Research Note

Seismic Fragility Assessment of Performance-Based Optimum Designed Steel Moment Frame

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ABSTRACT

Modeling the nonlinear behavior of structural elements is one of the important parameters for assessing the fragility of steel moment frames. Nonlinearity modeling of structural components is done with two methods: a) distributed plasticity, and b) concentrated plasticity. In distributed plasticity method, the element is considered fibrous and non-linear. In contrast, in concentrated plasticity method, the element is assumed to be elastic and the place of hinges formation is considered at its two ends. To investigate the effect of nonlinear modeling of structural elements on the fragility of steel moment frames a 6-story frame, which is optimally designed with RUPSO algorithm, is considered. The results show that the steel moment frame with concentrated nonlinearity is more fragile than the one with distributed nonlinearity.

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1. Introduction

There are many influencing variables in structural design. Achieving the optimal design without using optimization algorithms, especially in structures with numerous design variables, is exhausting or even impossible. The optimization procedure is performed using gradient-based methods or metaheuristic algorithms. Metaheuristic algorithms are far more effective and practical for optimizing systems with discrete and continuous design variables. One of the advantages of metaheuristic algorithms is that they are easy to apply, since the derivability of the objective functions is not required. Numerous metaheuristic algorithms have been introduced throughout the years, among which are the genetic algorithm (GA) (Holland et al., 1975), ant colony optimization (ACO) (Dorigo et al., 1992), particle swarm optimization (PSO) (Eberhart et al., 1995), big bang-big crunch (BB-BC) (Erol et al., 2006), Colliding Bodies Optimization (Kaveh et al., 2014).

Dadashi and Mohammadi (2023) introduced the random update particle swarm optimization (RUPSO) algorithm, which is a metaheuristic method based on the PSO algorithm. In this paper, the RUPSO algorithm has been used for the optimum performance-based design. The purpose of the optimal performance-based design of steel moment resisting frames (SMRFs) is to meet the needs of nonlinear deformation at different hazard levels with the lowest possible construction cost. (SMRFs) elements undergo plastic deformation due to their nonlinear behavior in strong earthquakes, which leads to structural failure and ultimately causes life and economic losses. This means that it is necessary to limit the amount of plastic deformation of structural components in different earthquake intensities. The use of performance-based design and modeling of the nonlinear behavior of structural elements can decrease the risk at various earthquake intensities. In performance-based design, the use of metaheuristic optimization algorithms leads to more economical design. In various studies, the optimal performance-based design process has been carried out using different metaheuristic algorithms with different objective functions and performance constraints.

Gholizadeh and Dadashi (2013) studied the optimal performance-based design of (SMRFs) according to FEMA-356 guidelines. In this research, in addition to the geometric constraints and the serviceability of gravity loads the plastic deformation of hinges in each of the hazard levels has been considered as a performance constraint and the weight of the structure has been minimized as an objective function. They demonstrated that the particle swarm optimization (PSO) algorithm performed better than the ant colony optimization (ACO), harmony search (HS) and genetic algorithms (GA).

Fragiadakis et al. (2006) performed the optimal performance-based design of a 10-story SMRF by using a genetic algorithm. They considered the plastic deformation of hinges as a constraint-based on FEMA-356 guidelines. The objective function was to minimize the cost of construction and probable seismic damage during the lifetime of the structure. The performance-based design is a probabilistic process and the effect of uncertainties is not considered in this method, thus the response of the structure will differ under different conditions. The sources of uncertainty can be investigated in two categories: (a) aleatory, and (b) epistemic uncertainties. The probability of exceedance of a certain limit state in several levels of earthquakes and under the effect of uncertainties can be obtained by using fragility curves. In this article, to determine the fragility curves, incremental dynamic analysis (IDA) has been used to evaluate the probability of exceedance of the specified limit states.

