given by EPRI and Romero and Rix [21,22]. Total of 40 ground motions (10 time histories for each hazard level) are considered for calculation of losses in Memphis site. For the Los Angeles site, suites of ground motions for three different hazard levels 2, 10, and 50 POE in 50 years (total of 30 ground motions) are taken from the SAC steel research project [23,24].

In the seismic demand analysis, the response of the structure subjected to the ground motions defined by the PSHA is used to calculate engineering demand parameters (EDPs). In this study, the EDPs are the inter-story drifts (ISD) and the peak floor accelerations (PFA) calculated from a non-linear time-history analysis of the structure using the DRAIN-2DX computer program [25]. The *EDP*s are linked to *DM*s that describe the physical condition and damage state of the buildings' components [26].

Probabilistic capacity analysis uses the fragility curve parameters, defined in the Hazus technical manual [27], for structural and non-structural members for different damage states. Fragility functions indicate the probability of damage to an element or system for a specific damage state as a function of a single demand parameter (e.g. ISD or the PFA) [28].

Probabilistic loss analysis estimates the consequences of structural damage from an earthquake and is used to evaluate decision variables (*DV*s). These variables are related to consequences of earthquake damage and can be expressed in terms of social losses or casualties or economic losses associated with repair cost or repair time. The *DV*s used in this study are direct economic and social losses due to earthquake events that are calculated considering different *DMs* (slight, moderate, extensive, and complete).

Direct economic loss is expressed in terms of the percentage of the building replacement cost (%BRC). Expected direct economic losses $E[L_{c,EDP}]$ (%BRC) for each component c, structural (SS), drift-sensitive non-structural (NSD), acceleration-sensitive non-structural (NSA), are calculated for a specific IM as

$$E[L_{c,EDP}] = \sum_{i=2}^{5} P[DM_{i,EDP}] \times RC_{DM_i,c} \tag{9}$$

where $L_{c,EDP}$ is the loss associated with each component (SS, NSD, and NSA), $P[DM_{i,EDP}]$ is calculated using fragility curves, and $RC_{DM_i,c}$ is defined as the repair cost for each component due to DM_i which varies from slight (i = 2) to complete (i = 5) [29]. Expected loss $E[L_{EDP}]$ for a particular structure and specific IM is calculated as the sum of losses for all components as:

$$E[L_{EDP}] = E[L_{SS,EDP}] + E[L_{NSD,EDP}] + E[L_{NSA,EDP}] \tag{10}$$

The total loss curve is obtained from the loss curves for each hazard level and hazard curve as

$$P[L > l] = \int_{\lambda} P[L > l|IM]\ d\lambda \approx \sum_{i=1}^{m} (1 - P[L < l|IM_i])\ \Delta\lambda_i \tag{11}$$

where $P[L > l]$ is probability of loss L exceeding a specific value l, which is obtained from the loss curves for each hazard level, λ is the annual rate of exceedance for each IM_i, m is the number of hazard levels considered, and $\Delta\lambda_i$ is the change in annual rate of exceedance associated with dividing the hazard curve into m different segments. The EAL is the area under the total loss curve.

EASL is calculated following the same procedure presented for EAL with direct social loss as the decision variable. The methodology presented in Hazus-MH [29] is used to perform probabilistic loss analysis with casualties as DV. This methodology assumes that there is a relationship between building damage and the number and severity of casualties and estimates casualties caused by both structural and nonstructural damage [29].

Social losses for indoors and outdoors injuries, $E[SL_{indoors,EDP}]$ and $E[SL_{outdoors,EDP}]$, respectively for a specific *IM* are calculated as

$$E[SL_{indoors,EDP}] = \sum_{i=2}^{4}\sum_{j=1}^{4} P[DM_{i,EDP}] \times P[CSL_j|DM_i] \times w_j \times \alpha + P[DM_{5,EDP}] \times \left[\sum_{j=1}^{4}(P[Collapse|DM_5] \times P[CSL_j|Collapse] + P[no-Collapse|DM_5] \times P[CSL_j|no-Collapse]) \times w_j \times \alpha\right] \quad (12)$$

$$E[SL_{outdoors,EDP}] = \sum_{i=2}^{5}\sum_{j=1}^{4} P[DM_{i,EDP}] \times P[CSL_j|DM_i] \times w_j \times \alpha \quad (13)$$

where DM_i is the damage measure for slight ($i = 2$) to complete ($i = 5$), CSL_j is the casualty severity level for j equal 1 (lowest severity level) to 4 (highest severity level), and w_j are the weights given to different *CSLs* based on financial costs. The probabilities for different *CSLs* are based on recommendations presented in Hazus-MH [29]. The weights w_j are chosen based on the comprehensive costs for different injury levels suggested by National Safety Council (NSC) [30] and α ($/person) is the comprehensive cost for CSL_1. Fig. 2 shows an overview of the Hazus methodology, in which casualties caused by an earthquake are modeled by developing a tree of events leading to their occurrence [29].

