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Reliability analysis of braced frames subjected to near field ground motions

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Abstract. Near field ground motions have caused several structural damages in recent decades. As a result, seismic codes are being updated with related requirements. In this paper a comparative study on the seismic behavior of concentrically braced frames (CBFs) designed based on different seismic codes is performed. Reliability of various frames with different heights and bracing types are analyzed based on the results of "Incremental Dynamic Analysis" (IDA) under near field ground motions. Fragility curves corresponding to IO (Immediate Occupancy) and CP (Collapse Prevention) limit states are extracted based on IDA curves. Results imply that, frames designed based on the near field seismic design criteria of UBC-97 are more reliable under near field ground motions and their failure probability is less comparing to others.

Keywords: incremental dynamic analysis; reliability; limit states; fragility

1. Introduction

Not long ago, seismic codes were only focusing on far field earthquakes and many of engineers were unaware of near field ground motions and their characteristics and effects. During recent decades, near field ground motions caused serious damages in the structures near seismic sources. These pulse-like waves, with their enormous energy contents, have basically different effects on structures comparing to far field ground motions. After Northridge (1994) and Kobe (1995) earthquakes, seismic reliability of existing and damaged structures was studied. Damages, failures and sever fractures indicated that there is something wrong about near-field ground motions in seismic codes. Thus, basic changes were necessary as happened in AISC (seismic provision 2005), FEMA-356 and FEMA-273. Several researchers have investigated seismic behavior of structures under near field ground motions. Some have used newer methods of assessing structural reliability using incremental dynamic analysis and fragility curves.

Luco (2002) studied the entity of ground motions and presented criteria for dividing them into three near, middle and far field categories. Chao *et al.* (2008) studied the effect of near-field ground motions on the inelastic demand of structures. Jalayer *et al.* (2004) studied the seismic

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reliability of steel structures and presented an introduction to the probabilistic seismic design of structures under near-field earthquakes. Haeri *et al.* (2013) assessed seismic reliability of 6 and 20 story reinforced concrete buildings with different lateral load resisting systems using incremental dynamic analysis under near-field and far-field ground motions. They calculated mean annual frequency of exceeding IO and CP limit states. Asgarian *et al.* (2012) assessed seismic performance of different kinds of steel moment resisting structures under near-field and far-field ground motions through incremental dynamic analysis. They considered PGA, $S_{a,}(T_1,5\%)$, $S_{a,}(N,5\%)$, and $S_{v,}(T_1,5\%)$ as intensity measures. Results showed that $S_{v,}(T_1,5\%)$ and $S_{a,}(N,5\%)$ are suitable as intensity measure of near filed ground motions for the studied structures and also special moment resisting frames have better performance than intermediate ones. Islam *et al.* (2012) proposed an analytical method of achieving fragility curves for reinforced concrete pier bridges. They also used these fragility curves for retrofitting and pre-earthquake planning. Rehan *et al.* (2011) presented a simplified fragility analysis for reinforced concrete frames on different soil types through pushover analysis. Their study showed that the probability of failure for frames on soft soil is more than the ones on stiffer soil. Billah *et al.* (2012) evaluated seismic vulnerability of retrofitted multi-column bridge bents. They presented fragility curves for evaluating relative performance of different retrofitting methods against near-field and far-field ground motions.

The aim of this study is to evaluate the performance of CBFs designed based on UBC-97 near-field seismic criteria and some other seismic codes which do not take into account these requirements. The Iranian seismic code (standard No.2800) which presents the same method and requirements as UBC-94, without considering near-field criteria is used in this comparative study. Asgarian and Jalaeefar (2011) used IDA for evaluating seismic behavior of CBFs under far field ground motions. Reliability and fragility analysis of the frames using near-field earthquakes is the main distinction between the present study and their research.

Several methods are available to perform such a study. Among them is the incremental dynamic analysis (IDA), an emerging analysis method in which capacity and limit states can be predicted using a series of nonlinear dynamic analyses. Ground motion records are scaled in multiple steps to perform such analyses and to trace the structural behavior from elastic range to total failure while evaluating its demand, capacity and limit states.

Using the results of incremental dynamic analysis, annual probability of exceeding limit states is calculated and fragility curves which are important in seismic damage assessment of buildings are developed and compared to each other.

2. About the Iranian seismic code

The Iranian seismic code (standard No. 2800) presents the same analysis method and requirements as UBC-94, without considering near-field criteria. The basic concept of the code is to provide requirements that:

- Cause buildings to remain stable in severe earthquakes and hence minimize mortality.
- Decrease structural damages due to earthquakes with low and moderate intensities in common buildings.
- Prevent structural damage due to earthquakes with low and moderate intensities in important buildings.

Severe earthquakes (design earthquakes) are the ones with 475 years return period (10% probability of occurrence in 50 years). Earthquakes with low and moderate intensities are the ones with 2475 years return period (99.5% probability of occurrence in 50 years).

Seismic design of the structure can be performed using following methods:

- Equivalent static analysis method.
- Dynamic analysis method.

Using the equivalent static method is allowed just for regular structures with less than 50 m of height or irregular ones with less than five stories or 18 m of height. Others must be designed using dynamic analysis method. The base shear in the equivalent static method is linearly distributed in the structure's height according to its first mode of vibration.

3. Design of the structures

To evaluate and compare the performance of CBFs, frames of 5, 8 and 12 stories with X and chevron (inverted V) braces are designed in a site of high seismic hazard (Tehran, Iran). These structures are assumed to be located on soil type B according to the Iranian seismic code classification (average shear wave velocity of 375-750 m/s). All of the structures are similar in plan (Fig. 1) and story heights. Vertical bracings are placed in middle bays in each edge of the plan, making the structures symmetric in plan. Rigid roof diaphragm is assumed as in usual structures.

Gravity loads are assumed to be similar to common residential buildings. Earthquake loads for different models are calculated based on the 1st, 2nd or 3rd editions of "Iranian Seismic Code" (Standard no. 2800) or UBC-97 near-field seismic design criteria. Also AISC-ASD89 or AISC-Seismic provision (2005) requirements are used for steel design of different structures. A total of 20 steel braced frames with different heights, bracing types and different design codes are considered in the present study. Details of the frames and their design considerations are summarized in Table 1.

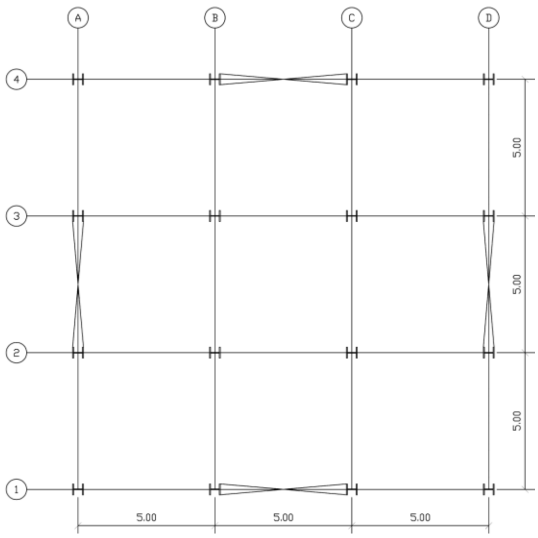

Fig. 1 Typical plan of the structures and brace arrangement

4. F.E modeling of the structures

OpenSees finite element software (Mazzoni 1999), generated and developed in west US universities is used for modeling of the structures. Nonlinear dynamic analyses are performed for a two dimensional frame of each structure.

