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SEISMIC TOPOLOGY OPTIMIZATION AND COLLAPSE SAFETY ANALYSIS OF CHEVRON BRACED FRAMES

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ABSTRACT

This study is devoted to seismic collapse safety analysis of performance based optimally seismic designed steel chevron braced frame structures. An efficient meta-heuristic algorithm namely, center of mass optimization is utilized to achieve the seismic optimization process. The seismic collapse performance of the optimally designed steel chevron braced frames is assessed by performing incremental dynamic analysis and determining their adjusted collapse margin ratios. Two design examples of 5-, and 10-story chevron braced frames are illustrated. The numerical results demonstrate that all the performance-based optimal designs are of acceptable seismic collapse safety.

Keywords: optimization; performance-based design; chevron braced frame; incremental dynamic analysis; collapse margin ratio.

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1. INTRODUCTION

One of the modern seismic design procedures for the rehabilitation of existing structural systems and the seismic design of new structures is performance-based design (PBD) [1] which its main objective is to decrease vulnerability of structures subject to earthquake. In the PBD approach, nonlinear analysis procedures are employed to evaluate the seismic response of structures. Pushover analysis is a simplified static nonlinear procedure in which a predefined pattern of earthquake loads is applied incrementally to structures until a plastic collapse mechanism is reached. One of the major concerns of structural engineers and designers is to find cost-efficient structures having acceptable performance subject to earthquake. To this end, structural optimization methodologies were developed in the last decades. Structural performance-based optimal design (PBOD) is a topic of growing interest [2-11]. One of the common lateral load resisting systems is steel braced frames that have a

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high stiffness compared with steel moment resisting frames. Concentrically steel braced frames are popular lateral load resisting systems because of their high axial stiffness. There are various barcing configurations as per in AISC 341-16 code [12] including, diagonal, X, V and inverted V-shaped braces. The V and inverted V-shaped braces are known as chevron braces that are suitable in architectural viewpoint. This study focuses on inverted V-shaped chevron braced frame (CBF) structures.

In order to deal with the PBOD problems, it is necessary to use global search algorithms such as metaheuristics. Metaheuristics are designated based on stochastic natural phenomena and they have attracted a great deal of attention during the last two decades. As the metaheuristic optimization techniques require no gradient computations, they are simple for computer implementation. During the recent years, researchers have designed many metaheuristic algorithms and many successful applications of them have been reported in optimization literature. In the current study, center of mass optimization (CMO) [13] is applied to solve PBOD problem of steel CBFs because its ability for tackling PBOD problems of steel structures have been demonstrated in the previous studies [13-14].

Seismic collapse capacity of structures is one of the most important concerns of structural engineers. In order to determine the seismic safety factor of structures, collapse fragility curves must be developed by performing incremental dynamic analysis (IDA) [15] for a prescribed set of ground motions whose amplitudes are scaled to reflect specified earthquake intensities. Subsequently, the collapse margin ratio (CMR) of structures can be determined based on the methodology of FEMA-P695 [16] provided for quantifying building system performance in the context of collapse safety.

In the present work, 5-, and 10-story CBFs are optimized in the framework of PBD by using CMO metaheuristic and then the seismic collapse safety of the optimal designs are assessed. The numerical results indicate that seismic collapse safety of the optimal CBFs are acceptable.

2. PERFORMANCE-BASED TOPOLOGY OPTIMIZATION

In PBD frameworks, a performance objective is defined as a given level of performance for a specific hazard level. To define a performance objective, at first the level of structural performance should be selected and then the corresponding seismic hazard level should be determined. In the present work, immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance levels are considered according to FEMA-356. Each objective corresponds to a given probability of being exceed during 50 years. A usual assumption is that the IO, LS and CP performance levels correspond respectively to a 20%, 10% and 2% probability of exceedance in 50 year period. In this work, the nonlinear static pushover analysis is utilized to quantify seismic induced nonlinear response of structures. Among various methods of static pushover analyses, the displacement coefficient method [1] procedure is adopted to evaluate the seismic demands on building frameworks under equivalent static earthquake loading. In this method the structure is pushed with a specific distribution of the lateral loads until the target displacement is reached. The target displacement can be obtained as follows:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{1}$$

where C_0 relates the spectral displacement to the likely building roof displacement; C_1 relates the expected maximum inelastic displacements to the displacements calculated for linear elastic response; C_2 represents the effect of the hysteresis shape on the maximum displacement response and C_3 accounts for P-D effects. T_e is the effective fundamental period of the building in the direction under consideration; S_a is the response spectrum acceleration corresponding to the T_e; and g is ground acceleration.