The expected number of occupants injured or killed in an event, $EN_{OI,EDP}$, for a specific *IM* is calculated as

$$EN_{OI,EDP} = N_o \times [n_i \times E[SL_{indoors,EDP}] + n_o \times E[SL_{outdoors,EDP}]] \quad (14)$$

where N_o is the number of occupants in building, n_i and n_o are factors that account for the distribution of people indoor and outdoor, considered recommendations from Hazus MH [29]. The expected number of occupants injured or killed is calculated for all ground motions and all hazard levels. EASL is calculated using the area under the total social loss curve obtained from the aggregation of the loss curves for each hazard level and hazard curve. Fig. 3 shows total loss curves for economic and social losses for Memphis, TN and Los Angeles, CA. Areas under the economic and social total loss curves are calculated as EAL and EASL, respectively.

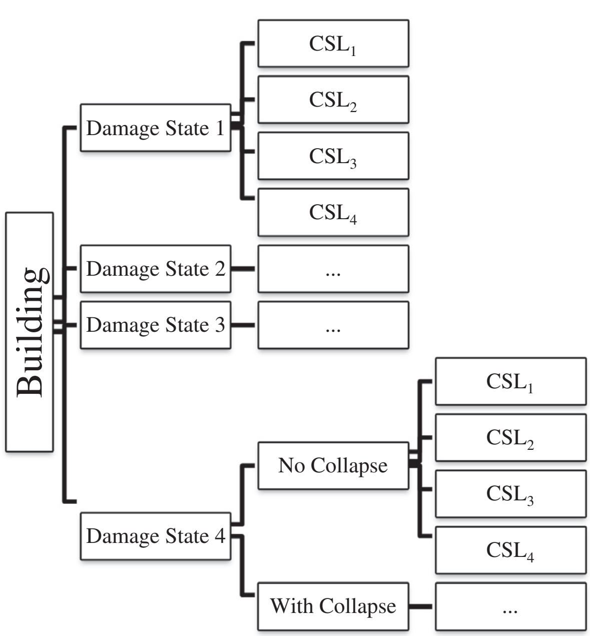


Fig. 2. Injury event tree model [28].

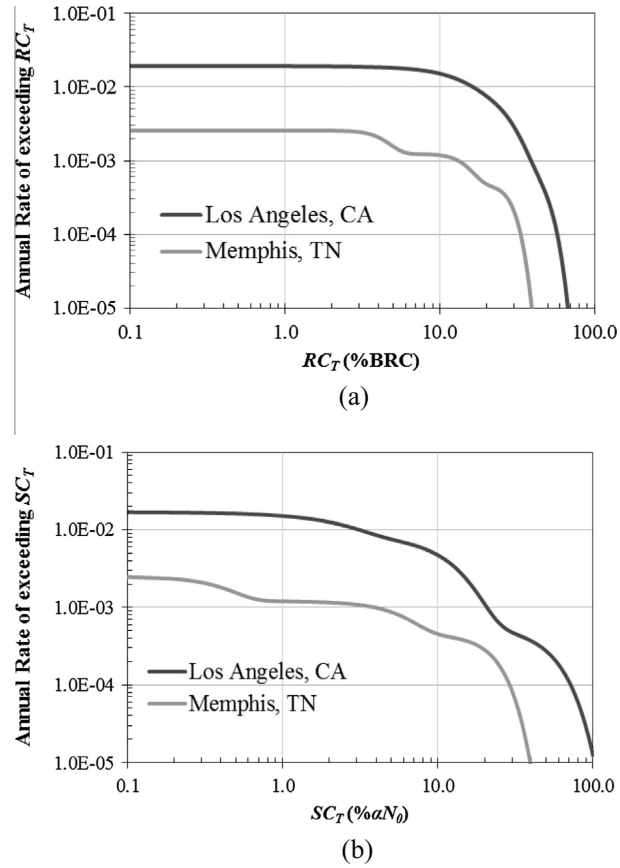


Fig. 3. Comparison of total loss curves for Memphis, TN and Los Angeles, CA sites for an example structure: (a) total economic loss curves, and (b) total social loss curves.