To model the St-37 steel behavior, ‘steel 02’ bilinear stress–strain curve is used from the library of materials introduced in OpenSees. Mechanical properties of ST-37 steel are presented in Table 2. A force-based beam–column element, consisted of fibers, is used for beams, columns and braces. In order to providing geometric nonlinearity conditions for columns and braces, an L/1000 initial imperfection is assumed at their mid-span.

Both lumped and distributed plasticity can be used for modeling material nonlinearity. The latter is used in the present study.

To consider geometric stiffness, “Corotational” method provided in Opensees software is used. Equilibrium equations are solved based on the deformed shaped coordinate system as in modified

Table 1 Model definition and design codes

Model Name	Design code	Number of stories	Bracing type	1st mode vibration period (sec)
X5-1	standard 2800 - 1st edition	5	X	0.84
V5-1	Standard 2800 - 1st edition	5	Inverted V	0.79
X5-2	standard 2800 - 2nd edition	5	X	0.66
X8-2	standard 2800 - 2nd edition	8	X	1.06
X12-2	standard 2800 - 2nd edition	12	X	1.68
V5-2	standard 2800 - 2nd edition	5	Inverted V	0.6
V8-2	standard 2800 - 2nd edition	8	Inverted V	1.01
V12-2	standard 2800 - 2nd edition	12	Inverted V	1.64
X5-3	standard 2800 - 3rd edition	5	X	0.64
V5-3	standard 2800 - 3rd edition	5	Inverted V	0.59
X5-3A	standard 2800 - 3rd edition + AISC	5	X	0.64
X8-3A	standard 2800 - 3rd edition + AISC	8	X	1.01
X12-3A	standard 2800 - 3rd edition + AISC	12	X	1.6
V5-3A	standard 2800 - 3rd edition + AISC	5	Inverted V	0.59
V8-3A	standard 2800 - 3rd edition + AISC	8	Inverted V	0.95
V12-3A	standard 2800 - 3rd edition + AISC	12	Inverted V	1.48
X5-U	UBC-97 (near field design criteria)	5	X	0.62
X12-U	UBC-97 (near field design criteria)	12	X	1.55
V5-U	UBC-97 (near field design criteria)	5	Inverted V	0.55
V12-U	UBC-97 (near field design criteria)	12	Inverted V	1.47

Table 2 Model definition and design codes

Ultimate failure stress (Fu)	370 MPa
Yield stress	240 MPa
Ultimate failure strain	28%
Modulus of elasticity	2104 GPa
Poisson's ratio	0.3
Coefficient of thermal expansion	12×10^{-6} (1/C)

Lagrangian method. Besides assuming only the degrees of freedom related to deformations, and not rigid movements, a basic system of coordinates is generated. Using such a system of coordinates, equilibrium equations are solved while stiffness matrices are updated in each step of solution.

5. F.E modeling of the structures

Incremental Dynamic Analysis (IDA) is an emerging analysis method through which capacity and limit states can be predicted via a series of nonlinear dynamic analyses. Ground motion records are scaled in multiple steps to perform such analyses and to trace the structural behavior from elastic range to total failure. Also demands, capacities and limit states are evaluated based on the results.

For performing IDA, a suite of ground motion records should be selected. Previous studies (Shome et al. 1999) have shown that 10 to 20 records are usually enough to provide sufficient accuracy in the assessment of seismic demands of mid-rise buildings. Consequently, a set of fifteen ground motion records, listed in Table 3 are selected with relatively large magnitudes of 6.5-7.1 and are closer than 10 km to the fault rupture, representing near-field earthquakes.

One of the most important issues in an IDA Procedure is selecting suitable "Intensity Measure", (IM), and "Damage Measure", (DM).

In this paper, four intensity measures are used namely:

- Peak Ground Acceleration (PGA)
- first-mode spectral acceleration with 5% damping ($S_a(T_1,5\%)$)
- first-mode spectral velocity with 5% damping ($S_v(T_1,5\%)$)
- ($S_a(N,5\%)$) with 5% damping, which is defined as:

$$S_a(N,5\%) = \sqrt[n]{S_a(T_1,5\%)S_a(T_2,5\%)\ S_a(T_3,5\%)\ \ldots S_a(T_n,5\%)} \tag{1}$$

Also selecting a DM can be application-specific. In this study, the maximum inter-story drift ratio (θ_{max}) which is commonly used as a limit-state criterion is selected. Each record is scaled in multiple steps to cover the entire range of structural response, from elastic range to yielding and collapse. Sample IDA curves for some of the frames are shown in Figs. 2-5.

Based on the results, $S_a(T_1,5\%)$ causes the least dispersion in the results among the four intensity measures used. Also for the selected structures the effect of higher modes in the behavior

Table 3 Near-field ground motions used for IDA

No	Event	year	Station	Distance (Km)
1	Imperial Valley	1979	EC County center FF	7.6
2	Imperial Valley	1979	EC Meloland Overpass FF	0.5
3	Imperial Valley	1979	El Centro Array #8	3.8
4	Imperial Valley	1979	El Centro Array #4	4.2
5	Imperial Valley	1979	942 El Centro Array #6	1
6	Imperial Valley	1979	952 El Centro Array #5	1
7	Northridge	1994	Newhall -west pico Conyon Rd	7.1
8	Northridge	1994	75 Sylmar -Converter Sta. east	6.1
9	Northridge	1994	74 Sylmar -Converter Sta.	6.2
10	Northridge	1994	24087 Arleta-Nordhoff fire Sta.	9.2
11	Park Filed	1966	1013 Cholame#2	0.1
12	Northridge	1994	Rinaldi Receiving Station	7.1
13	Duzce,Turkey	1999	Duzce	8.2
14	Loma Prieta	1989	57007 Coralitos	5.1
15	Coyote Lake	1979	Gilroy Array#6	3.1