In topology optimization of CBFs, besides the cross-section of members, the placement of braces are taken as the topology design variables. In this case, during the optimization process, unnecessary bracing members are removed from a fully braced frame. In the topology optimization problem of CBFs, the aim is to minimize the weight of the structure under some behavioral constraints. For a CBF consisting of a number of members that are collected in *ng* design groups, the optimization problem can be formulated as follows:

$$\text{Find: } X \tag{2}$$

$$\text{To minimize: } w(X) = \sum_{i=1}^{ng} \rho_i A_i \sum_{j=1}^{nm} L_j \tag{3}$$

$$\text{Subject to: } g_k(X) \le 0\,,\; k = 1,2,\cdots,nc \tag{4}$$

where X is a vector of design variables; w represents the weight of the frame, ρ_i and A_i are weight of unit volume and cross-sectional area of the ith group section, respectively; nm is the number of elements collected in the ith group; L_j is the length of the jth element in the ith group; $g_k(X)$ is the kth behavioral constraint; and nc is the number of constraints. In the present study, design variables are selected from standard sections found in the AISC design manual.

A set of design constraints should be checked during the optimization process. The geometric constraints are checked to ensure the consistency of dimensions of beams and columns in all framing joints. The strength of structural elements is checked for gravity loads to perform serviceability checks based on AISC [17] design code. If the serviceability checks are not satisfied then the candidate design is rejected, else a nonlinear pushover analysis is conducted in order to evaluate the structural responses at performance levels. In order to implement pushover analysis to evaluate the seismic demands of the structures, the target displacement should be determined. To achieve this task, S_a should be calculated for the three performance levels. In this case three acceleration design spectra, which represent three different earthquake levels corresponding to 20%, 10%, and 2% probability of exceeding in a 50-year period, are taken as the basis for calculating the seismic loading for the three performance levels IO, LS, and CP, respectively. In the present work, the acceleration design spectra are determined for each hazard level according to the Standard No. 2800 [18]. The inter-story drift constraints, the plastic rotation constraints for beams and columns and the axial deformation constraints of the braces at each performance level are checked during the optimization process according to FEMA-356 [1] and ASCE 41-13 [19]. In addition, to implement the design constraints, the exterior penalty function method (EPFM) [20] is used.

3. METAHEURISTIC ALGORITHMS

The main idea behind designing the metaheuristic algorithms is to tackle complex optimization problems where other optimization methods have failed to be effective. Metaheuristics are applied to a very wide range of problems and they mimic natural metaphors to solve complex optimization problems. In this study, CMO metaheuristic is applied to solve the PBOD optimization problem of CBFs.

3.1 Center of mass optimization

CMO was proposed in [13] based on the concept of center of mass in physics. In CMO algorithm, a population including np randomly selected particles $(X_i\, , i \in [1,np])$ is generated in design space. The mass of ith particle m_i is determined as follows

$$m_i = \frac{1}{f(X_i)} \tag{5}$$

Particles are sorted based on their mass values in ascending order and then they are equally divided into two groups of G1 and G2. The first half of particles are put in G1 and the others in G2. The particles in G1 are paired with their corresponding ones in G2. The position of center of mass and the distance between jth (j=1,..., $np/2$) pair of particles in iteration t are determined as follows

$$X_j^C(t) = \frac{m_j X_j(t) + m_{j+\frac{np}{2}} X_{j+\frac{np}{2}}(t)}{m_j + m_{j+\frac{np}{2}}} \tag{6}$$

$$d_j(t) = \left| X_j(t) - X_{j+\frac{np}{2}}(t) \right| \tag{7}$$

In order to switch between exploration and exploitation of CMO algorithm, the following parameter is computed in which t_{max} is the maximum number of iterations.

$$CP(t) = \exp\left(-\frac{5t}{t_{max}}\right) \tag{8}$$

The position of jth couple of particles is updated using the following equations

$$if\ d_j(t) > \ CP(t) \tag{9}$$

$$X_j(t+1) = X_j(t) - R_1\left(X_j^C(t) - X_j(t)\right) + R_2\left(X_b - X_j(t)\right) \tag{10}$$

$$X_{j+\frac{np}{2}}(t+1) = X_{j+\frac{np}{2}}(t) - R_3\left(X_j^C(t) - X_{j+\frac{np}{2}}(t)\right) + R_4\left(X_b - X_{j+\frac{np}{2}}(t)\right) \tag{11}$$

$$if\ d_j(t) \leq \ CP(t) \tag{12}$$

$$X_j(t+1) = X_j(t) + R_5\left(X_j^C(t) - X_{j+\frac{np}{2}}(t)\right) \tag{13}$$

$$X_{j+\frac{np}{2}}(t+1) = X_{j+\frac{np}{2}}(t) + R_6\left(X_j^C(t) - X_{j+\frac{np}{2}}(t)\right) \tag{14}$$

where R_1 to R_6 are vector of random numbers in [0,1]; and X_b is the best solution found.

