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Seismic performance evaluation of buckling restrained braced frames (BRBF) using incremental nonlinear dynamic analysis method (IDA)

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Abstract. In this paper, the seismic behavior of BRBF structures is studied and compared with special concentric braced frames (SCBF). To this purpose, three BRBF and three SCBF structures with 3, 5 and 10 stories are designed based on AISC360-5 and modelled using OpenSees. These structures are loaded in accordance with ASCE/SEI 7-10. Incremental nonlinear dynamic analysis (IDA) are performed on these structures for 28 different accelerograms and the median IDA curves are used to compare seismic capacity of these two systems. Results obtained, indicates that BRBF systems provide higher capacity for the target performance level in comparison with SCBF systems. And structures with high altitude (in this study, 5 and 10 stories) with the possibility of exceeding the collapse prevention performance level, further than lower altitude (here 3 floors) structures.

Keywords: incremental nonlinear dynamic analysis (IDA); buckling restrained braced frame (BRBF); special concentric braced frame (SCBF); seismic performance; level exceedance probability; analytical fragility curve

1. Introduction

Concentric braced frames are one of the most common lateral resistance, structural system in steel structures. The wide use of this system is due to its relatively simple design and construction process. In addition, concentric braces are economically efficient in comparison with the other existing systems. Anyway, severe damages to braced structures in past earthquakes such as Mexico (1985), Loma Prieta (1989), Northridge (1994), Kobe (1995) challenged structural engineers on the seismic performance of the traditional concentric braced systems. It is evident that buckling of braces in compression is the main cause of undesirable behavior of traditional bracing systems. Therefore, extensive research works have been performed during past two decades to develop braces with more favorable and appropriate elasto-plastic performance. One of the outcomes of these attempts is Buckling Restrained Brace (BRB). A BRB is a steel brace which is prevented from buckling in compression by means of an external mechanism (Mohammadhassani *et al*. 2012, Mohammadhassani *et al*. 2013, Mohammadhassani *et al*. 2014). The most prevalent method to prevent from compression buckling is to place the steel brace as core inside a steel casing and filling the casing with a mortar or concrete. This system can yield in both tension and compression and therefore its energy dissipation capacity increases significantly. Using BRB instead of traditional braces in steel structures improves seismic behavior of structures, Furthermore, it leads to a more efficient analysis and design process, because nonlinear dynamic analysis can be based on a more reliable and realistic modeling of braces.

A lot of researchers have done studies in terms of improvement of the building component performances in order to introduce solutions to have more resistance structures as well as reducing the probable damage under any kind of disasters such as earthquake, strong winds and flood (Azimi *et al*. 2015, Azimi *et al*. 2015, Alhajri *et al*. 2016, Bazzaz *et al*. *et al*. 2016, Ma *et al*. 2016, Ma *et al*. 2016). Considerable advances in the field of computer science have led to the development of various nonlinear analysis methods to simulate the structural performance, in recent years. Among them, incremental nonlinear dynamic analysis or IDA is a parametric technique which has been proposed to study seismic behavior of structures. Today, the nonlinear response of a structure subjected to a suite of ground motions is predictable by a relatively new approach so called Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002) in that response history

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analyses of a given structure is calculated through a systematic manner which will be discussed later. This method is widely used for seismic evaluation of nonlinear response of structures subjected to suite of severe strong motion (Niknam *et al*. 2007, Jalali *et al*. 2012, Farahi and Mofid 2013, Azimi *et al*. 2015)

In this paper, seismic response of buckling restrained braced frames and special concentric braced structures is evaluated for various performance levels using IDA.

2. Incremental nonlinear dynamic analysis (IDA)

Incremental nonlinear dynamic analysis (IDA) is an effective method for seismic analysis of structures in which a structure is subjected to one or many ground motions, that are scaled equivalently, and is reanalyzed for increasing intensities until collapse. This method is introduced by Bertero in 1977 (Popov and Pister 1980). Since the introduction, IDA method was utilized for various applications by many researchers (Nassar and Krawinkler 1991, Bazzaz *et al*. 2015, Bazzurro and Cornell 1994, Luco and Cornell 1998, Mehanny and Deierlein 1999, Luco and Cornell 2000, Yun *et al*. 2002, Hakim *et al*. 2011, Mohammadhassani *et al*. 2012). By applying IDA, different intensity measures (IM) such as peak ground acceleration (PGA) or modal spectral acceleration can be selected. The structure is analyzed consecutively for the increasing IM. Based on the analysis purposes, one of the structural responses such as maximum floor accelerations or maximum inter-story displacement is selected as damage measure (DM). The structure is analyzed for increasing IM and the maximum DM is recorded until the structure fails. An IDA curve is resulted by plotting recorded DMs versus corresponding IMs. Finally, seismic behavior of structures is evaluated by the definition of various limit states and combining IDA curves and probabilistic analysis diagrams.

3. Appropriate selection of IM and DM

Intensity and density measures should be selected based on the general behavior structure and its type of service. Nowadays, peak ground acceleration, PGA, and the first mode spectral acceleration, Sa (T_1,5%) are widely used as IM. Between these IMs, the later leads to lower dispersal of IDA data sets and is preferred more than PGA.