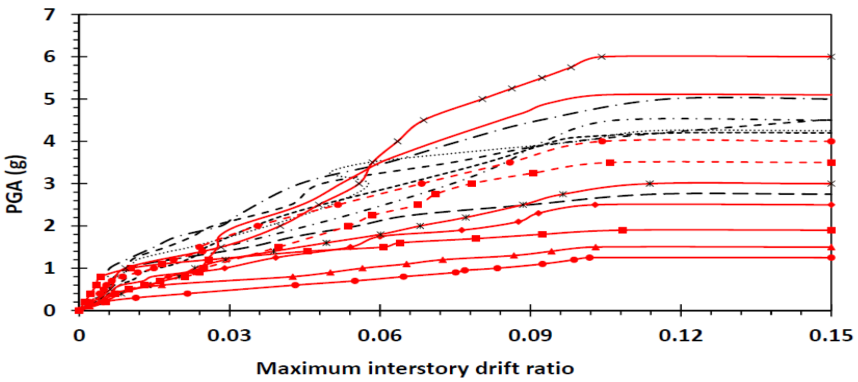

Fig. 2 IDA curves for X5-3A, using PGA intensity measure

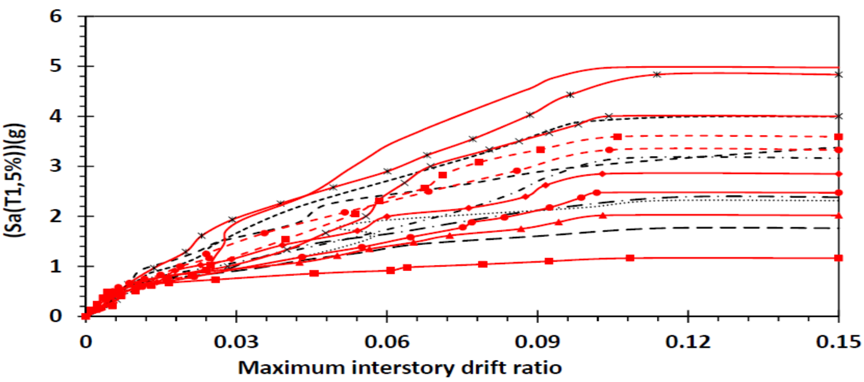

Fig. 3 IDA curves for X5-3A, using $S_a(T_1,5\%)$ intensity measure

of the structures under near-field earthquakes is negligible and IDA curves with $S_a(T_1,5\%)$ IM are close to the ones with $S_a(N,5\%)$ IM (N=3). Thus $S_a(T_1,5\%)$ is selected for rest of the study.

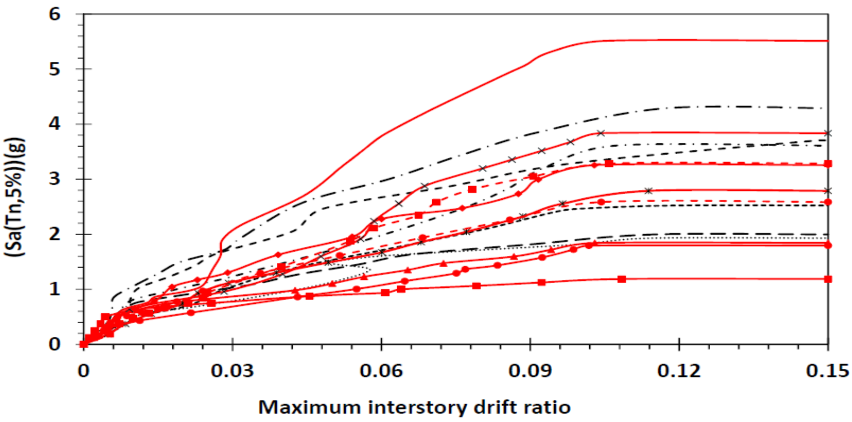

Fig. 4 IDA curves for X5-3A, using PGA intensity measure

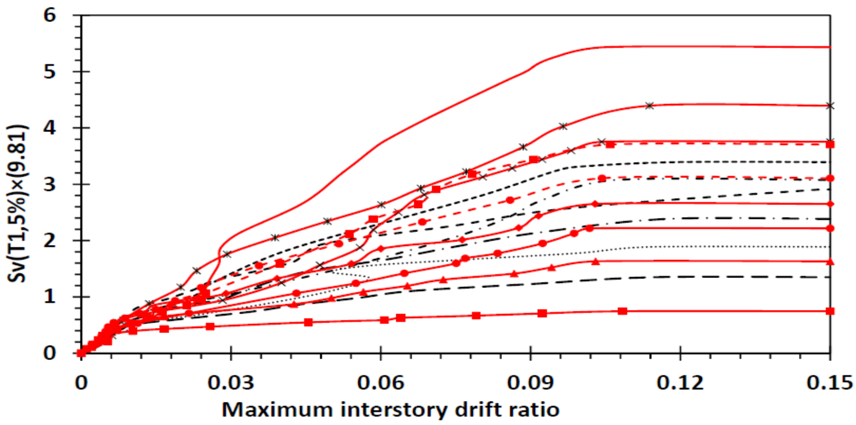

Fig. 5 IDA curves for X5-3A, using $S_a(T_1,5\%)$ intensity measure

6. Performance evaluation

Performance evaluation procedure in this research involves the assessment of structural confidence levels corresponding to desired performance objectives outlined in FEMA.

6.1 Introduction of limit states

For evaluating the performance of the frames, limit states should be defined. Two common limit state definitions are presented in FEMA namely: “Immediate Occupancy”, (IO), and “Collapse Prevention”, (CP). The “IO” limit-state occurs at 2% maximum inter-story drift. For defining “CP” limit, two criteria are proposed, which ever happens first, defines the CP limit state in IM terms. The first is the local slope of the IDA curve to be less than 20% of the initial elastic slope. The second is the maximum inter-story drift to reach 10%.

6.2 Interpretation of the results

As seen, IDA curves cover a wide range of behaviors, presenting large record-to-record dispersion. Thus, it is essential to summarize this mass of data and to quantify their randomness using statistical central values and dispersion measures. Consequently the 16%, 50% and 84% fractiles of DM and IM can be calculated for each IDA curve (Vamvatsikos 2002). The 50% fractile (median), is used to compare the behavior of the different frames in the present study.

Median IDA curves of sample 5, 8 and 12 story frames are shown in Figs. 6 and 7. Also Figs. 8-13 show median IDA curves for CBFs with similar heights and different seismic design codes. Also Table 4 summarizes the structural capacities for both IO and CP limit states.