There is a mutation operator in CMO to decrease the probability of local optima entrapment. A mutation rate $mr = 0.2$ is taken and in iteration t a number between 0 and 1 is randomly selected for each particle in group G1 (X_j , j=1,..., $np/2$).

$$r_j(t) \in [0, 1] \tag{15}$$

$$X_j(t) = \{x_{j1}(t) \quad x_{j2}(t) \quad \dots \quad x_{ji}(t) \quad \dots \quad x_{jm}(t)\}^{\mathrm{T}} \tag{16}$$

For jth particle, if the selected random number is less than the mutation rate, one randomly selected component will be regenerated in the design space as follows

$$if\ r_j(t) \le mr \ \rightarrow \ x_{ji}(t) = x_{ji}^L + \mu(t) \times \left(x_{ji}^U - x_{ji}^L\right) \tag{17}$$

where $\boldsymbol{\mu}$ is a random number in interval [0, 1] in iteration $\boldsymbol{t}$; and $\boldsymbol{x_{ij}^L}$ and $\boldsymbol{x_{ij}^U}$ are lower and upper bounds of $\boldsymbol{x_{ji}}$ in design space.

4. SEISMIC COLLAPSE SAFETY ASSESSMENT

The methodology proposed by FEMA-P695 [16] is an efficient IDA-based approach to assess the collapse capacity of structures. In this methodology, many nonlinear time-history analyses should be implemented for a suit of 22 ground motions listed in Table 1.

Table 1: Ground motion records set

Name	M	Year	Record Station
Northridge	6.7	1994	Beverly Hills - Mulhol
Northridge	6.7	1994	Canyon Country-WLC
Duzce, Turkey	7.1	1999	Bolu
Hector Mine	7.1	1999	Hector
Imperial Valley	6.5	1979	Delta
Imperial Valley	6.5	1979	El Centro Array #11
Kobe, Japan	6.9	1995	Nishi-Akashi
Kobe, Japan	6.9	1995	Shin-Osaka
Kocaeli, Turkey	7.5	1999	Duzce
Kocaeli, Turkey	7.5	1999	Arcelik
Landers	7.3	1992	Yermo Fire Station
Landers	7.3	1992	Coolwater

Loma Prieta	6.9	1989	Capitola
Loma Prieta	6.9	1989	Gilroy Array #3
Manjil, Iran	7.4	1990	Abbar
Superstition Hills	6.5	1987	El Centro Imp. Co.
Superstition Hills	6.5	1987	Poe Road (temp)
Cape Mendocino	7.0	1992	Rio Dell Overpass
Chi-Chi, Taiwan	7.6	1999	CHY101
Chi-Chi, Taiwan	7.6	1999	TCU045
San Fernando	6.6	1971	LA - Hollywood Stor
Friuli, Italy	6.5	1976	Tolmezzo

IDA curves are developed by recording maximum inter-story drift ratio versus the 5% damped spectral acceleration at structural fundamental period, $S_a(T_1,5\%)$. Collapse margin ration (CMR) of structures is defined as the ratio of the spectral acceleration for which half of the pre-defined earthquake records cause collapse ($S_a^{50\%}$) to the spectral acceleration of the maximum considered earthquake (MCE) ground motion (S_a^{MCE}) as follows:

$$CMR = \frac{S_a^{50\%}}{S_a^{\text{MCE}}} \quad (18)$$

To account for the spectral shape of ground motion records, an adjusted collapse margin ration (ACMR) is defined as follows in which SSF is the spectral shape factor [16].

$$ACMR = SSF \times CMR \quad (19)$$

Furthermore, to address the effect of different uncertainty sources in seismic collapse safety analysis of structures the composite uncertainty parameter, β_{TOT}, should be determined.