The present value of the total economic cost PC_t^T considering initial cost and seismic economic loss for a lifetime period of *t* years, is estimated as

$$PC_t^T = C^I + PL_t^S \quad (15)$$

where C^I is the initial cost of the structure and PL_t^S is the present value of the seismic direct economic loss. The initial cost C^I is

$$C^I = \rho \times W \quad (16)$$

where *W* is the weight of the frame and ρ is the cost per unit weight of the frame. Present value of the seismic economic loss PL_t^S is calculated as

$$PL_t^S = EAL \times \frac{(1 - e^{-i_r t})}{i_r} \quad (17)$$

where i_r is discount rate [31] assumed to be 2% and *t* is 50 years. The value of EAL in Eq. (17) is calculated by considering the BRC to be equal to C^I.

4. Calculation of confidence levels

The calculation of confidence levels (CLs) in meeting the performance objectives are based on a procedure presented in [14]. Performance objectives are collapse prevention for hazard level of 2% POE in 50 years and immediate occupancy for hazard level of 50% POE in 50 years. Structural demands for the earthquake

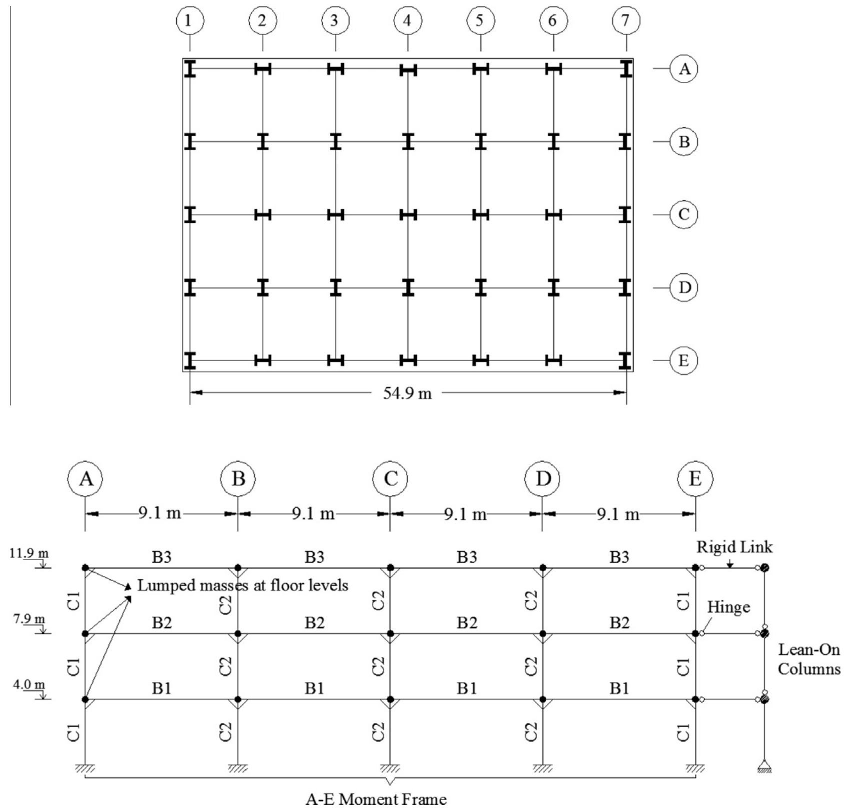


Fig. 4. Considered structure and the steel moment frame model.

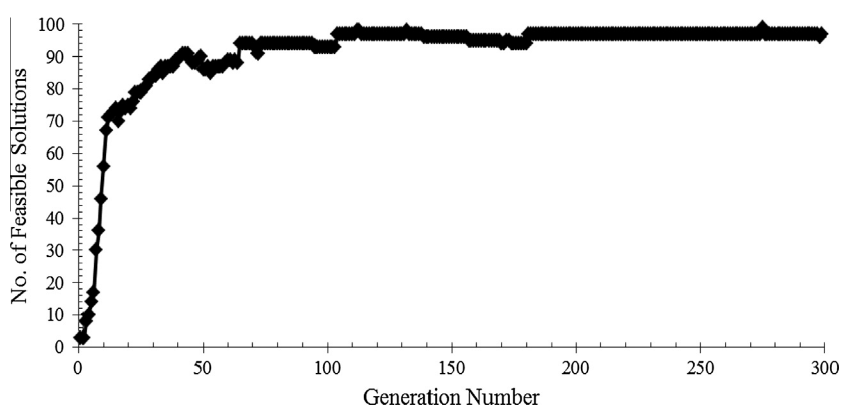


Fig. 5. Typical convergence history for example frame.