Hosseinpour and Abdolnabi (2017) showed that the vertical component of the earthquake does not affect fragility curves at different performance levels. They demonstrated that, for the intensity measure of a fragility curve, the spectral acceleration parameter (Sa) is better than peak ground acceleration (PGA). They also showed that structural vulnerability was raised with an increase in the number of stories.

Mohammadi et al. (2021) studied the seismic vulnerability of 2- and 4-story SMRFs with and without unreinforced masonry infills (UMI). The study suggests that structures with UMI have greater seismic resilience compared to those without UMI, particularly during high-intensity earthquakes and at immediate occupancy and life safety levels. Also, the bare frames compared to the frames with UMI have been seismically resilient at the collapse prevention level in all earthquake intensities. To calculate the fragility curves, two nonlinear modeling methods with distributed plasticity and concentrated plasticity are considered.

This article aims to explore how nonlinearity modeling affects the fragility of SMRF. To this end, a 6-story frame is optimally designed using the RUPSO algorithm, with the performance constraint of story drift. Subsequently, the fragility curves are calculated and compared using two different nonlinearity methods.

2. RUPSO Metaheuristic Optimization Algorithm

In the RUPSO algorithm, the same concepts as the PSO algorithm are used. Due to the local optimal solutions in some optimization problems, it is more difficult to reach the overall optimal solution. To effectively search the algorithm in the design space, Equation (1) is proposed for the random update of the particle position:

$$X(i)_{(t+1)} = X(X_g)_{(t)} + C^* \left(X(X_p)_{(t)} - X_r(i) \right) \quad (1)$$

where C is a number between 0.5 and 1 and depends on the domain of the design variables. If the design variables are close to each other, C takes on a value close to 0.5 and if the design variables are far from each other, it is closer to 1. $X_r(i)$ is an arbitrary position vector among the existing vectors. An unlimited number of acceptable answers could exist in the search space, with some being better than others.

Each vector in the population matrix conserves its best location to reach a better answer. Until a specific iteration, the best location of each vector is shown by $X(X_p)$, and The best answer among vectors is shown by $X(X_g)$.

3. Performance-Based Design of SMRF

In the performance-based design method, the structure is designed for predetermined performance levels under selected hazard levels. Failure levels, which are introduced as performance levels, are defined as deformation quantities such as strain, curvature, rotation, or displacement. Different hazard levels are defined as the probability of occurrence during the lifetime of the structure or with the average return period. Due to the uncertainty in the occurrence of different earthquake intensities, it is necessary to consider several intensity levels in the performance-based design to define performance aims. In the performance-based design method, at least three hazard levels are considered:

1) Basic service level earthquake: the probability of its occurrence is considered 50% in 50 years. Therefore, the probability of occurrence in the lifetime of the structure is high. The average return period of this earthquake is 72 years.
2) Design level earthquake: the probability of its occurrence is considered to be 10% in 50 years. The average return period of this earthquake is 475 years.
3) Maximum possible earthquake: the probability of its occurrence is considered to be 2% in 50 years. The average return period of this earthquake is 2475 years.

According to FEMA-302 guidelines, building performance is placed in one of three design groups (SUGs). Most residential, commercial, and industrial buildings are in the SUG-I group. Buildings containing hazardous materials and with numerous residents are classified in the SUG-II group. Buildings whose service is necessary after an earthquake are placed in the SUG-III group. According to FEMA-302 guidelines, for each design group, performance levels corresponding to hazard levels are suggested as shown in Figure (1).