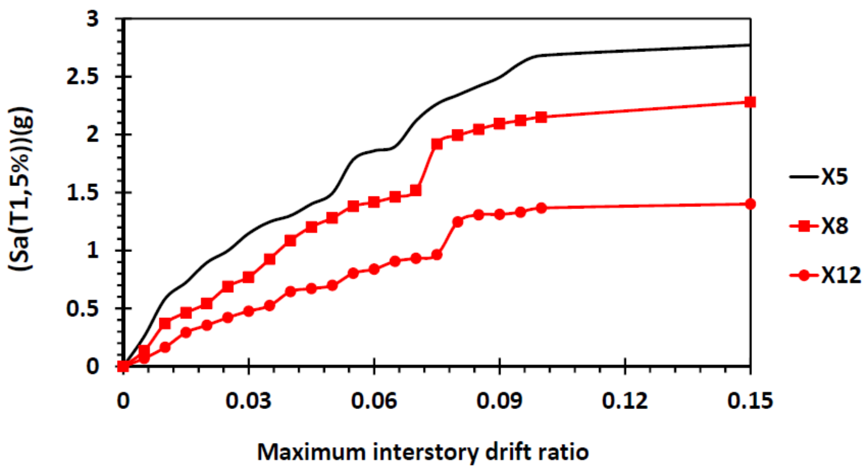

Fig. 6 Median IDA curves X5-2, X8-2 and X12-2 frames

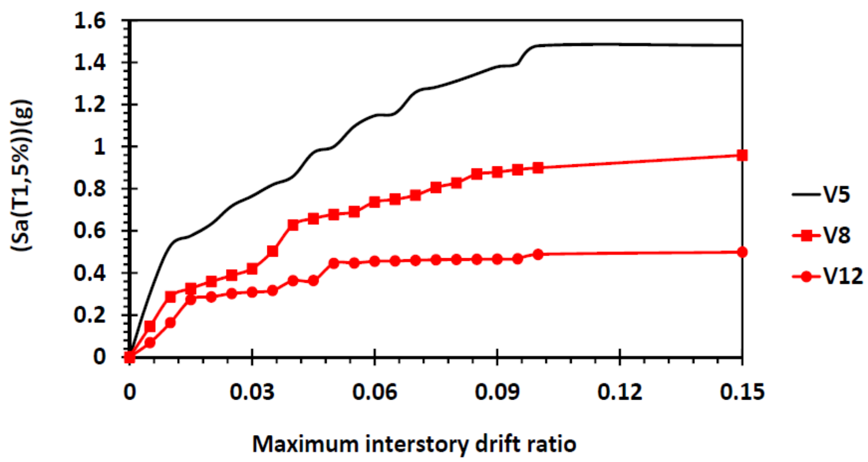

Fig. 7 Median IDA curves V5-2, V8-2 and V12-2 frames

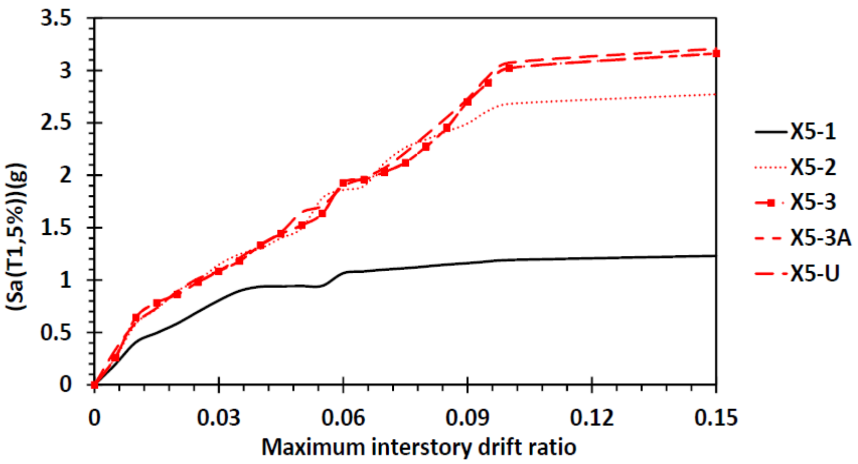

Fig. 8 Median IDA curves for 5 story X-braced frames designed based on different seismic codes

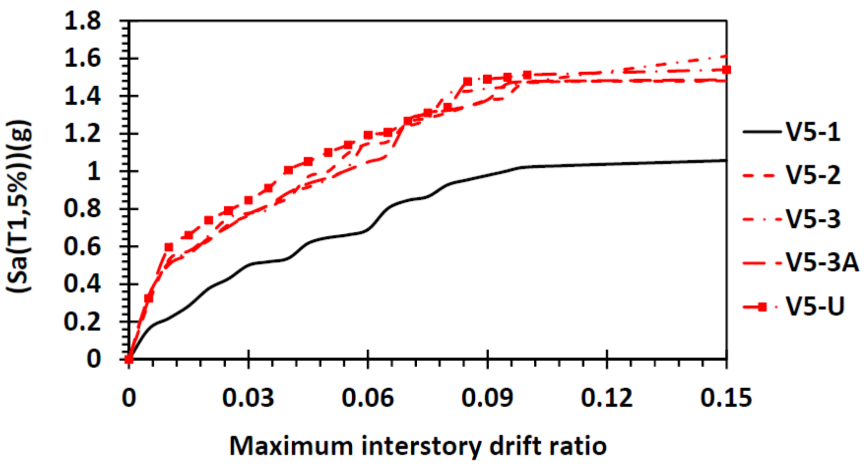

Fig. 9 Median IDA curves for 5 story chevron frames designed based on different seismic codes

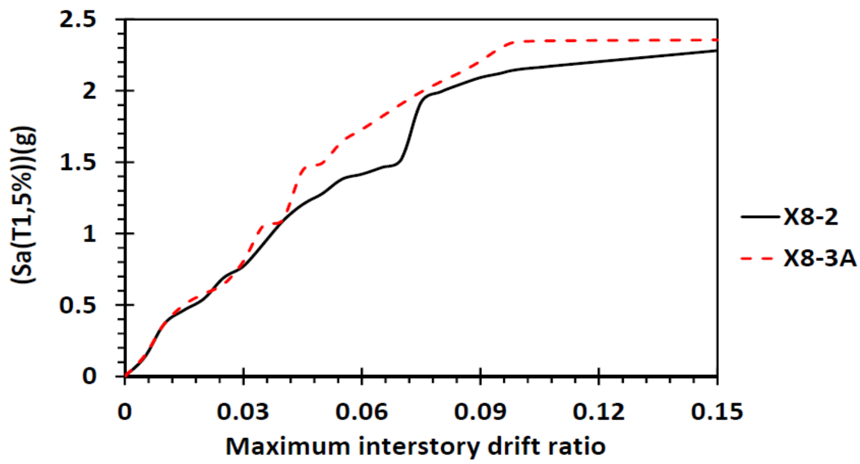

Fig. 10 Median IDA curves for 8 story X-braced frames designed based on different seismic codes

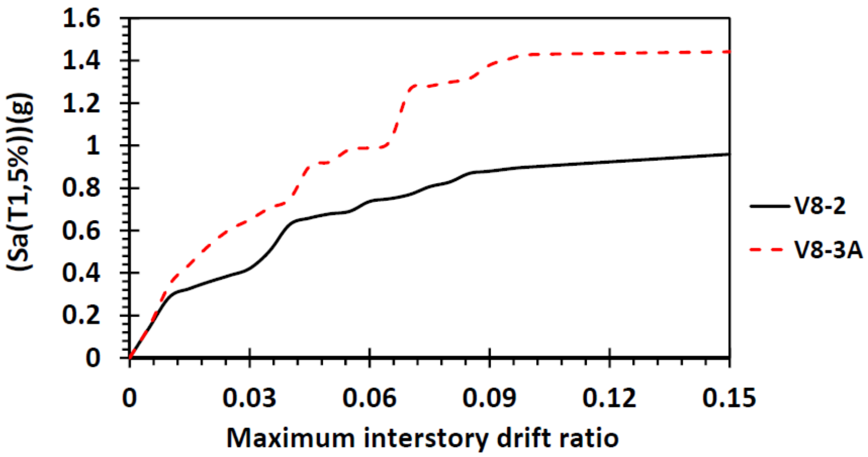

Fig. 11 Median IDA curves for 8 story chevron frames designed based on different seismic codes