Like IM, selection of DM depends on the target of analysis. For example, maximum roof accelerations are appropriate criteria for judgment about damage level of nonstructural components. On the other hand, maximum inter-story displacement (drift), θ_{max} (Maximum relative displacement of all stories from full time history analyses) is a suitable criterion for the global dynamic instability and higher performance levels. Therefore, in this study Sa (T_1,5%) and θ_{max} are selected as IM and DM.

4. Structural models

To study the seismic behavior of buckling restrained braced frames (BRBFs) for various performance levels and comparing their responses with special concentric braced frames (SCBFs), three steel structures with 3, 5 and 10 stories have been designed in accordance to AISC360-05 LRFD requirements. Plan of the designed structures is shown in Fig. 1.

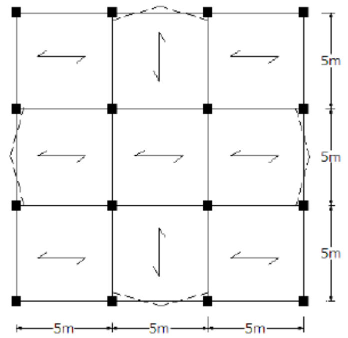

Fig. 1 Plan of the designed structures

Table 1 Uniform live and dead load

Load Type	Story	Intensity (kg/m^2)
Dead	Roof	540
	Other Stories	650
Live	Roof	150
	Other Stories	200

Table 2 Seismic parameters of BRBF and SCBF

Parameter	BRBF Values	SCBF Values
Seismic design category	D	D
R	8	6
Ω_o	2.5	2.0
C_d	5.0	5.0

*R: Response modification factor.
*Ω_o: Over strength factor.
*C_d: Deflection amplification factor.

These structures whose plan is depicted in Fig. 1, consist of three 5 meters' spans in both directions and 3.2 meters as height of stories. As it can be seen from Fig. 1, braces are placed in central spans in both directions. Chevron bracing arrangement is selected for this study. Table 1 lists applied uniform dead and live loads intensities.

In the 3 and 5 story buildings, ST37 steel is used. Based on the German standard (DIN), minimum yield stress of ST37 is equal to 2400 kg/cm^2. In designed structures, IPE and HE profiles are used for beams and columns, respectively. For the high-rise building (10 stories structure), ST52 steel with F_y=3600 kg/cm^2 is utilized. The beams are IPE profiles and columns are made of box sections.

For the special braces HSS profiles are used and BRBs are Unbounded Brace TM-Model: JIS-G3136-SN400B manufactures by Nippon steel company in Japan which according to the Japan standard its minimum yield stress is equal to F_y=2672 kg/cm^2 (López and Sabelli 2004). Moreover, information regarding the modelling of the cyclic behavior of braces has to be provided in SIE (2001).

Table 3 Cross sections for all members of Models

BRB FRAME											
10 Story Structure				5 Story Structure				3 Story Structure			
St.	Columns	Beams	Brace	St.	Columns	Beams	Brace	St.	Columns	Beams	Brace
1,2	HE 650	IPE 330	L5×5	1	HE 300	IPE 300	L5×4	1	HE 200	IPE 300	L5×3
3	HE 400	IPE 330	L5×5	2	HE 260	IPE 300	L5×4	2	HE 200	IPE 300	L5×3
4,5,6	HE 400	IPE 330	L5×4	3	HE 260	IPE 300	L5×3	3	HE 120	IPE 270	L3×3
7,8	HE 220	IPE 330	L4×4	4	HE 140	IPE 300	L5×3				
9	HE 120	IPE 330	L3×3	5	HE 140	IPE 270	L3×3				
10	HE 120	IPE 270	L3×3								
SCBF FRAME											
10 Story Structure				5 Story Structure				3 Story Structure			
St.	Columns	Beams	Brace	St.	Columns	Beams	Brace	St.	Columns	Beams	Brace
1,2	BOX 400×400×35	IPE 450	HSS 6×6×0.5	1	HE 400	IPE 330	2UNP 180	1	HE 200	IPE 300	2UNP 140
3,4	BOX 400×400×25	IPE 450	HSS 6×6×0.5	2	HE 280	IPE 330	2UNP 180	2	HE 200	IPE 300	2UNP 120
5	BOX 400×400×25	IPE 360	HSS 6×6×0.5	3	HE 260	IPE 300	2UNP 160	3	HE 120	IPE 270	2UNP 100
6	BOX 300×300×20	IPE 360	HSS 6×6×0.5	4	HE 140	IPE 300	2UNP 160				
7	BOX 300×300×20	IPE 360	HSS 5×5×0.5	5	HE 140	IPE 270	2UNP 100				
8	BOX 300×300×20	IPE 330	HSS 5×5×0.5								
9	BOX 260×260×16	IPE 330	HSS 5×5×0.375								
10	BOX 260×260×16	IPE 270	HSS 5×5×0.375								

The structures are loaded in accordance with ASCE/SEI 7-10 and ANSI/AISC 360-5 (AISC 2005, AISC 2005). The structures are assumed to be located in Panorama City, California at geographical coordinates 34.228 and 118.434. The site soil is type D and earthquake hazard is very high. According to ASCE/SEI 7-10, shear wave velocity in Soils of Type D is $180 \leq V_s \leq 360$. Therefore, this type of soil is equivalent to Type III in Iran provisions for the design of earthquake resistant buildings (2800 code). Seismic parameters of BRBF and SCBF according to ASCE/SCI 7-10 are listed in Table 2.