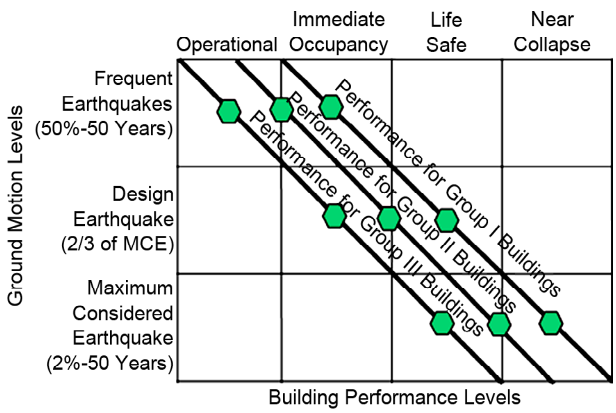

Figure 1. Performances corresponding to hazard levels based on the building design group.

In this figure, the vertical axis shows the earthquake hazard levels and the horizontal axis shows the corresponding performance levels for each design group. In evaluating the seismic performance of existing buildings and the performance-based design of structures, it is necessary to use non-linear analysis methods to ensure the achievement of desired performance levels. The use of non-linear static analysis has become more popular due to the inherent complexity and longtime of non-linear dynamic analysis calculations.

Nevertheless, non-linear static analysis is an approximate method and is not suitable for buildings that require the effects of higher modes. In non-linear static analysis, it is assumed that the response of the structure is controlled by the frequency of the first mode and the shape of this mode remains constant during the analysis, in this case, the effect of other modes is ignored. In this article, for the performance-based design of SMRF, the ASCE41-17 regulations have been used to perform non-linear static analysis. The target displacement corresponding to each hazard level is obtained according to Equation (2).

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{2}$$

In Equation (2), C_0 is the correction coefficient of the spectral displacement of the single degree of freedom system to the displacement of multi-degree of freedom for different loading patterns and is obtained according to the table (5-7) of the regulations. C_1 is the correction coefficient of the maximum expected inelastic displacement to the maximum expected elastic displacement, and is equal to 1 for periods greater than 1 second. The C_2 factor takes into account the effects of reducing the stiffness and resistance of structural members on the maximum displacement due to their non-linear behavior, and for periods greater than 0.7 second is equal to 1. The coefficient S_a is the structure's acceleration response spectrum based on the structure's main period. The lateral load distribution pattern is considered the same as the linear static method. In FEMA-350, story drift has been considered as a performance-based design constraint.

To carry out the performance-based design of the 6-story SMRF, it is assumed that the building will be built in Tehran and the Pasdaran area. The dead and live loads have been considered 650 and 200 kg/m² respectively. The uniform response spectrum for hazard levels of 50%, 10%, and 2% in 50 years is shown in Figure (2) (BHRC).

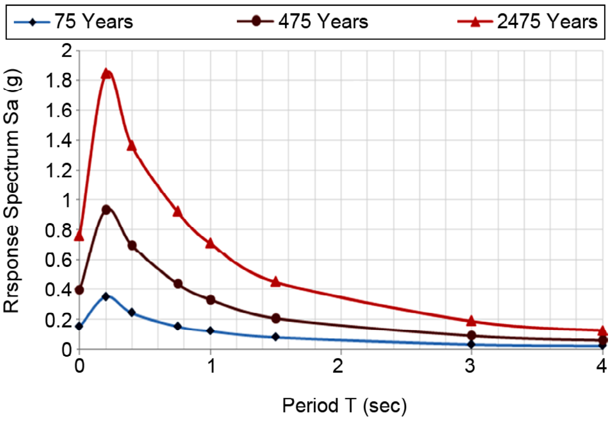

Figure 2. Uniform response spectrum of hazard levels of 50%, 10%, and 2% in 50 years (BHRC).