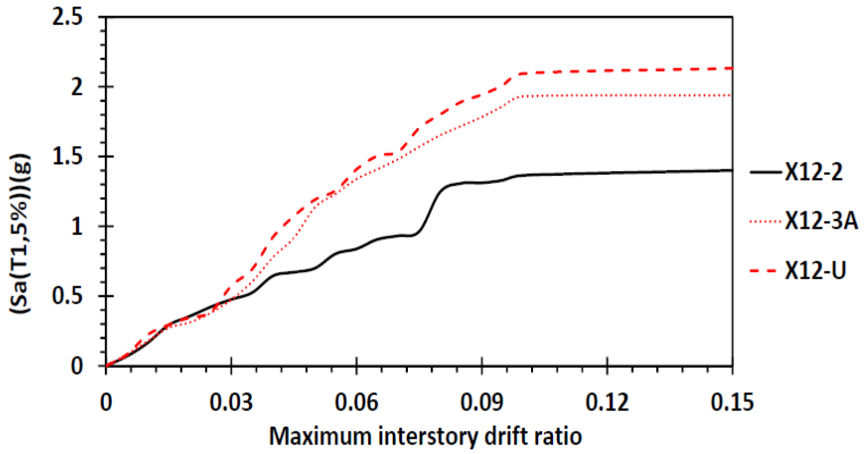

Fig. 12 Median IDA curves for 12 story X-braced frames designed based on different seismic codes

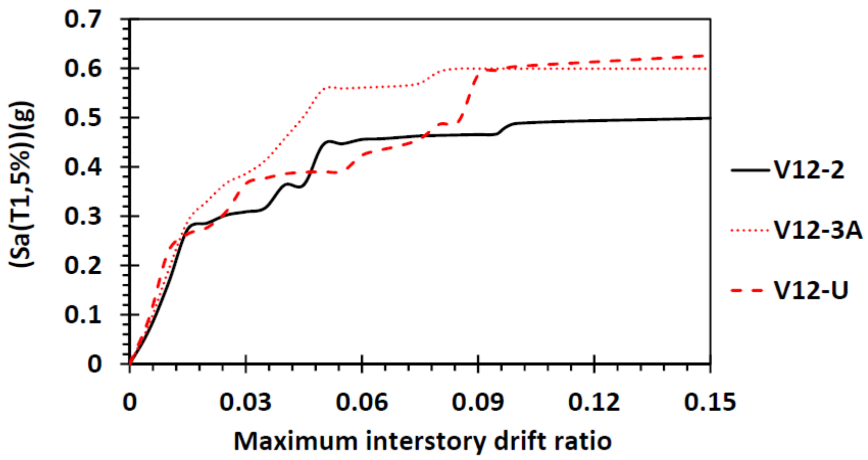

Fig. 13 Median IDA curves for 12 story chevron frames designed based on different seismic codes

Table 4 Limit states of the structures ($S_a(T_1,5\%)$)

Model*	Structural Capacity (IO)	Structural Capacity (CP)	Model*	Structural Capacity (IO)	Structural Capacity (CP)
X5 - 1	0.58 g	1.19 g	V5 - 1	0.38 g	1.02 g
X5 - 2	0.83 g	2.68 g	V5 - 2	0.63 g	1.47 g
X5 - 3	0.86 g	3.02 g	V5 - 3	0.66 g	1.47 g
X5 - 3A	0.86 g	3.02 g	V5 - 3A	0.64 g	1.47 g
X5 - U	0.89 g	3.07 g	V5 - U	0.74 g	1.51 g
X8 - 2	0.54 g	2.15 g	V8 - 2	0.36 g	0.90 g
X8 - 3A	0.58 g	2.34 g	V8 - 3A	0.53 g	1.43 g
X12 - 2	0.30 g	1.36 g	V12 - 2	0.28 g	0.49 g
X12 - 3A	0.31 g	1.93 g	V12 - 3A	0.32 g	0.60 g
X12 -U	0.35 g	2.09 g	V12 -U	0.28 g	0.61 g

Based on the IDA curves the following conclusions are drawn:

- As expected, by increasing the height of the frames, the DM values increase in a constant IM. Also the value of the initial elastic stiffness decreases with the increase in the height of the structures. Besides, higher frames reach limit states (IO and CP) in lower intensity measures.
- The initial elastic stiffness of chevron-braced frames is less than X-braced frames and X-braced frames reach limit states in higher intensity measures than chevron-braced frames.
- The initial tangent slope (say, elastic stiffness) of the IDA curves increases moving from the 1st to the 3rd edition of standard no.2800, and frames behave with higher stiffness. Also frames designed based on the 1st edition of standard no.2800 show an extremely poor performance against near field ground motions.
- Frames designed based on the 2nd edition of standard no.2800 exhibit a better performance. Generally it can be claimed that the requirements of the 3rd edition of Iranian seismic code, AISC seismic provisions and UBC-97 near field design criteria are effective in improving the structural behavior under near filed ground motions. Among them, UBC-97 requirements are the most efficient ones and the frames designed based on the UBC-97 requirements have the highest capacity and reach the limit states in higher intensity measures.
- Also by increasing the frames height, the differences between the behaviors of frames designed based on different seismic codes increase. Thus, earlier editions of the Iranian seismic code are not efficient for seismic design of medium and tall buildings.

7. Reliability analysis

One of the aims of reliability analysis is to determine the mean annual frequency or annual probability of exceeding limit states which is used in “Performance Based Earthquake

Engineering”, (PBEE). This annual probability can be calculated according to Eq. (2). (Vamvatsikos 2002).

$$\lambda_{LS} = \lambda_{(IM\,50\%)} \times \exp(\frac{1}{2}(k \times S_{\ln(IM)})^2) \tag{2}$$

In which:

λ_{LS} = Mean annual frequency of exceeding limit states

$$\lambda_{IM\,50\%} = k_0 \times IM_{50\%}^{-k} \tag{3}$$

$\lambda_{IM\,50\%}$ = Mean annual frequency of exceeding limit states in terms of $IM_{50\%}$

$$S_{\ln(IM)} = \ln(IM_{50\%}) - \ln(IM_{16\%}) \tag{4}$$

$$k = \frac{1.65}{\ln\left(\frac{S_{1(2/50)}}{S_{1(10/50)}}\right)} \tag{5}$$

$S_{1,\,(2/50)}$ = First mode spectral acceleration with 2% probability of occurrence in 50 years
$S_{1,\,(10/50)}$ = First mode spectral acceleration with 10% probability of occurrence in 50 years

And k_0 is calculated using Eq. (6) as follows

$$\frac{1}{475} = k_0 \times (S_{1(10/50)})^{-k} \tag{6}$$

The first step is to extract $S_{1,\,(2/50)}$ and $S_{1,\,(10/50)}$ from an earthquake hazard spectrum. In this paper two different “site specific” hazard spectra are selected which are related to two zones of city of Tehran as shown in Figs. 14 and 15 (http://iranhazard.mporg.ir). Tehran is located in a region of high seismicity covered with multiple active and overlapping faults. The selected locations are assumed near to and with less than 15 km distance to active faults. But the hazard spectra is calculated using several historical earthquakes of near to and far from the site. It can be seen from these figures that hazard spectra for zone 1 is much higher than zone 2. The curves represent two different near-fault points with the same soil type. First mode spectral acceleration with 2% and 10% probability of occurrence in 50 years can be calculated using the curves.