Acceptable values of $ACMR$ for a single design and a design group are denoted in FEMA-P695 by $ACMR_{20\%}$ and $ACMR_{10\%}$, respectively. In other words, a single design can be considered of acceptable collapse safety when its $ACMR$ is greater than $ACMR_{20\%}$. In addition, for a group of desgins, if the average of their $ACMR$s is greater than $ACMR_{10\%}$ their colllapse safety is considered to be acceptable.

5. NUMERICAL EXAMPLES

In this study, CBFs are modeled based on pinned-ended braces and beams. For braces the uniaxial phenomenological model is considered according to FEMA-274 [21]. The nonlinear behavior of columns is modeled by a simple bilinear stress-strain relationship with strain hardening 0.3% of the elastic modulus. The modulus of elasticity and the yield stress are respectively 200 GPa and 344.74 MPa. The dead and live loads of 2500 and 1000 kg/m are applied to the all beams, respectively. the sections of structural elements are selected from the database of steel sections listed in Table 2.

Table 2: The available list of standard sections

Columns		Beams		Bracings	
No.	**Profile**	**No.**	**Profile**	**No.**	**Profile**
1	W14×48	1	W12×19	1	HSS5×5×0.500
2	W14×53	2	W12×22	2	HSS6×6×0.500
3	W14×68	3	W12×35	3	HSS6×6×0.625
4	W14×74	4	W12×50	4	HSS5×5×0.500
5	W14×82	5	W18×35	5	HSS6×6×0.500
6	W14×132	6	W16×45	6	HSS6×6×0.625
7	W14×145	7	W18×40	7	HSS5×5×0.500
8	W14×159	8	W16×50	8	HSS6×6×0.500
9	W14×176	9	W18×46	9	HSS6×6×0.625
10	W14×193	10	W16×57	10	HSS5×5×0.500
11	W14×211	11	W18×50	11	HSS6×6×0.500
12	W14×233	12	W21×44	12	HSS6×6×0.625
13	W14×257	13	W21×50	13	HSS5×5×0.500
14	W14×283	14	W21×57	14	HSS6×6×0.500
15	W14×311	15	W24×55	15	HSS6×6×0.625
16	W14×342	16	W21×68	16	HSS8×8×0.500
17	W14×370	17	W24×62	17	HSS7×7×0.625
18	W14×398	18	W24×76	18	HSS8×8×0.625
19	W14×426	19	W24×84	19	HSS9×9×0.625
20	W14×455	20	W27×94	20	HSS10×10×0.625

5.1 Example 1: 5-story CBF

In the first example, the optimal designs for fixed bracing topologies, indicated by DFT1 to DFT3, and for 9 designs with optimal bracing topologies, denoted by DT1 to DT9, are reported in Fig. 1 and Table 3.

Table 3: PBD topology optimization results for 5-story CBF

Variables	**DFT1**	**DFT2**	**DFT3**	**DT1**	**DT2**	**DT3**	**DT4**	**DT5**	**DT6**	**DT7**	**DT8**	**DT9**
C1	6	6	7	5	5	5	5	5	5	5	5	5
C2	3	4	5	2	2	2	2	2	2	2	2	2
C3	3	4	5	2	2	2	2	2	2	2	2	2
C4	7	8	10	6	6	6	6	6	6	6	6	6
C5	3	3	3	2	2	2	2	2	2	2	2	2
C6	3	2	3	2	2	2	2	2	2	2	2	2
Br1	9	7	9	7	7	6	7	7	7	7	7	7
Br2	15	9	11	14	14	7	13	13	13	15	15	15
Br3	6	7	10	5	5	5	5	5	5	5	5	5
Br4	5	9	13	14	14	14	14	14	14	14	14	14
Br5	9	9	13	7	7	12	8	8	14	7	7	7
B1	10	10	5	6	6	6	6	6	6	6	6	6
Weight (kg)	15435.2	17119.7	17503.4	12943.6	12943.6	13125.3	13425.3	13425.3	13527.1	13823.3	13823.3	13823.3

The numerical results reported in Table 3 indicate that DFT1 is the best design with fixed bracing topologies and DT1 is the best topologically optimal design that its weight is 16.14% lighter than that of DFT1.