5. OpenSees modeling

An equivalent column model is applied to model gravitational force resistant frames. The equivalent column is connected to the braced span in a way that lateral displacement in both is equal. Due to the existence of the brace connection plates, beam to column connections are modeled as rigid in the analytical model (Uriz 2005). For beams and columns, "nonlinear beam-column" element and steel02 material is used and the sections are discretized to fibers.

These fibers are elasto-plastic and hardening behavior is also considered. Therefore, it is possible to reflect distributed plasticity in the model. The equivalent beam is modeled by "elastic column" elements and "Truss" elements are used for BRBs.

The low-cycle fatigue phenomenon is also considered by utilizing Fatigue Material model. In this model, when damage index of a fiber reach to one, the fiber stress becomes zero and it is removed (Mazzoni *et al*. 2006). One of the most important failure modes of braced structures is column buckling which can occur either in-plane or out-of-plane. All cases provided in this study modeled in-plane, and also column buckling not seen out-of-plane. For modeling the buckling column in plane, used equivalent column and Euler buckling limit load have been defined to this column.

During structure analysis, upon passing existence load from Euler buckling limit, this column will fail. Fig. 2 demonstrate the modeled structures in OpenSees program and first mode period time shows in Table 4.

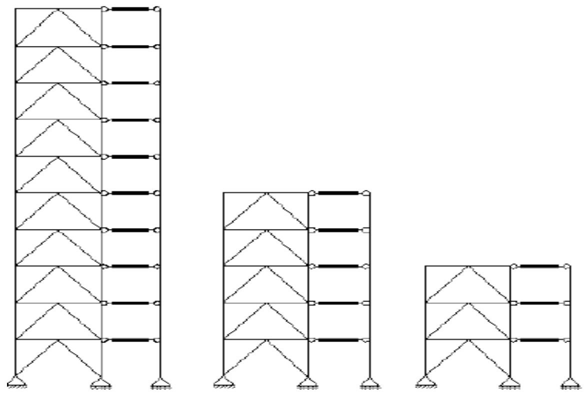

Fig. 2 The modeled BRBFs in OpenSees

Table 4 First mode period times (T_1)

	10 Story	5 Story	3 Story
First Mode Period for BRBF	1.526 s	0.697 s	0.451 s
First Mode Period for SCBF	0.728 s	0.413 s	0.313 s

6. Ground motion records

To perform IDA analysis in this study, 28 far-fault

Table 5 Selected accelerograms for IDA analysis

No.	Event	Station No.	Station Name	Soil*	R^*(km)	M^*	PGA (g)
1	Imperial Valley-06,1979	931	El Centro Array #12	D	27.94	6.53	0.138
2	Livermore-01,1980	57187	San Ramon - Eastman Kodak	D	----	5.80	0.107
3	Loma Prieta,1989	57382	Gilroy Array #4	D	23.81	6.93	0.304
4	Loma Prieta,1989	1601	Palo Alto - SLAC Lab	D	30.62	6.93	0.228
5	Morgan Hill,1984	57425	Gilroy Array #7	D	22.06	6.39	0.144
6	N. Palm Springs,1986	5070	North Palm Springs	D	---	6.06	0.590
7	Northridge-01,1994	90034	LA - Fletcher Dr	D	25.66	6.69	0.207
8	San Fernando,1971,1971	24303	LA - Hollywood Stor FF	D	22.77	6.61	0.210
9	Superstition Hills-02,1987	11369	Westmorland Fire Sta	D	23.03	6.54	0.210
10	Whittier Narrows-01,1987	90078	Compton - Castlegate St	D	28.32	5.99	0.331
11	Parkfield,1966	1015	Cholame - Shandon Array #8	D	22.90	6.19	0.264
12	Morgan Hill,1984	47380	Gilroy Array #2	D	23.68	6.19	0.187
13	Westmorland,1981	5060	Brawley Airport	D	25.28	5.90	0.157
14	Landers,1992	22074	Yermo Fire Station	D	23.62	7.28	0.223
15	Northridge-01,1994	90094	Bell Gardens - Jaboneria	D	41.27	6.69	0.079
16	Northridge-01,1994	90099	Arcadia - Arcadia Av	D	39.41	6.69	0.095
17	Coyote Lake,1979	57191	Halls Valley	D	33.69	5.74	0.042
18	Cape Mendocino,1992	89156	Petrolia	D	---	7.01	0.624
19	Borrego Mtn,1968	117	El Centro Array #9	D	45.12	6.63	0.088
20	Landers,1992	12025	Palm Springs Airport	D	36.15	7.28	0.093
21	Landers,1992	12026	Indio - Coachella Canal	D	54.25	7.28	0.106
22	Whittier Narrows-01,1987	90003	Northridge - 17645 Saticoy St	D	38.04	5.99	0.144
23	N. Palm Springs,1986	12331	Hemet Fire Station	D	34.48	6.06	0.128
24	N. Palm Springs,1986	12202	San Jacinto - Valley Cemetary	D	30.07	6.06	0.057
25	Big Bear-01,1992	23542	San Bernardino - E & Hospitality	D	---	6.46	0.090
26	Coalinga-01,1983	36227	Parkfield - Cholame 5W	D	47.88	6.36	0.136
27	Coalinga-01,1983	36226	Parkfield - Cholame 8W	D	50.98	6.36	0.093
28	El Alamo,1956	117	El Centro Array #9	D	---	6.80	0.046