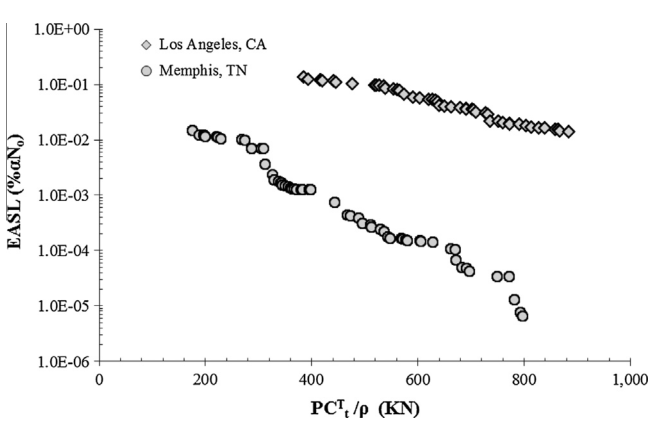


Fig. 6. Pareto fronts for site locations in Memphis, TN and Los Angeles, CA.

ground motions associated with selected hazard levels are the median values of maximum ISD and maximum column compressive forces of structure for suites of ground motions in each hazard level and are determined using non-linear time-history analysis. The confidence parameter λ_{CL} is calculated as the factored demand-to-capacity ratio as

$$\lambda_{CL} = \frac{\gamma \gamma_a D}{\varphi C} \tag{18}$$

where γ is the demand variability factor, γ_a is an analysis uncertainty factor, D is the calculated demand on a structure, C is the median estimate of the capacity of the structure, and ϕ accounts for the uncertainty in the prediction of structural capacity. Values for these parameters are the recommended values by FEMA [14]. Confidence level CL is calculated as