Having the first mode period of each frame, it is possible to calculate $S_{1,\,(2/50)}$, $S_{1,\,(10/50)}$ from the hazard spectra. Thus, coefficients “k” and “k_0” can be calculated from Eqs. (5) and (6).

The next step is to determine the mean annual frequency of exceeding limit states (IO and CP) in terms of $IM_{50\%}$ using Eq. (3). By calculating $S_{\ln,\,(IM)}$ from Eq. (4), all the parameters of Eq. (2) are defined. Thus it is possible to calculate mean annual frequency of exceeding limit states (IO and CP) for each frame.

The return period for occurring each limit state is simply calculated using Eq. (7)

$$T = 1/\lambda_{LS} \tag{7}$$

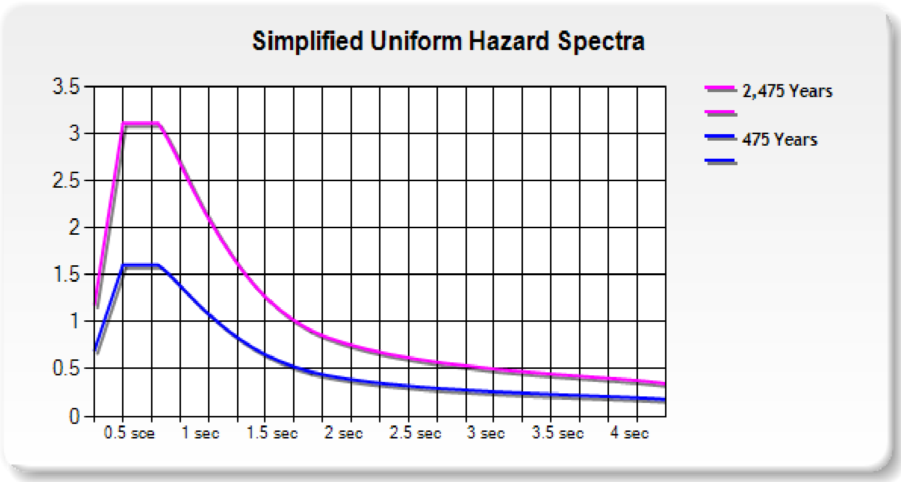

Fig. 14 Uniform hazard spectra for zone 1 (http://iranhazard.mporg.ir)

Table 5 Return periods (years) for earthquakes causing IO and CP limit states in zone 1

Model*	T_{IO}	T_{CP}	Model*	T_{IO}	T_{CP}
X5 - 1	103.20	354.65	V5 - 1	23.41	301.91
X5 - 2	156.54	1624.42	V5 - 2	65.80	351.89
X5 - 3	159.47	2293.92	V5 - 3	66.25	334.22
X5 - 3A	159.47	2293.92	V5 - 3A	61.00	301.56
X5 - U	160.80	2382.75	V5 - U	87.09	341.54
X8 - 2	173.58	4375.91	V8 - 2	54.37	556.24
X8 - 3A	177.23	5093.28	V8 - 3A	124.97	1277.38
X12 - 2	183.63	4018.87	V12 - 2	106.26	468.52
X12 - 3A	186.91	16713.99	V12 - 3A	110.96	741.98
X12 -U	223.93	17329.16	V12 -U	113.10	591.02

*Model definitions are presented in Table 1

The return periods of earthquakes causing the IO and CP limit states for each of the two seismic zones are summarized in Tables 5 and 6.

Considering the results, the return period of earthquakes leading to IO and CP limit states for different structures is not the same. It is interesting to know that, by increasing the height, the return period of the earthquake causing limit states in the structures designed with the same code will increase. Although taller frames reach limit states in lower IMs (as seen in Table 4), the earthquake hazard for longer periods is much smaller than the hazard for shorter ones. The greater amount of earthquake hazard in low rise frames (as shown in Figs. 14-15) affects the return period much more than the structural capacity. Thus the return period of earthquakes causing limit states in taller frames is more than low rise ones. Also the structures designed based on the UBC-97 near field seismic design criteria, have longer return periods comparing to the others. This proves the wider safety margin for the structures designed based on UBC-97 in near-field ground motions.

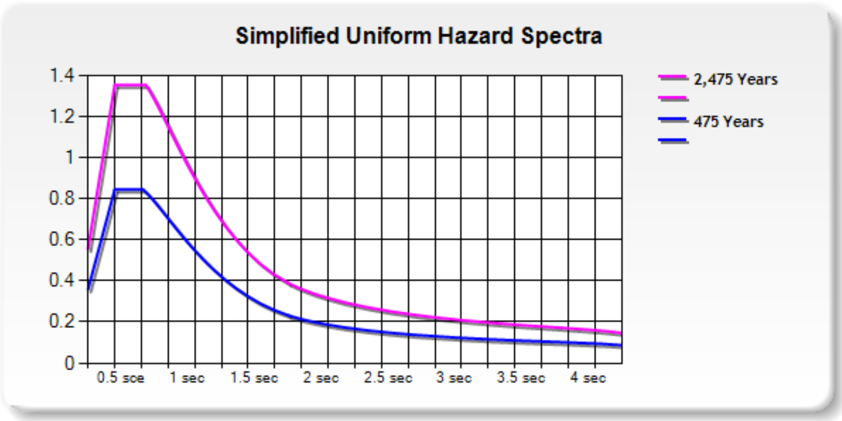

Fig. 15 Uniform hazard spectra for zone 2 (http://iranhazard.mporg.ir)

Table 6 Return periods (years) for earthquakes causing IO and CP limit states in zone 2

Model*	T_{IO}	T_{CP}	Model*	T_{IO}	T_{CP}
X5 - 1	554.75	2268.71	V5 - 1	67.44	2252.84
X5 - 2	986.74	17886.12	V5 - 2	294.41	2350.46
X5 - 3	1020.26	29888.03	V5 - 3	327.30	2367.24
X5 - 3A	1020.26	29888.03	V5 - 3A	292.77	1978.10
X5 - U	1044.51	33417.64	V5 - U	424.51	2318.59
X8 - 2	1081.18	69721.84	V8 - 2	258.68	5335.60
X8 - 3A	1191.33	88014.96	V8 - 3A	752.88	16583.04
X12 - 2	1496.50	48210.83	V12 - 2	597.15	4073.41
X12 - 3A	1544.12	434092.69	V12 - 3A	800.05	8336.31
X12 -U	1925.16	441227.13	V12 -U	693.91	5161.56

*Model definitions are presented in Table 1

7.1 Fragility analysis

Fragility curves indicate cumulative probability of meeting a specific performance level (e.g. limit states) for different earthquake intensities which would help to determine structural vulnerability. These curves are extracted from IDA outputs in the present study.