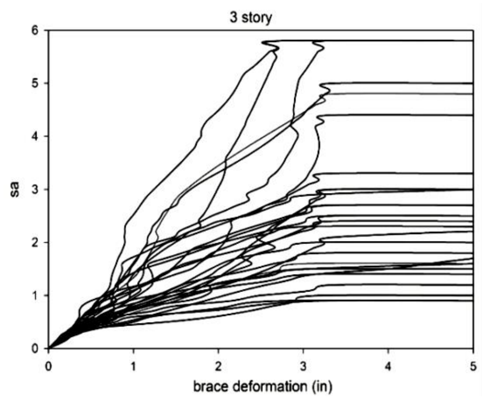

Fig. 3 IDA curves for BRBF-3 story structure

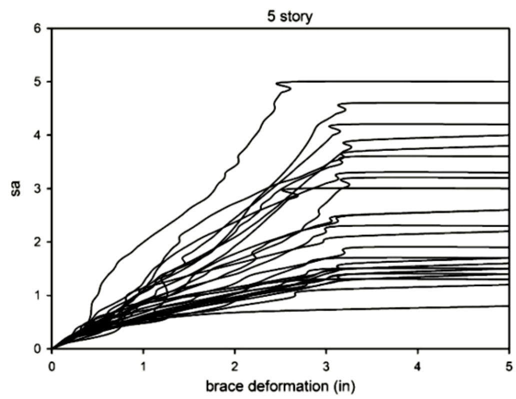

Fig. 4 IDA curves for BRBF-5 story structure

ground motion records on type D soil (ASCE/SEI 7-10) are selected. These motions and their characteristics are listed in Table 5.

7. Results and discussions

7.1 IDA curves and limit states

The braces deformations and the first mode spectral acceleration Sa (T_1, 5%) are selected as DM and IM for the IDA analysis. IDA analysis are performed on the three structures for both systems (i.e., BRBF and SCBF) and the obtained IDA curves are presented in Figs. 3, 4, 5, 6, 7 and 8.

The previous figures, demonstrate IDA curves for all the 28 records and therefore considerable dispersion exist for multiple ground motions. To reach compact responses and evaluate general behavior of the structures, three statistical curves corresponding to 16%, 50% and 84% are extracted from the presented IDA curves and depicted in Figs. 9, 10, 11, 12, 13 and 14.