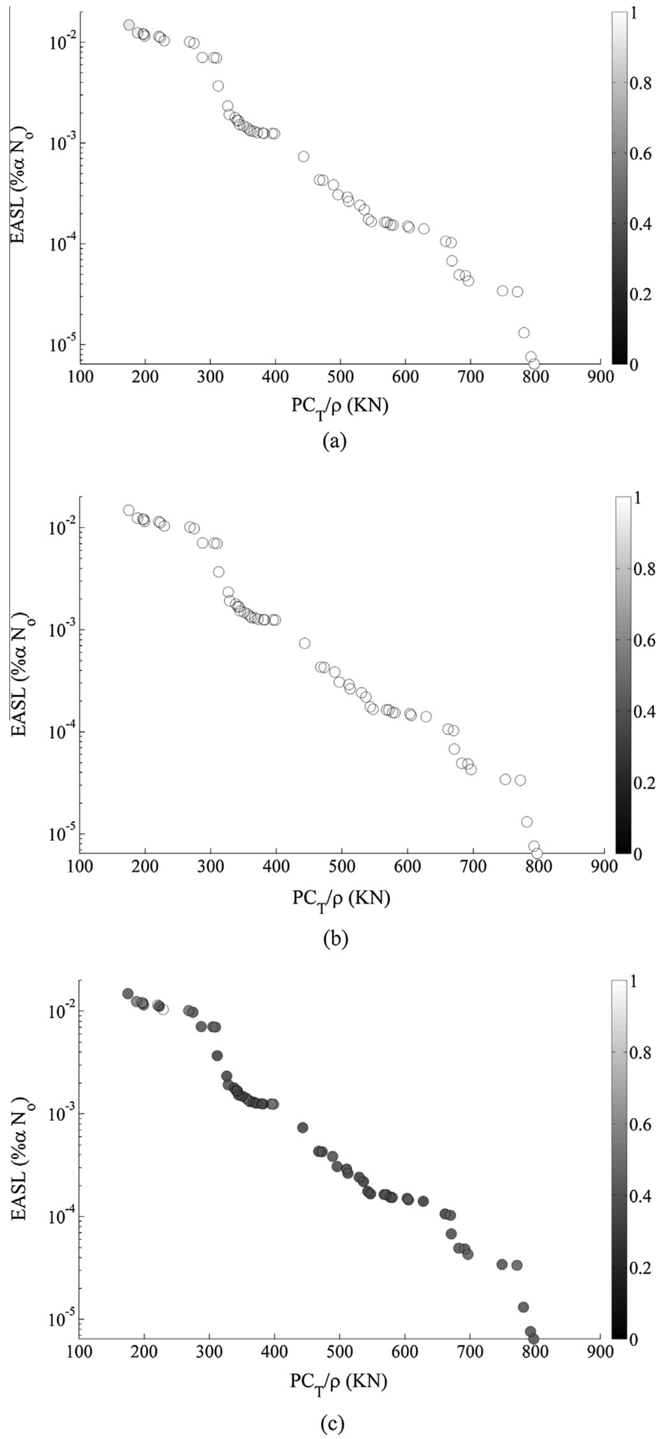


Fig. 7. Comparison of variation in different criteria along the Pareto front for Memphis, TN site: (a) C1, (b) C2, and (c) C3.

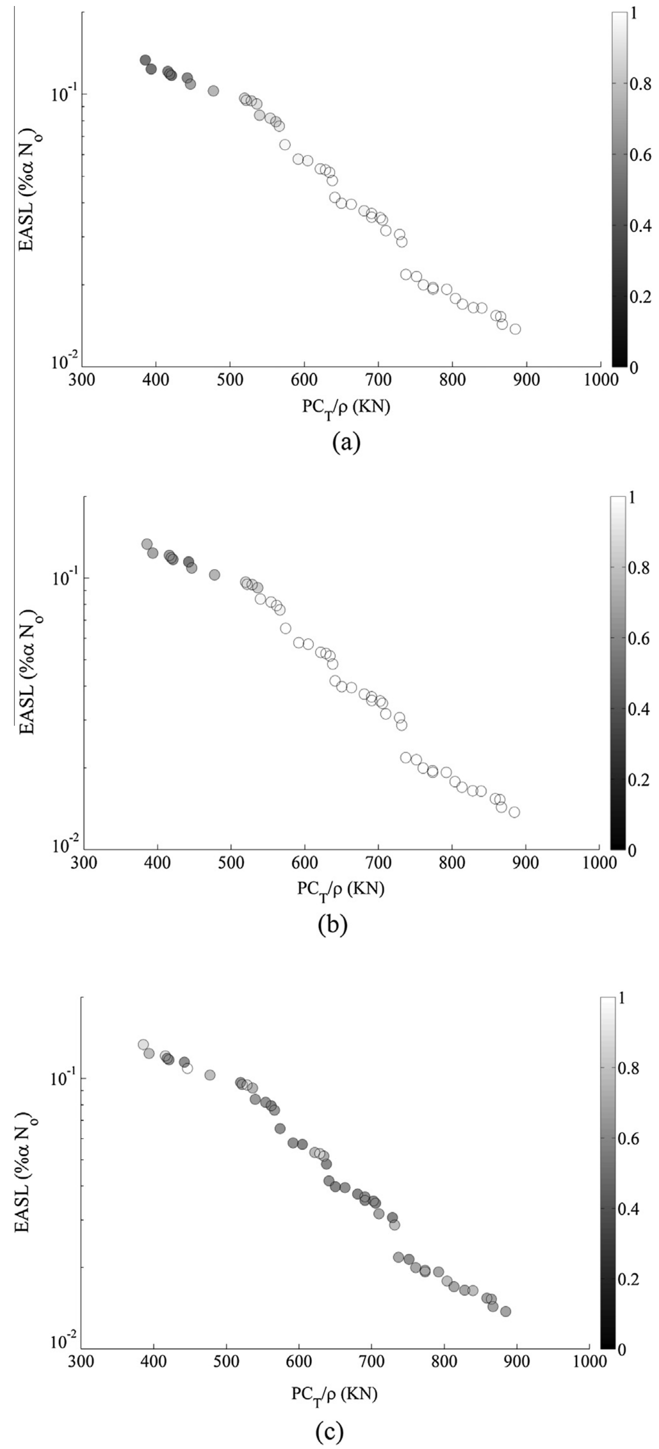


Fig. 8. Comparison of variation in different criteria along the Pareto front for Los Angeles, CA site: (a) C1, (b) C2, and (c) C3.