Three constraints have been considered in the optimally performance-based design of the 6-story SMRF. The first type is the geometric constraint of sections at its junction. The second constraint relates to the design criteria for SMRF members under gravity loads. According to LRFD-AISC, each member of a structure must satisfy the following constraints:

$$\text{For } \frac{P_u}{\phi_c P_n} < 0.2$$

$$\frac{P_u}{2\phi_c} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) - 1 \le 0 \tag{3}$$

$$\text{For } \frac{P_u}{\phi_c P_n} \ge 0.2$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9} \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) - 1 \le 0 \tag{4}$$

where P_n is the available axial strength, P_u is the required axial strength and ϕ_c is the reduction factor of the axial strength and is equal to 0.9. M_{ux} and M_{uy} are the required bending strengths, M_{ny} and M_{nx} are the available bending strengths in relation to the Y and X axes and ϕ_b is the strength reduction factor for bending and is equal to 0.9. The third constraint is the story drift criterion in the performance-based design of the SMRFs. The story drifts in the IO, LS, and CP levels are limited to 0.7%, 2.5%, and 5%, respectively. MATLAB software was utilized to optimally design of the SMRF, and the OpenSees software was used for the nonlinear analyses.

4. Nonlinearity Behavior Modeling of SMRF Elements

Two approaches are considered in this paper for modeling the nonlinear behavior of SMRF members to design and evaluate their performance. To design and evaluate the performance of a 6-story SMRF, all the elements of the beam and column of the frame are assumed to be nonlinear using the fiber element.

Distributed plasticity has been used to model the fiber nonlinear element. Also, to evaluate the performance of SMRF, concentrated plasticity has been considered. The modified Ibarra-Kravinkler model is used to model the non-linear concentrated hinge. In this modeling method, beams and columns are considered elastic elements, and it is assumed that the plastic hinges are formed at both ends of the element. The nonlinear behavior curve of both methods is shown in Figures (3) and (4).

The 6-story SMRF has nine design variables. Figure (5) shows the model of finite elements with concentrated plasticity and the typification of frame elements. The *W* sections obtained in the optimization process for the beam and column elements of the frame are according to Table (1).

Table 1. Sections of optimal 6-story SMRF.

Design Variables	Sections
C_1	W16 × 57
C_2	W16 × 40
C_3	W12 × 40
C_4	W14 × 68
C_5	W14 × 68
C_6	W12 × 40
B_1	W16 × 36
B_2	W16 × 36
B_3	W21 × 40
Total Weight (Kg)	11144

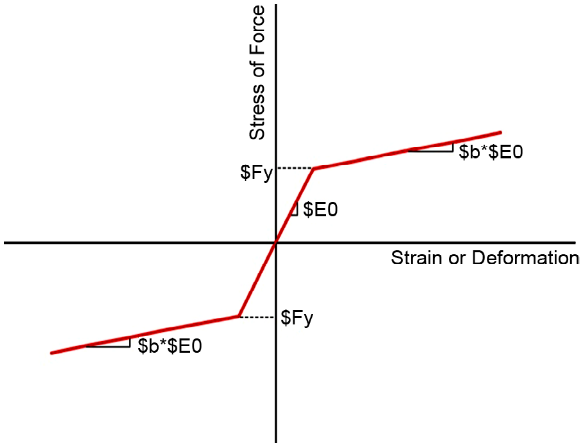

Figure 3. Bilinear model of nonlinear behavior with extensive plasticity (Lignos et al., 2014).

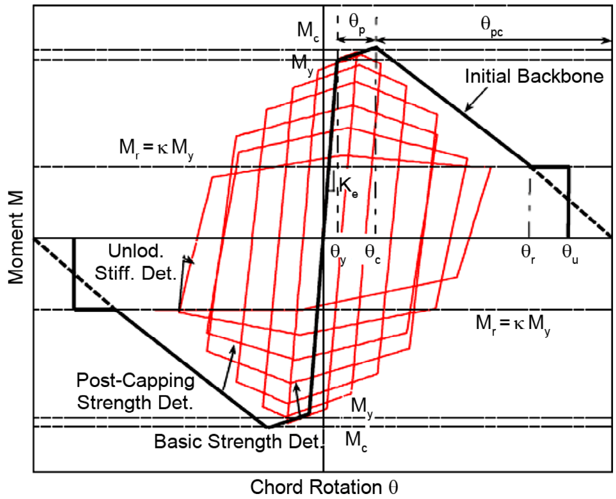

Figure 4. Backbone curve for modified Ibarra-Krawinkler model (Lignos et al., 2012).