According to FEMA 350, the immediate occupancy

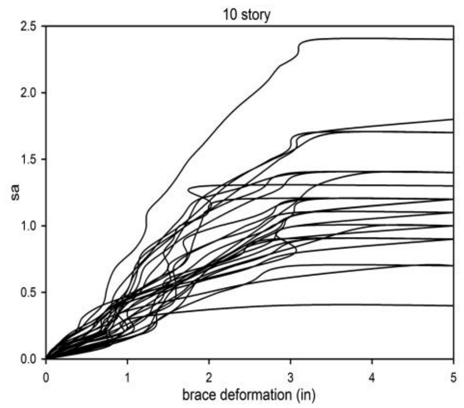

Fig. 5 IDA curves for BRBF-10 story structure

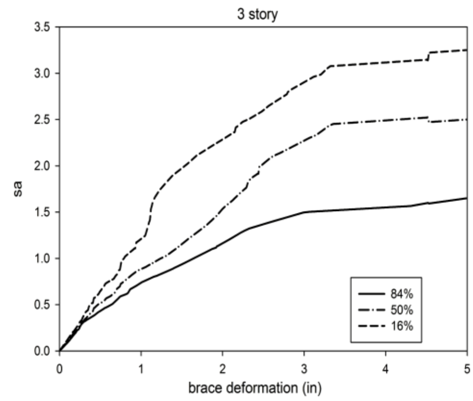

Fig. 9 Compact IDA curves for BRBF-3 story structure

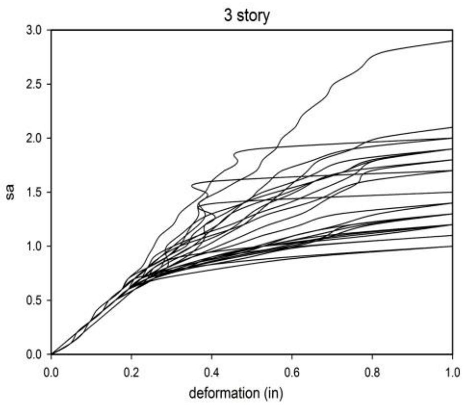

Fig. 6 IDA curves for SCBF-3 story structure

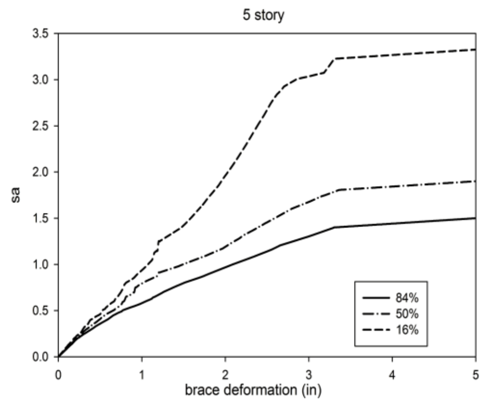

Fig. 10 Compact IDA curves for BRBF-5 story structure

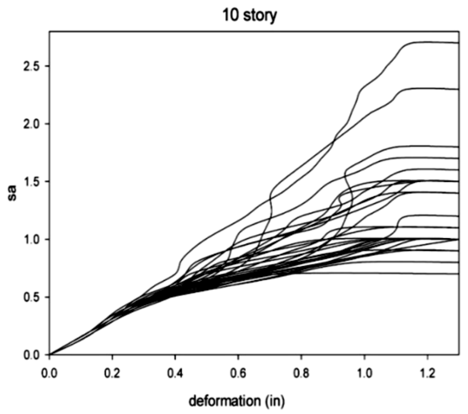

Fig. 7 IDA curves for SCBF-5 story structure

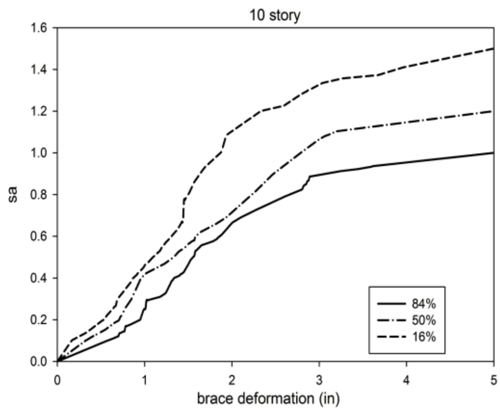

Fig. 11 Compact IDA curves for BRBF-10 story structure

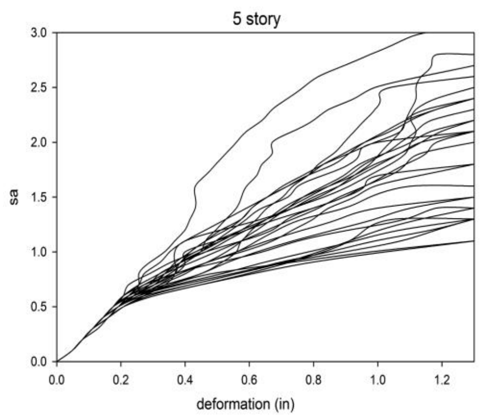

Fig. 8 IDA curves for SCBF-3 story structure

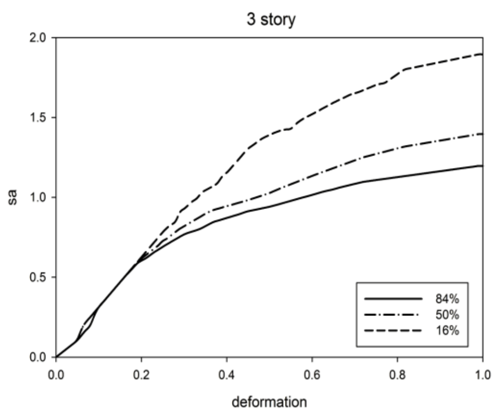

Fig. 12 Compact IDA curves for SCBF-3 story structure

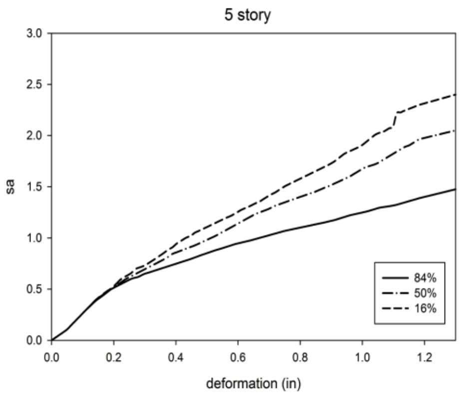

Fig. 13 Compact IDA curves for SCBF-5 story structure

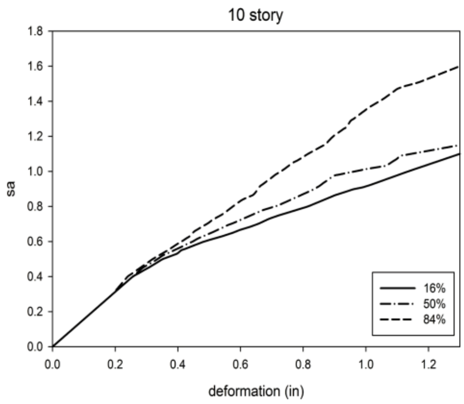

Fig. 14 Compact IDA curves for SCBF-10 story structure

Table 6 Acceptance criteria for braces

	IO	LS	CP
Brace in Compression	$0.25\Delta c$	$5\Delta c$	$7\Delta c$
Brace in Tension	$0.25\Delta c$	$7\Delta c$	$9\Delta c$

Table 7 *Sa* (T_1, 5%) and Δ_{max} of the 3 story structures at different performance levels