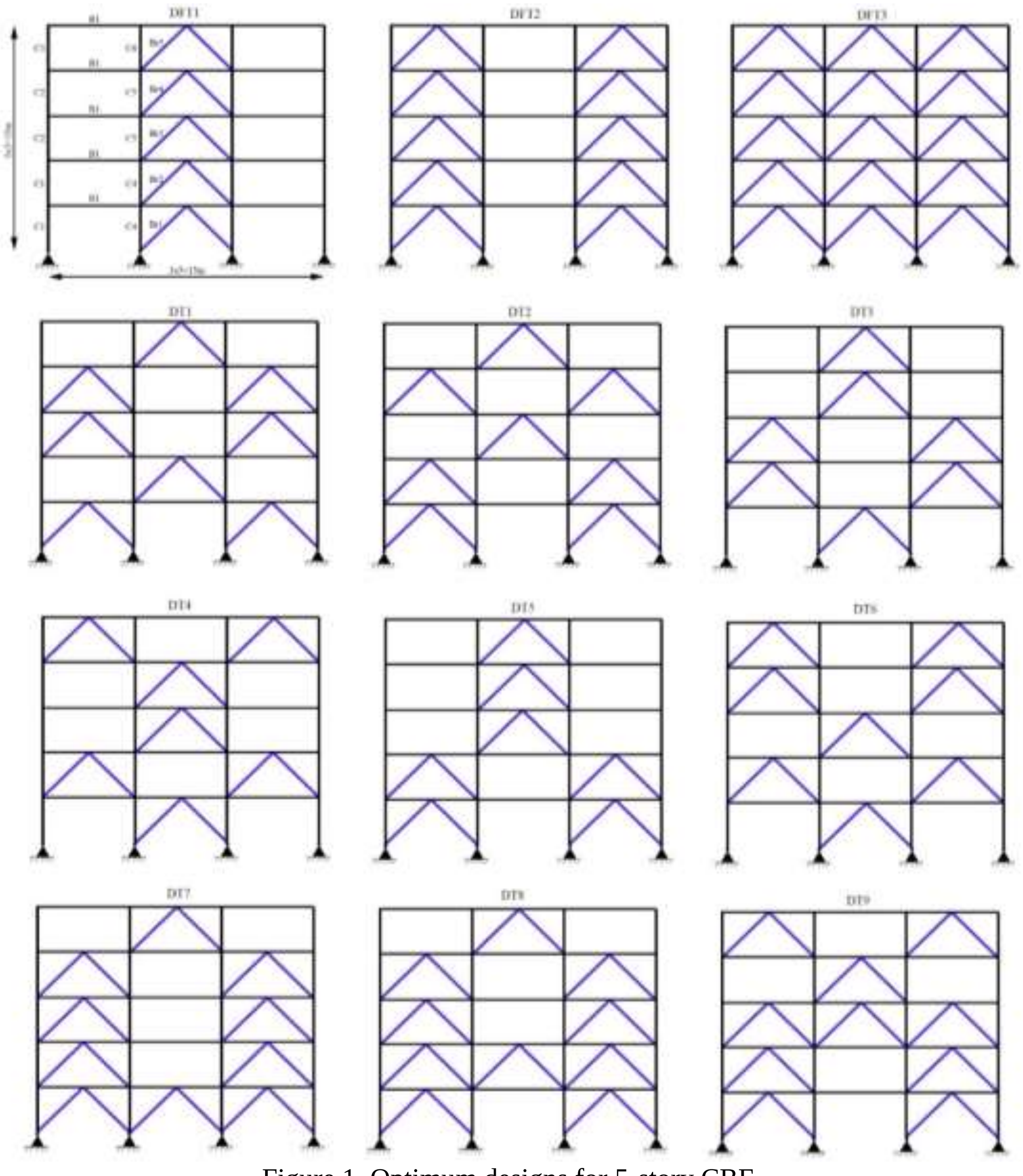

Figure 1. Optimum designs for 5-story CBF

The fragility curves of the optimal designs are depicted in Fig. 2. The results of seismic collapse safety assessment of the optimally designed 5-story CBF structures are summarized in Table 4. It can be observed that *ACMR* values of all the optimal designs are greater than $ACMR_{20\%}$ and these optimal designs are of significant collapse safety. The average *ACMR* for the designs with fixed and optimal bracing topologies are 3.94 and 3.95, respectively.

The numerical results of the present example demonstrate that among the all the DT optimal designs, DT8 provides the largest *ACMR*.