$$CL = \Phi(K_x) \tag{19}$$

where $\Phi(K_x)$ is the normal cumulative distribution function value corresponding to K_x which is a standard Gaussian variant associated with probability x of not being exceeded [14] and is calculated as

$$K_x = \frac{k\beta_{UT}}{2} - \frac{\ln(\lambda_{CL})}{b\beta_{UT}} \tag{20}$$

where β_{UT} is an uncertainty measure equal to the vector sum of the logarithmic standard deviation of the variations in demand and capacity resulting from uncertainty, b is a coefficient relating the incremental change in demand (ISDs and column forces) to an incremental change in ground shaking intensity at each hazard level, taken as 1.0 [14], and k is the slope of the hazard curve, in natural log coordinates, at the hazard level of interest [14].

5. Example structural design

The multi-objective SPBD optimization problem is applied to an example SAC structure originally presented by FEMA [24]. Fig. 4

Table 1
Three designs selected from the obtained Pareto fronts for site locations in Memphis, TN and Los Angeles, CA.

Designs	C1	C2	B1	B2	B3	PC_t^T/ρ (KN)	*EASL* (%αN_o)
MEMPHIS, TN							
Min PC_t^T	W14X109	W14X109	W21X50	W21X44	W18X46	175.01	0.01485
Midpoint front	W14X176	W14X233	W27X94	W30X99	W18X46	312.01	0.00369
Min *EASL*	W14X550	W14X605	W40X183	W40X199	W36X160	797.61	0.00001
LOS ANGELES, CA							
Min PC_t^T	W14X257	W14X257	W27X84	W24X76	W24X68	385.69	0.13349
Midpoint front	W12X336	W14X398	W36X150	W30X116	W24X104	573.95	0.06531
Min *EASL*	W14X550	W14X605	W40X199	W40X199	W40X183	884.62	0.01375

Table 2
Calculated loss values for the designs located on Pareto front for site locations in Memphis, TN and Los Angeles, CA.

Designs	W (KN)	EAL (%BRC)	PC_t^T/ρ (KN)	*EASL* (%αN_o)	CL_{CP} (%)	CL_{IO} (%)
MEMPHIS, TN						
Min PC_t^T	173.33	0.0307	175.01	0.0149	98.50	100.00
Midpoint front	310.27	0.0177	312.01	0.0037	100.00	100.00
Min *EASL*	796.69	0.0036	797.61	0.0000	100.00	100.00
LOS ANGELES, CA						
Min PC_t^T	345.73	0.3657	385.69	0.1335	90.95	76.02
Midpoint front	523.12	0.3076	573.95	0.0653	99.55	99.62
Min *EASL*	818.16	0.2571	884.62	0.0138	98.50	100.00

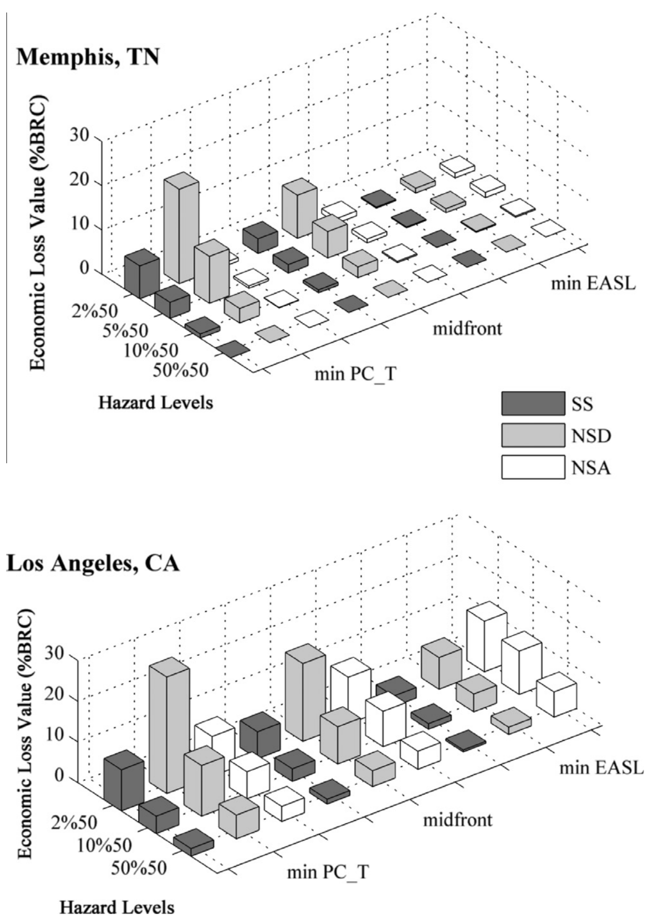


Fig. 9. Distribution of economic losses for different components of the building for structures located in Memphis, TN and Los Angeles, CA.