For drawing a fragility curve, intensity measures corresponding to a specific performance level (e.g. IO) for all ground motion time histories are defined using IDA curves. The IMs are arranged in a descending way. By normalizing these sorted values, the possibility of meeting the performance level can be calculated using a “Cumulative Distribution Function” (CDF).

Figs. 16-21 show sample fragility curves for some of the assumed frames in IO and CP limit states. Using these curves, the cumulative probability of meeting limit states can be calculated for various earthquake intensity levels. Also IMs corresponding to a definite performance level can be extracted. Table 7 summarizes the values of $S_a(T_1,5\%)$ corresponding to 50% possibility of meeting IO and CP limit states.

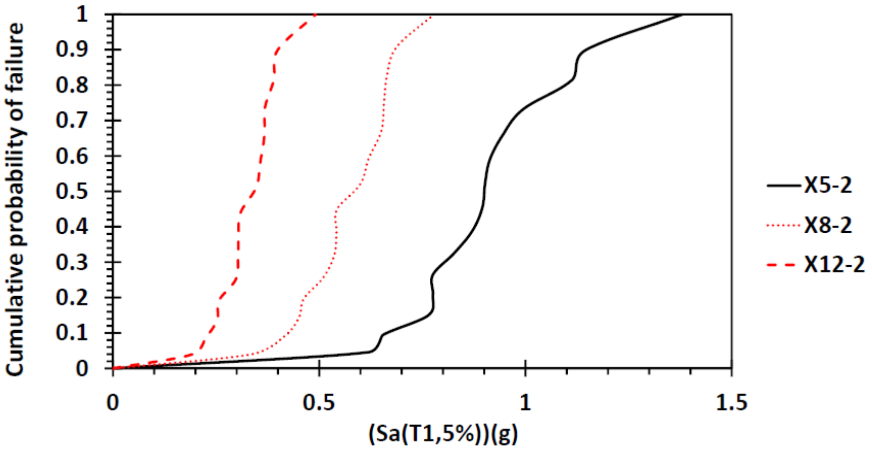

Fig. 16 Fragility curves for X5-2, X8-2 and X12-2 frames for IO performance level

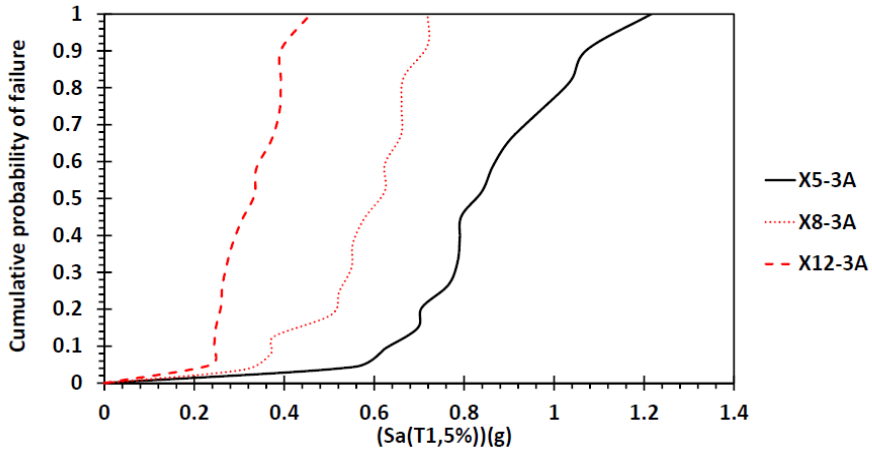

Fig. 17 Fragility curves for X5-3A, X8-3A and X12-3A frames for IO performance level

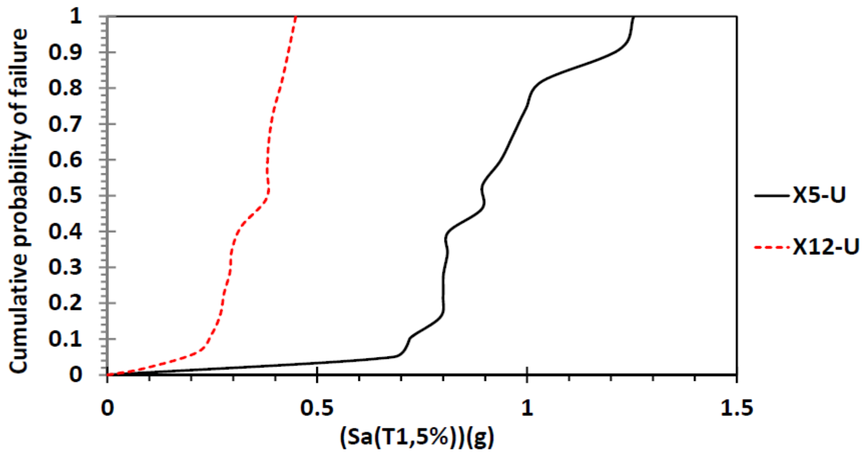

Fig. 18 Fragility curves for X5-U and X12-U frames for IO performance level

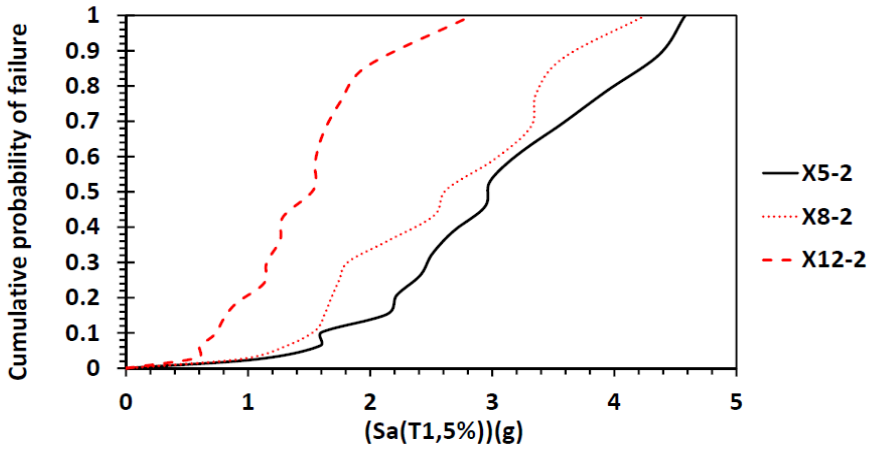

Fig. 19 Fragility curves for X5-2, X8-2 and X12-2 frames for CP performance level