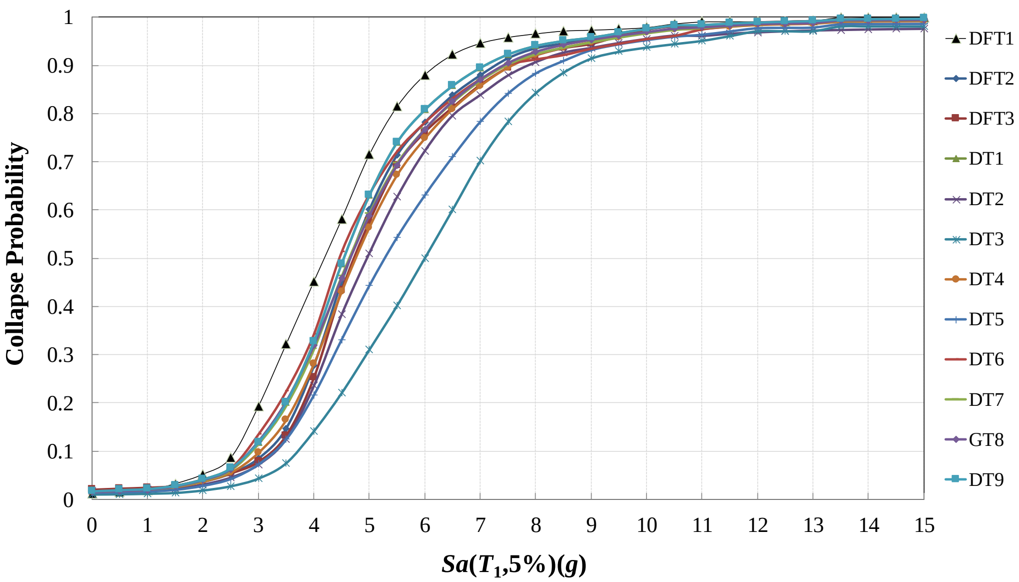

Figure 2. Fragility curves for the optimum 5-story CBFs

Table 4: Seismic collapse safety parameters for optimal 5-story CBFs

Optimal Designs	*CMR*	*SSF*	*ACMR*	$ACMR_{20\%}$	Pass/Fail
DFT1	3.01	1.14	3.43	1.52	P
DFT2	3.39	1.15	3.89	1.52	P
DFT3	3.94	1.14	4.49	1.52	P
DT1	3.45	1.14	3.93	1.52	P
DT2	3.35	1.14	3.82	1.52	P
DT3	3.47	1.13	3.92	1.52	P
DT4	3.38	1.13	3.82	1.52	P
DT5	3.16	1.13	3.57	1.52	P
DT6	3.13	1.15	3.59	1.52	P
DT7	3.69	1.15	4.25	1.52	P
DT8	3.79	1.16	4.39	1.52	P
DT9	3.71	1.15	4.26	1.52	P

5.2 Example 2: 10-story CBF

Three optimal designs for fixed bracing topologies (DFT1 to DFT3), and for 9 optimal designs with optimal bracing topologies (DT1 to DT9) are given in Fig. 3 and Table 5.

The numerical results reported in Table 5 indicate that DFT3 is the best design with fixed bracing topologies and DT1 is the best topologically optimal design that its weight is 12.46% lighter than that of DFT3.