	BRBF Structure					
	Sa (T_1,5%) g			Δ_{max} (in)		
	IO	LS	CP	IO	LS	CP
16%	0.424	2.32	2.65	0.323	2.067	2.584
50%	0.354	1.58	2.08	0.323	2.067	2.584
84%	0.330	1.19	1.38	0.323	2.067	2.584
	SCBF Structure					
	Sa (T_1,5%) g			Δ_{max} (in)		
	IO	LS	CP	IO	LS	CP
16%	0.034	0.825	1.140	0.0164	0.262	0.394
50%	0.034	0.760	0.940	0.0164	0.262	0.394
84%	0.034	0.720	0.865	0.0164	0.262	0.394

(IO), life safety (LS) and collapse prevention (CP) performance levels are selected to define limit states. For the BRBs acceptance criteria in tension and for the SCBs acceptance criteria in compression are used. Acceptance criteria for brace in compression and tension present in following Table 6. The following Tables 7, 8, 9 present maximum deformation of braces according to acceptance

Table 8 *Sa* (T_1, 5%) and Δ_{max} of the 5 story structures at different performance levels

	BRBF Structure					
	Sa (T_1,5%) g			Δ_{max} (in)		
	IO	LS	CP	IO	LS	CP
16%	0.328	2.030	2.800	0.323	2.067	2.584
50%	0.285	1.220	1.504	0.323	2.067	2.584
84%	0.250	0.989	1.170	0.323	2.067	2.584
	SCBF Structure					
	Sa (T_1,5%) g			Δ_{max} (in)		
	IO	LS	CP	IO	LS	CP
16%	0.033	0.672	0.930	0.0164	0.262	0.394
50%	0.033	0.633	0.856	0.0164	0.262	0.394
84%	0.033	0.607	0.744	0.0164	0.262	0.394

Table 9 *Sa* (T_1, 5%) and Δ_{max} of the 10 story structures at different performance levels

	BRBF Structure					
	Sa (T_1,5%) g			Δ_{max} (in)		
	IO	LS	CP	IO	LS	CP
16%	0.136	1.123	1.220	0.323	2.067	2.584
50%	0.099	0.740	0.930	0.323	2.067	2.584
84%	0.057	0.681	0.780	0.323	2.067	2.584
	SCBF Structure					
	Sa (T_1,5%) g			Δ_{max} (in)		
	IO	LS	CP	IO	LS	CP
16%	0.365	0.561	0.770	0.0236	0.377	0.566
50%	0.365	0.540	0.690	0.0236	0.377	0.566
84%	0.365	0.510	0.640	0.0236	0.377	0.566

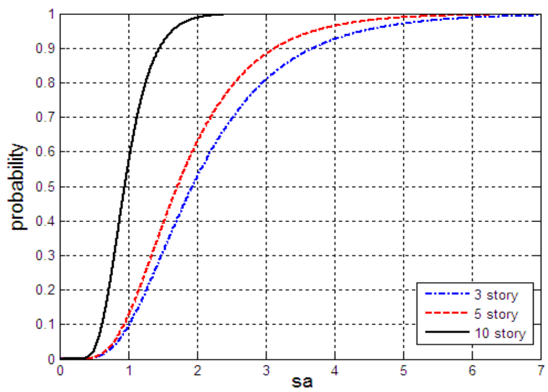

Fig. 15 Fragility curves at CP level for BRBF system

criteria from Table 6 and the first mode spectral acceleration for the structures at the evaluated performance levels. It must be noted that *Sa* (T_1, 5%) corresponding to CP level is the maximum seismic capacity of the structures.

7.2 Evaluation of CP level exceedance probability

The probability of being or exceeding a damage level is modeled with a cumulative distribution Eq. (1) or cumulative lognormal distribution. Such a distribution is expressed in Eq. (2) (Stergiou and Kiremidjian 2008).

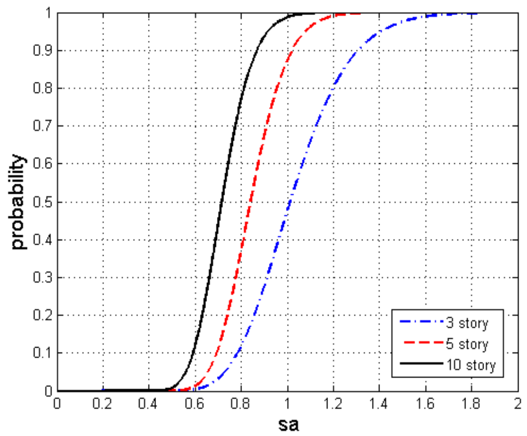

Fig. 16 Fragility curves at CP level for SCBF system

$$P[CP/x_i]=\phi\left[\frac{xi-\bar{x}}{\beta}\right] \quad (1)$$

$$P[CP/x_i]=\phi\left[\frac{\ln x_i-\ln\bar{x}}{\beta}\right] \quad (2)$$

Where;

P, is the probability of a performance level (hear collapse prevention performance level or CP),

x_i, is one the earthquake parameters such as spectral acceleration (*Sa*),

$\bar{x}$, is the average relative displacement at desired spectral acceleration,

β, is the standard deviation and,

ϕ, is the log-normal distribution function.

To plot these functions only mean and standard deviation are needed (Council 2000). The fragility curves for the CP level are depicted in Figs. 15 and 16. It is evident that the exceedance probability for the CP level in SCBF system is higher than BRBF. In other words, BRBF is a more reliable earthquake resisting system.