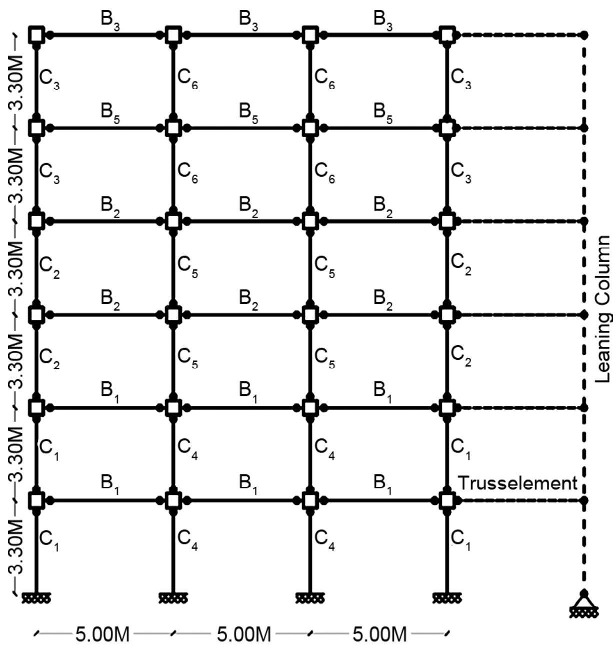

Figure 5. Finite element model of the 6-story frame with concentrated plasticity.

5. Fragility Curves for 6-Story SMRF

Incremental nonlinear time-history analysis was performed using 30 earthquake records to assess the performance of an optimally designed frame. These records are listed in Table A-4A of FEMA-P695 for soil type D. Considered records are listed in Table (2). Each of the records has two horizontal components.

The fragility curve expresses the probability of exceedance corresponding to a certain limit state in several levels of seismic ground motion. The mathematical function for calculating the fragility of a structural system is expressed as Equation (5).

$$P_{(Exceedance_i\,|GMI\,)} = \Phi\left[\frac{1}{\beta}\ln\left(\frac{GMI}{LS_i}\right)\right] \qquad (5)$$

where $P_{(Exceedance_i\,|GMI\,)}$ is the probability of exceedance in the i^{th} limit state in ground motion intensities-(GMI), Φ is the standard normal cumulative distribution function, and LS_i is the average value of the limit state capacity of a structure where it reaches the desired limit state. FEMA-P695 guidelines have introduced four important sources of uncertainty to evaluate the performance of structures, which are not considered in the performance-based design of structures. Record-to-record uncertainty (β_{RTR}), related to the difference in the response of the structure under different accelerograms, which is obtained from the results of the analysis of the records. Uncertainty of design criteria and detailing (β_{DR}), assuming ordinary design quality equal to 0.3, is considered. Uncertainty of test data (β_{TD}) is related to the quality of tests to determine the behavior of materials, which is assumed to be equal to 0.3 with the assumption of ordinary quality. The uncertainty related to the quality of nonlinear modeling of the structure (β_{MDL}) is assumed to be equal to 0.25. Total uncertainty (β_{TOT}) is calculated through the combination of the squares (SRSS) of the uncertainties as:

$$\beta_{TOT} = \sqrt{(\beta_{RTR})^2 + (\beta_{DR})^2 + (\beta_{TD})^2 + (\beta_{MDL})^2} \qquad (6)$$

The 6-story SMRF fragility curves under 30 records listed in Table (A-4A) of FEMA-P695 guidelines are as follows, Figures (6) and (7).

Table 2. Specifications of accelerograms used to evaluate frames.