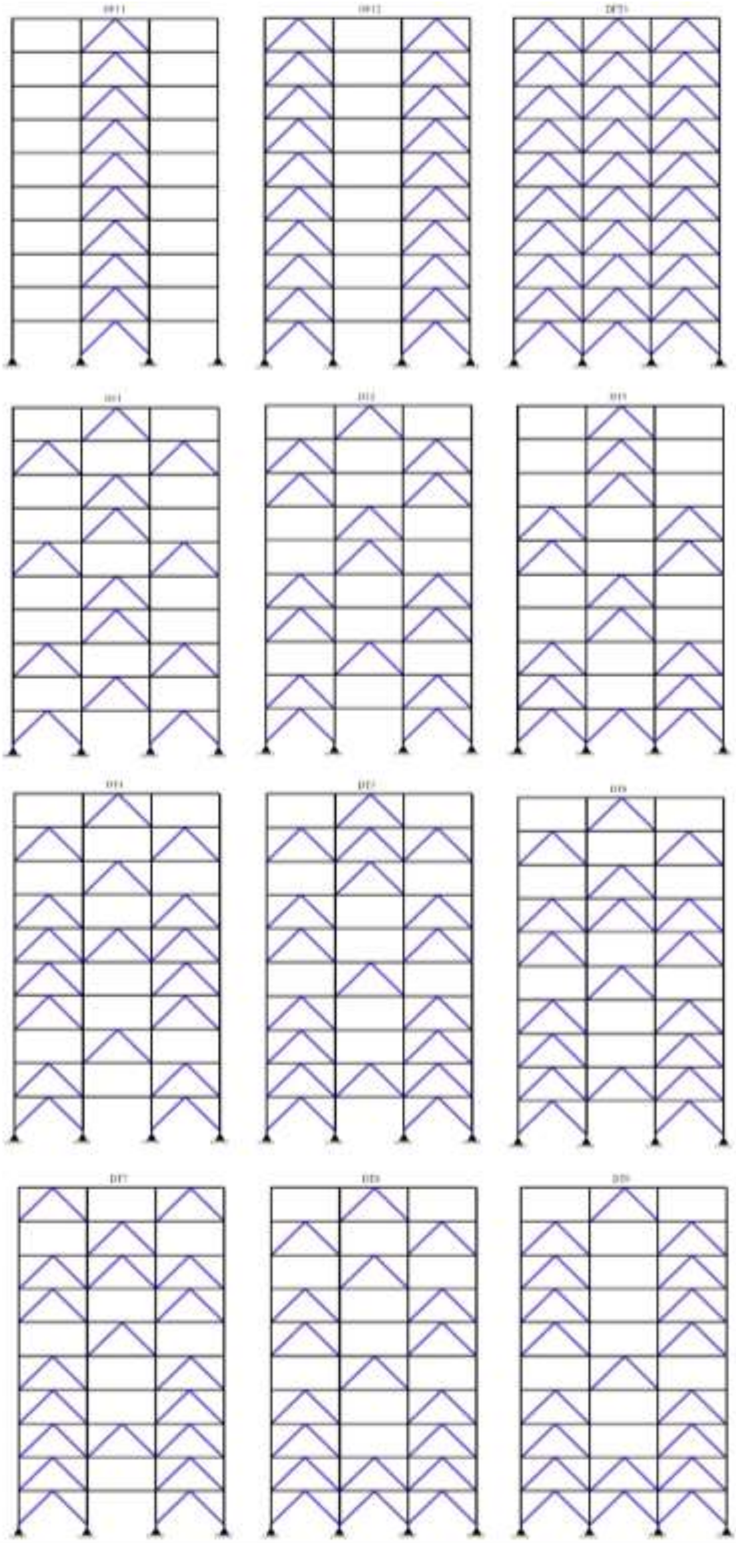

Figure 3. Optimum designs for 10-story CBF

Table 5: PBD topology optimization results for 10-story CBF

Variables	**DFT1**	**DFT2**	**DFT3**	**DT1**	**DT2**	**DT3**	**DT4**	**DT5**	**DT6**	**DT7**	**DT8**	**DT9**
C1	7	6	6	6	6	10	1	6	6	6	1	6
C2	6	6	5	2	6	6	2	6	6	6	1	2
C3	5	3	3	2	2	5	2	3	2	3	2	1
C4	1	1	1	1	1	1	1	1	1	1	1	1
C5	1	1	1	1	1	1	1	1	1	1	2	1
C6	11	15	5	3	2	1	2	2	1	2	2	3
C7	7	13	1	1	2	3	2	1	4	2	1	2
C8	6	6	1	1	1	1	1	1	1	1	2	1
C9	1	1	1	3	1	1	1	1	1	1	1	1
C10	1	1	1	1	1	1	1	1	1	1	1	1
Br1	19	14	14	18	18	18	19	19	18	19	18	18
Br2	18	14	14	19	17	17	18	18	18	18	18	18
Br3	18	14	13	19	18	14	19	18	17	18	17	16
Br4	17	14	13	19	13	18	14	16	17	17	17	16
Br5	17	14	13	18	13	18	17	16	18	17	18	18
Br6	17	14	13	15	14	13	17	19	14	18	14	14
Br7	16	14	12	16	14	13	17	18	14	15	14	14
Br8	16	13	12	16	10	12	14	18	13	15	13	12
Br9	14	13	7	12	10	12	8	8	14	16	14	12
Br10	10	12	7	8	8	7	8	8	14	10	7	7
B1	3	3	3	3	3	3	3	3	3	3	3	3
Weight (kg)	38584.2	39482.5	38358.7	33576.4	33985.1	34468.8	34896.7	35126.6	35796.2	36076.8	36483.1	36865.5

The fragility curves of the optimal designs are depicted in Fig. 4. The results of seismic collapse safety assessment of the optimally designed 10-story CBF structures are summarized in Table 6. It can be observed that *ACMR* values of all the optimal designs are greater than $ACMR_{20\%}$ and these optimal designs are of significant collapse safety. The average *ACMR* for the designs with fixed and optimal bracing topologies are 2.92 and 2.84, respectively.