8. Conclusions

In this paper, seismic behavior of three BRBF and SCBF structure with 3, 5 and 10 stories were evaluated using IDA analysis. To perform IDA analysis in this study, 28 far-fault ground motion records on type D soil (ASCE/SEI 7-10) are selected. Fragility curves developed in this study from the obtained results and the median curve.

The following conclusions based on the analyzed structures, and it is obvious that for a general conclusion, more analysis are needed. According to the IDA curves which are obtained for both the BRBF and the SCBF structures, the SCBF experienced nonlinear behavior sooner than the BRBF structure. In addition, the buckling restrained braces undergoes larger displacements in the elastic domain.

The obtained fragility curves show that the 3 and 5 story structures experiences CP level at relatively equal Sa, while for the IO performance level, there are more differences between these values. It is also evident that for the high-rise structures (the five and ten-story buildings in this study), the probability of exceeding the CP performance level is higher than this probability in low-rise structures (the three-story buildings in this study).

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Reference

AISC (2005), AISC 360-05, Specification for Structural Steel Buildings, American Institute of Steel Construction, Chicago, IL.

AISC (2005), Specification for Structural Steel Buildings, American Institute of Steel Construction, Inc., Chicago, IL.

Alhajri, T., Tahir, M., Azimi, M., Mirza, J., Lawan, M., Alenezi, K. and Ragaee, M. (2016), “Behavior of pre-cast U-shaped composite beam integrating cold-formed steel with ferro-cement slab”, *Thin Wall. Struct.*, **102**, 18-29.

Azimi, M., Adnan, A.B., Tahir, M.M., Sam, A.R.B.M. and Razak, S.M.B.S.A. (2015), “Seismic performance of ductility classes medium RC beam-column connections with continuous rectangular spiral transverse reinforcements”, *Lat. Am. J. Solid. Struct.*, **12**(4), 787-807.

Azimi, M., Bagherpourhamedani, A., Tahir, M.M., Sam, A.R.B.M. and Ma, C.K. (2016), “Evaluation of new spiral shear reinforcement pattern for reinforced concrete joints subjected to cyclic loading”, *Adv. Struct. Eng.*, **19**(5), 730-745.

Bazzaz M., Andalib, Z., Kafi, M.A. and Kheyroddin, A. (2015), ”Evaluating the performance of OBS-CO in steel frames under monotonic load”, *Earthq. Struct.*, **8**(3), 697-710..

Bazzaz, M., Andalib, Z., Kheyroddin, A. and Kafi, M.A. (2015), ”Numerical comparison of the seismic performance of steel rings in off-centre bracing system and diagonal bracing system”, *Steel Compos. Struct.*, **19**(4), 917-937.

Bazzurro, P. and Cornell, C.A. (1994), “Seismic hazard analysis of nonlinear structures. II: Applications”, *J. Struct. Eng*., **120**(11), 3345-3365.

Council, B.S.S. (2000), “Prestandard and commentary for the seismic rehabilitation of buildings”, Report FEMA-356, Washington, DC.

Dimitrios, V. and Michalis, F. (2009), “Incremental dynamic analysis for estimating seismic performance sensitivity and uncertainty”, *Earthq. Eng. Struct. Dyn.*, **39**(2), 141-163.

Ghowsi, A.F. and Sahoo, D.R. (2015), “Fragility assessment of buckling-restrained braced frames under near-field earthquakes”, *Steel Compos. Struct.*, **19**(1), 173-190.

Giugliano, M.T., Longo, A., Montuori, R. and Piluso, V. (2010), “Plastic design of CB-frames with reduced section solution for bracing members”, *J. Constr. Steel Res.*, **66**(5), 611-621.

Giugliano, M.T., Longo, A., Montuori, R. and Piluso, V. (2011), “Seismic reliability of traditional and innovative concentrically braced frames”, *Earthq. Eng. Struct. Dyn.*, **40**(13), 1455-1474.

Hakim, S.J.S., Noorzaei, J., Jaafar, M., Jameel, M. and Mohammadhassani, M. (2011), “Application of artificial neural networks to predict compressive strength of high strength concrete”, *Int. J. Phys. Sci*., **6**(5), 975-981.

Hsu, H., Juang, J. and Chou, C. (2011), “Experimental evaluation on the seismic performance of steel knee braced frame structures with energy dissipation mechanism”, *Steel Compos.*