Earthquake			Recording Station
Name	**Year**	**Magnitude (Ms)**	**Name**
Northridge	1994	6.7	Beverly Hills-Mulholland
Northridge	1994	6.7	Canyon Country-WLC
Duzce, Turkey	1999	7.1	Bolu
Imperial Valley	1979	6.5	Delta
Imperial Valley	1979	6.5	El Centro array
Kobe, Japan	1995	6.9	Shin-Osaka
Kocaeli, Turkey	1999	7.5	Duzce
Landers	1992	7.3	Yermo Fire Station
Landers	1992	7.3	Cool Water
Loma Prieta	1989	6.9	Capitola
Loma Prieta	1989	6.9	Gilroy Array
Superstition Hills	1987	6.5	El Centro imp
Superstition Hills	1987	6.5	Poe Road
Chi-Chi, Taiwan	1999	7.6	CHY101
San Fernando	1971	6.6	LA-Hollywood Stor

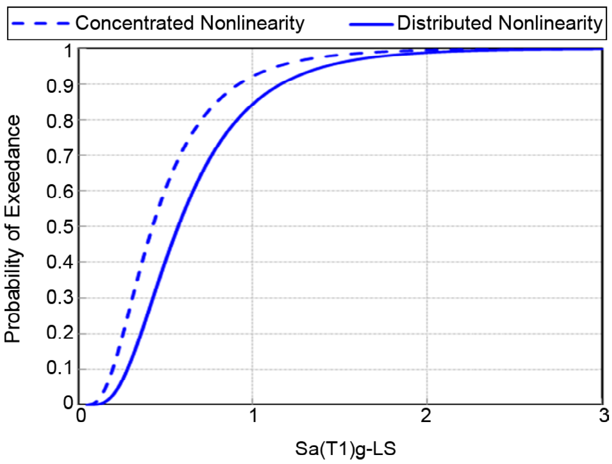

Figure 6. Fragility curves of 6-story SMRF at Life Safety performance level.

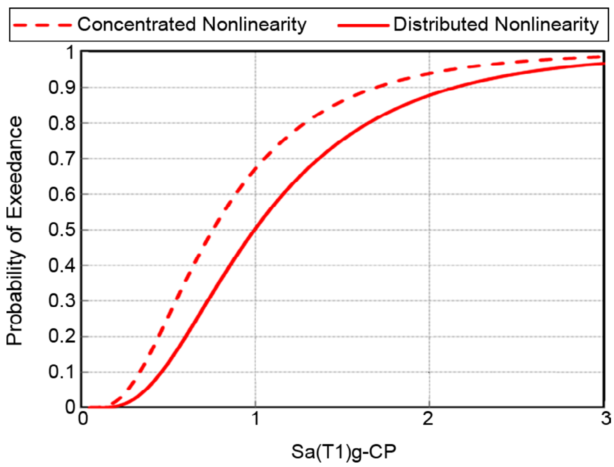

Figure 7. Fragility curves of 6-story SMRF at Collapse Prevention performance level.

6. Conclusions

The 6-story SMRF has been optimally designed using the RUPSO algorithm with the performance constraint of story drift and fiber elements. The objective function was to minimize the weight of the structure. The performance of the optimally designed frame has been assessed using fragility curves with two nonlinearity modeling methods. The results are as follows:

- The fragility of the SMRF with nonlinear modeling of concentrated plasticity is higher than the same frame with distributed plasticity modeling in both Collapse Prevention and Life Safety performance levels.
- The probability of exceeding the Life Safety performance level of the SMRF with concentrated plasticity at Sa = 0.6g has increased by 26% compared to the same frame with distributed plasticity.
- The probability of exceeding the Collapse Prevention performance level of the SMRF with concentrated plasticity at Sa = 0.6g has increased by 44% compared to the same frame with distributed plasticity.
- The probability of exceedance in SMRF with concentrated nonlinearity compared to distributed nonlinearity in collapse prevention performance level has increased compared to life safety performance level.

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