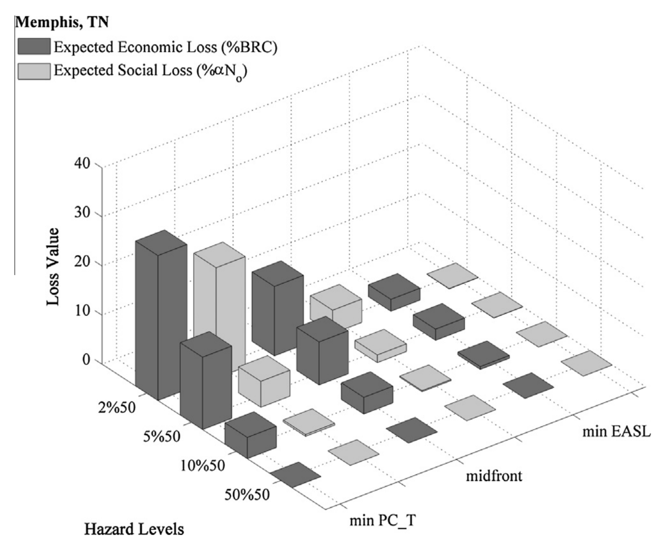


Fig. 10. Comparison between distribution of direct economic losses and direct social losses for structures in Memphis, TN.

shows the SAC structure and the frame modeled in DRAIN-2DX [25]. Based on the loading definitions, the seismic mass for the entire structure is 1034.71 KN s^2/m for roof and 956.34 KN s^2/m for floors [26]. Masses are lumped at the beam-to-column locations. Performance evaluation is performed based on FEMA recommendations [14]. The moment frame A–E is considered in the PBD optimization. The effects of the gravity frames on the second-order analysis of the A–E frame are applied by utilizing lean-on columns, as shown is Fig. 3. The design variables for seismic PBD optimization: two column types C1 and C2 and three beam types B1, B2, and B3 are defined in Fig. 3.

The search space for the considered design variables in the optimization problem includes a list of 60 AISC W sections for columns (non-slender W10, W12, and W14 sections) and another list of 64 AISC W sections (W18, W21, W24, W27, W30, W33, W36, and W40, with nominal weight range of 0.58–2.93 KN/m) for beam elements. For this example, the GA uses a population size of 100, a stopping criterion of 300 generations, a roulette wheel selection method, a uniform crossover operator with a probability of 0.6, mutation probability of 0.03, and binary encoding. Fig. 5 shows a

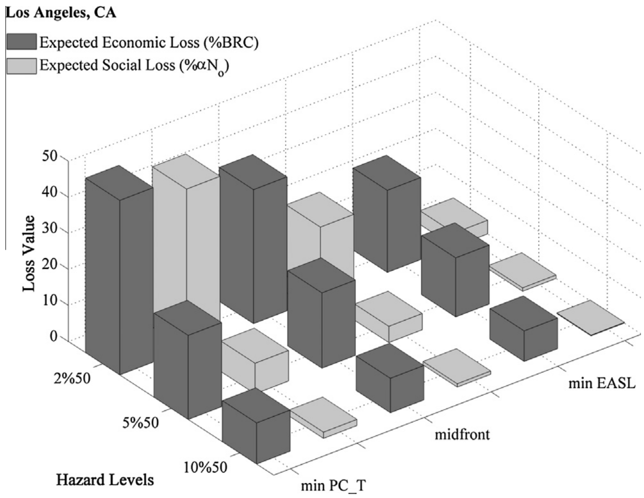


Fig. 11. Comparison between distribution of direct economic losses and direct social losses for structures in Los Angeles, CA.

typical convergence history for the example frame. Fig. 6 shows the results of the multi-objective optimization in the form of Pareto fronts for site located in Memphis, TN and Los Angeles, CA. All presented results are feasible solutions, which mean that the value of the penalty function φ is one for all these solutions. The results are presented as EASL (%α N_o) versus PC_t^T/ρ, where N_o is the number of occupants in building, α (in \$/person) is the comprehensive cost for CSL_1 and ρ is the cost per unit weight of the frame.