Struct., **11**(1), 77-91.
Iran National Provisions for Design of Earthquake Resistant Buildings (2004), 2800 Code, 3rd Edition, Building and Housing Research Center, September.
Isolation Engineering, for Nippon Steel Corporation to Ove Arup & Partners California, Ltd. and Office of Statewide Health Planning and Development.
Khoo, H.H., Tsai, K.C., Tsai, C.Y., and Wang, K.J. (2016), "Bidirectional substructure pseudo-dynamic tests and analysis of a full-scale two-story buckling-restrained braced frame", *Earthq. Eng. Struct. Dyn.*, **45**(7), 1085-1107
Kim, J., Park, J. and Kim, S.D. (2009), "Seismic behavior factors of buckling-restrained braced frames", *Struct. Eng. Mech.*, **33**(3), 261-284.
Kim, S.H. and Choi, S.M. (2015), "Structural behavior of inverted V-braced frames reinforced with non-welded buckling restrained braces", *Steel Compos. Struct.*, **19**(6), 1581-1598.
Longo, A., Montuori, R. and Piluso, V. (2008), "Influence of design criteria on seismic reliability of X-braced frames", *J. Earthq. Eng.*, **12**(3), 406-431.
Longo, A., Montuori, R. and Piluso, V. (2008), "Plastic design of seismic resistant V-braced frames", *J. Earthq. Eng.*, **12**(8), 1246-1266.
Longo, A., Montuori, R. and Piluso, V. (2008), "Failure mode control of X-braced frames under seismic actions", *J. Earthq. Eng.*, **12**(5), 728-759.
Longo, A., Montuori, R. and Piluso, V. (2009), "Seismic reliability of V-braced frames: Influence of design methodologies", *Earthq. Eng. Struct. Dyn.*, **38**(14), 1587-1608.
Longo, A., Montuori, R. and Piluso, V. (2015), "Seismic design of chevron braces cupled with MRF fail safe systems", *Earthq. Struct.*, **8**(5), 1215-1239.
Longo, A., Montuori, R. and Piluso, V. (2016), "Moment frames-concentrically braced frames dual systems: analysis of different design criteria", *Struct. Infrastr. Eng.*, **12**(1), 122-141
López, W.A. and Sabelli, R. (2004), "Seismic design of buckling-restrained braced frames", *Steel Tips*, 78.
Luco, N. and Cornell, C.A. (1998), "Effects of random connection fractures on the demands and reliability for a 3-story pre-Northridge SMRF structure", *Proceedings of the 6th US National Conference on Earthquake Engineering*.
Luco, N. and Cornell, C.A. (2000), "Effects of connection fractures on SMRF seismic drift demands", *J. Struct. Eng.*, **126**(1), 127-136.
Ma, C.K., Awang, A.Z., Garcia, R., Omar, W., Pilakoutas, K. and Azimi, M. (2016), *Nominal Curvature Design of Circular HSC Columns Confined with Post-tensioned Steel Straps. Structures*, Elsevier.
Ma, C.K., Awang, A.Z., Omar, W., Liang, M., Jaw, S.W. and Azimi, M. (2016), "Flexural capacity enhancement of rectangular high-strength concrete columns confined with post-tensioned steel straps: Experimental investigation and analytical modeling", *Struct. Concrete*, **17**(4), 668-676.
MATLAB (2004), The Language of Technical Computing, Version 7 (R14).
Mazzoni, S., McKenna, F., Scott, M.H. and Fenves, G.L. (2006), "OpenSees command language manual", Pacific Earthquake Engineering Research (PEER) Center.
McKenna, F., Fenves, G., Jeremic, B. and Scott, M. (2008), Open System for Earthquake Engineering Simulation, http://opensees.berkeley.edu.
Mehanny, S.S.F. and Deierlein, G.G. (1999), "Modeling and assessment of seismic performance of composite frames with reinforced concrete columns and steel beams", Stanford University.
Mohammadhassani, M., Jumaat, M.Z. and Jameel, M. (2012), "Experimental investigation to compare the modulus of rupture in high strength self compacting concrete deep beams and high strength concrete normal beams", *Constr. Build. Mater.*, **30**, 265-273.
Mohammadhassani, M., Nezamabadi-Pour, H., Jumaat, M., Jameel, M., Hakim, S. and Zargar, M. (2013), "Application of the ANFIS model in deflection prediction of concrete deep beam", *Struct. Eng. Mech.*, **45**(3), 319-332.
Mohammadhassani, M., Suhatril, M., Shariati, M. and Ghanbari, F. (2014), "Ductility and strength assessment of HSC beams with varying of tensile reinforcement ratios", *Struct. Eng. Mech.*, **48**(6), 833-848.
Nassar, A.A. and Krawinkler, H. (1991), "Seismic demands for SDOF and MDOF systems", John A. Blume Earthquake Engineering Center, Department of Civil Engineering, Stanford University.
Niknam, A., Ahmadi, H. and Mahdavi, N. (2007), "Using nonlinear incremental dynamic analysis (IDA) to study seismic behavior of structures", *2nd National Conference on Retrofitting and Rehabilitation of Structures*, Iran. (in Persian)
Popov, E.P. and Pister, K.S. (1980), *Structural Engineering and Structural Mechanics: A Volume Honoring Egor P. Popov*, Prentice Hall.
Saxey, B. and Daniels, M. (2014), "Characterization of overstrength factors for buckling restrained braces", *Australasian Structural Engineering (ASEC) Conference*, Auckland, New Zealand.
SIE (2001), "Cyclic tests of nippon steel corporation unbonded braces", Unpublished Teport by Seismic.
Stergiou, E. and Kiremidjian, A.S. (2008), "Treatment of uncertainties in seismic-risk analysis of transportation systems", PEER Report 2008/02.
Uriz, P. (2005), "Towards earthquake resistant design of concentrically braced steel structure", University of California, Berkeley.
Vamvatsikos, D. and Cornell, C.A. (2002), "Incremental dynamic analysis", *Earthq. Eng. Struct. Dyn.*, **31**(3), 491-514.
Yun, S.Y., Hamburger, R.O., Cornell, C.A. and Foutch, D.A. (2002), "Seismic performance evaluation for steel moment frames", *J. Struct. Eng.*, **128**(4), 534-545.

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