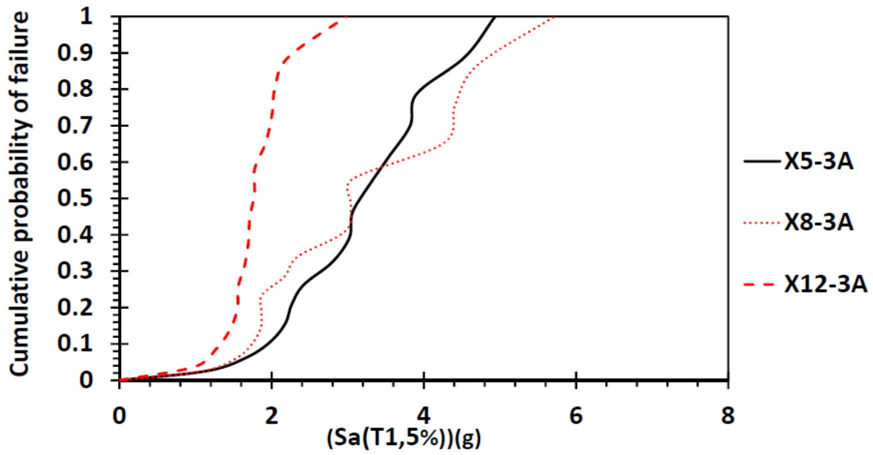

Fig. 20 Fragility curves for X5-3A, X8-3A and X12-3A frames for CP performance level

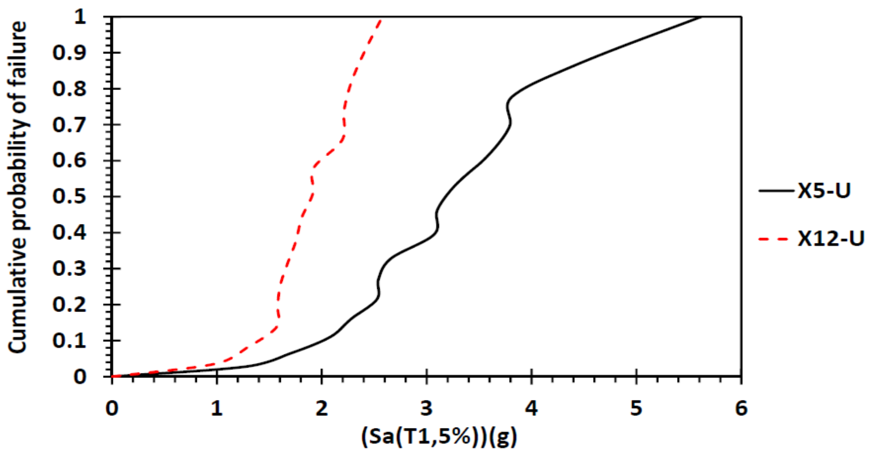

Fig. 21 Fragility curves for X5-U and X12-U frames for CP performance level

Table 7 Values of $S_a(T_1,5\%)$ corresponding to 50% possibility of meeting limit states

Model*	IM for %50 probability of meeting IO	IM for %50 probability of meeting CP	Model*	IM for %50 probability of meeting IO	IM for %50 probability of meeting CP
X5 - 1	0.66 g	1.35 g	V5 - 1	0.39 g	1.13 g
X5 - 2	0.90 g	2.96 g	V5 - 2	0.63 g	1.81 g
X5 - 3	0.83 g	3.17 g	V5 - 3	0.67 g	1.87 g
X5 - 3A	0.83 g	3.17 g	V5 - 3A	0.64 g	1.60 g
X5 - U	0.90 g	3.19 g	V5 - U	0.76 g	1.78 g
X8 - 2	0.57 g	2.61 g	V8 - 2	0.36 g	0.89 g
X8 - 3A	0.61 g	3.04 g	V8 - 3A	0.55 g	1.44 g
X12 - 2	0.34 g	1.52 g	V12 - 2	0.29 g	0.69 g
X12 - 3A	0.33 g	1.76 g	V12 - 3A	0.34 g	0.72 g
X12 -U	0.38 g	1.91 g	V12 -U	0.28 g	0.61 g

*Model definitions are presented in Table 1

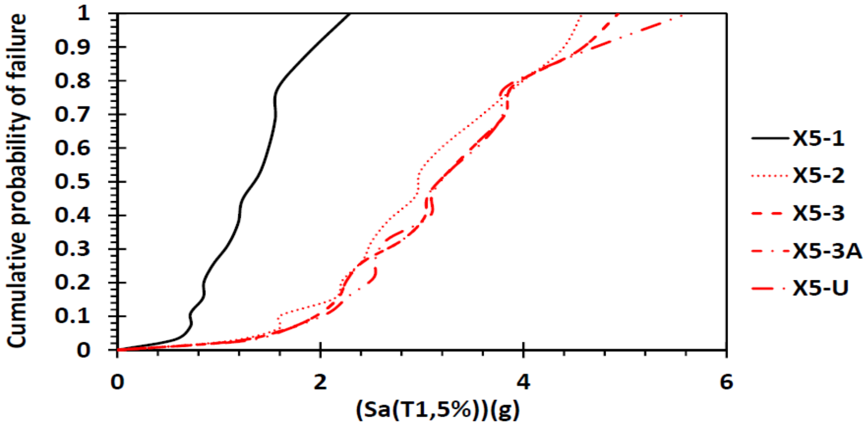

Fig. 22 Fragility curves for 5 story X-braced frames (CP limit state)

It can be seen that, by increasing the height of buildings, the possibility for satisfying performance levels in a constant level of seismic intensity decreases. Thus taller frames are more probable to meet IO and CP limit states than low rise ones. Also, frames designed based on the UBC-97 near-field criteria are more reliable and their probability of meeting limit states is less than the others.

Figs. 22-24 compare the fragility curves for X-braced frames designed based on different seismic codes.

8. Conclusions

Seismic performance of concentrically braced frames of 5, 8 and 12 stories, with X and Chevron braces, designed based on the Iranian seismic design code, AISC seismic provisions and

UBC-97 requirements is studied through IDA. Reliability of the frames is assessed based on the results and fragility curves are extracted for IO and CP limit states. The following conclusions can be drawn:

- By increasing the height, the structural capacity significantly decreases which indicates the more destructive effects of near field ground motions on the tall buildings.
- The initial tangent slope (say, elastic stiffness) of the curves increases from the first edition to the third, and frames behave with higher stiffness.
- The 1st edition of standard no.2800 has no special requirements for reinforcing the braced frames against near field ground motions. Thus, frames designed with this edition of standard no.2800 have extremely lower capacities. In contrast, the 2nd and 3rd editions standard no.2800, increase the capacity of braced frames by reinforcing the columns and braces.
- Structures designed based on UBC-97 near fault seismic design requirements have higher capacities comparing to ones designed based on the other codes and meet IO and CP limit states in greater IMs. The differences are more distinct in taller buildings.
- Taking into account the seismic provisions of AISC, link beams in chevron-braced frames are reinforced. The Chevron-braced frames designed based on AISC-seismic provision have higher capacities comparing to the ones designed based on the 3rd edition standard no.2800.
- Considering fragility curves, the probability of exceeding limit states in a constant level of earthquake intensity increases by increasing the structures height. Although taller frames reach limit states in lower IMs, the earthquake hazard for longer periods is much smaller than the hazard for shorter ones. The greater amount of earthquake hazard in low rise frames affects the return period much more than the structural capacity.
- The probability of meeting IO and CP limit state in the structures designed based on UBC-97 near field requirements is less than the ones designed based on the other seismic codes.

Structures designed based on UBC-97 have earthquake with longer return periods causing IO and CP limit states.

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