Pareto fronts in this multi-objective optimization problem are defined as sets of feasible non-dominated solutions that are similar to optimization results using different weights for each of the two specified objectives. In order to determine which constraint has the most effect on the optimization, the constraints defined in Eqs. (4)–(6) are scaled as follows:

$$C_i = \frac{c_i \times l_i - l_i}{1 - l_i} \quad (for\ i = 1 \text{ and } 2) \tag{21}$$

$$C_3 = \frac{c_3 - l_3}{c_{3\ \max} - l_3} \tag{22}$$

Figs. 7(a–c) and 8(a–c) show scaled constraint values for the example steel structure located in Memphis and in Los Angeles; respectively. In both figures, darker colored circles specify smaller values for the C_i's for designs which is an indicator of associated constraints being closer to the defined limit states. As expected, when designs move along the Pareto front towards higher cost, the values of C_1 and C_2, defined by confidence levels for CP and IO performance objectives, increase. However, no specific pattern is observable for C_3 (SCWB criterion). Comparing the C_i values in Figs. 7 and 8 shows that C_3 is often controlling for both sites, which implies that the SCWB requirement is typically the controlling constraint in the optimization problem.

In order to better compare the losses at the two sites, three designs are selected along the Pareto fronts for each site. Table 1 lists the selected designs and their corresponding values of optimization objectives. Table 2 lists the calculated loss values for these designs for both sites.

Fig. 9 shows the distribution of direct economic losses associated with structural (SS), displacement sensitive non-structural (NSD), and acceleration sensitive non-structural (NSA) components for the Memphis and Los Angeles sites. For both locations, contribution of SS and NSD components decreases for designs associated with minimum PC_t^T to the minimum EASL design, In contrast, of NSA components in the calculated total loss value increase. In most cases, NSD components have the greatest contribution to the total economic loss value.

Figs. 10 and 11 show a comparison between distribution of direct economic losses and direct social losses for the SAC structures at both sites. Both loss parameters have a descending trend from the designs that minimized for PC_t^T to the designs minimized for *EASL*.

6. Summary and conclusions

The objective of this study is to develop an optimal performance based design (PBD) procedure that considers the economic and social losses associated with probable future earthquakes. Designs for a steel moment frame structure are developed using the proposed PBD procedure. The PBD of a structure is accomplished using a multi-objective optimization considering two objectives. Seismic losses, calculated through the integration of four steps of probabilistic seismic hazard analysis, probabilistic demand analysis, probabilistic damage analysis, and probabilistic loss analysis, by implementing total probability theorem, are used to evaluate optimization objectives. The first optimization objective is the present value of the total cost, calculated based on the initial construction cost and expected annual loss (EAL) associated with seismic direct economic losses. The second optimization objective is the direct social loss modeled as expected annual social loss (EASL) which is a parameter developed in this study to facilitate the interpretation of social loss in calculations and to provide a comparison tool between economic and social loss parameter values. The multi-objective optimization results are presented in the form of Pareto fronts which may be used to visualize the trade-offs between the various objectives and assist decision makers in quantifying the importance of their individual objectives (possibly in the form of weights assigned to each objective).

An evaluation of the critical optimization criteria for designs along the Pareto fronts indicates that the strong-column weak-beam constraint often controls the feasibility of designs generated by the optimization.

A comparison of the economic and social expected annual losses shows that these loss values are considerably lower for a site located in Memphis, TN than a site located in Los Angeles, CA. This variance can be explained by the difference in site seismicity characteristics and the less steep slope of the hazard curve for Memphis, TN as compared to Los Angeles, CA and indicates the significance of seismicity characteristics of the region in the evaluation of expected annual seismic loss parameters. Additionally, the ratio of change in PC_t^T to change in EASL between extreme designs along the Pareto front (i.e. min PC_t^T and min *EASL* designs) is several times larger for designs in Los Angeles as compared to Memphis. This higher ratio implies that for the structure located in Los Angeles site, a specific increase in the value of PC_t^T would result in more reduction in the *EASL* value as compared to the structure located in Memphis site.

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