The numerical results of the present example demonstrate that among the all optimal designs, the largest *ACMR* belongs to DT4 and DT8.

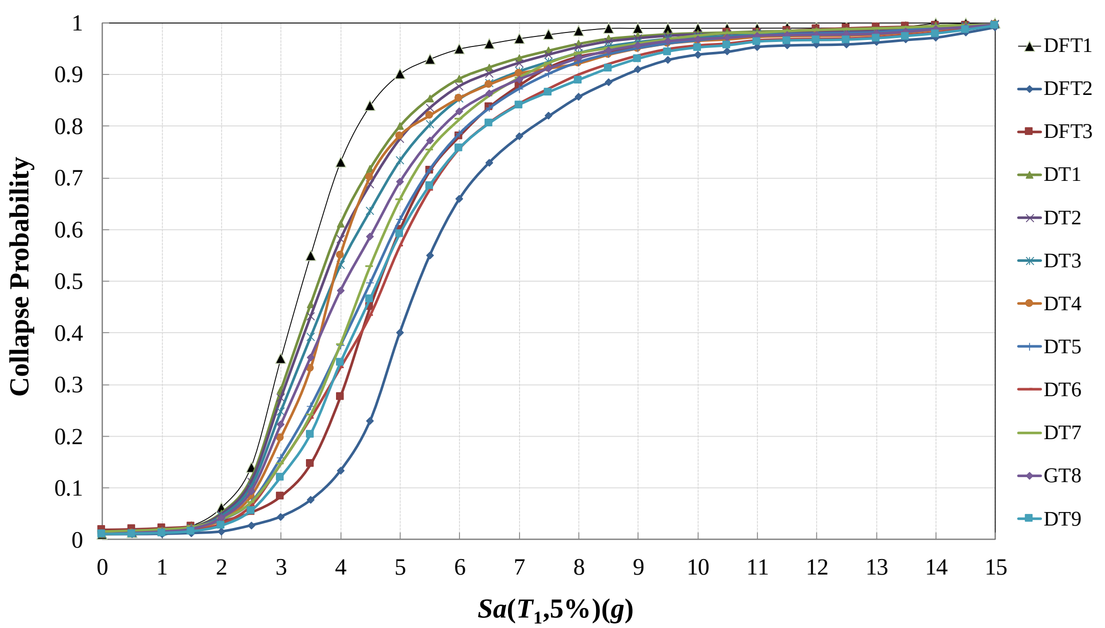

Figure 4. Fragility curves for the optimum 10-story CBFs

Table 6: Seismic collapse safety parameters for optimal 10-story CBFs

Optimal Designs	***CMR***	***SSF***	***ACMR***	$ACMR_{20\%}$	**Pass/Fail**
DFT1	2.03	1.13	2.31	1.52	P
DFT2	2.86	1.11	3.17	1.52	P
DFT3	3.32	1.11	3.28	1.52	P
DT1	2.26	1.11	2.51	1.52	P
DT2	2.30	1.11	2.56	1.52	P
DT3	2.27	1.12	2.54	1.52	P
DT4	2.85	1.12	3.19	1.52	P
DT5	2.31	1.12	2.59	1.52	P
DT6	2.67	1.12	2.99	1.52	P
DT7	2.68	1.13	3.03	1.52	P
DT8	2.82	1.13	3.19	1.52	P
DT9	2.61	1.13	2.95	1.52	P

6. CONCLUSIONS

Performance-based topology optimization and seismic collapse safety analysis of steel chevron braced frame structures is implemented in the present study. Center of mass optimization as an efficient meta-heuristic is selected as the optimizer of this work. During the optimization process, the design spectra of the Iranian seismic code 2800 are used and the constraints are checked according to FEMA-356 and AISC 341-16 design codes. Incremental dynamic analysis is performed for seismic collapse performance analysis of optimally designed steel chevron braced frames. The seismic collapse capacity of the obtained optimal designs is assessed based on the methodology of FEMA-P695. Two illustrative examples of 5-, and 10-story chevron braced frames are presented. The results indicate that in 5-, and 10-story chevron braced frames the optimal design having the best topology in terms of structural weight is 16.14% and 12.46% lighter than optimal designs with fixed bracing topology, respectevily. Based on the results of seimic collapse fragility analysis it can be stated that all the optimal designs are of acceptable seismic collapse safety. Finally it can be concluded that, in comparision with pure size optimization, seismic topology optimization of CBFs leads to better designs in terms of optimal weight with almost the same level of seismic collapse safety.

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