**Structural Engineering and Mechanics**, *Vol. 63, No. 2 (2017) 259-268*
DOI: https://doi.org/10.12989/sem.2017.63.2.259


# Evaluation of the seismic performance of special moment frames using incremental nonlinear dynamic analysis

Majid Khorami[1,2a], Masoud Khorami[3], Hedayatollah Motahar[3], Mohammadfarid Alvansazyazdi[4,5a], Mahdi Shariati[6,7b], Abdolrahim Jalali[*6] and M.M. Tahir[7a]

[1]*Facultad de Arquitectura y Urbanismo, Universidad Tecnológica Equinoccial, Calle Rumipamba s/n y Bourgeois, Quito, Ecuador*
[2]*Facultad de Ingeniería Civil y Ambiental, Escuela Politécnica Nacional, Ladrón de Guevara E11-253L, Quito, Ecuador*
[3]*Civil Engineering Department, Islamic Azad University, Tehran, Iran*
[4]*Facultad Ingeniería Ciencias Físicas y Matemática, Carrera Ingeniería Civil, Universidad Central del Ecuador, Quito, Ecuador*
[5]*Facultad de Arquitectura, Universidad Laica Eloy Alfaro de Manabí, Ecuador*
[6]*Faculty of Civil Engineering, University of Tabriz, Tabriz, Iran*
[7]*UTM Construction Research Centre, Faculty of Civil Engineering, Universiti, Teknologi Malaysia (UTM), 81310 Johor Bahru, Johor, Malaysia*

*(Received May 10, 2016, Revised December 25, 2016, Accepted February 9, 2017)*

**Abstract.** In this paper, the incremental nonlinear dynamic analysis is used to evaluate the seismic performance of steel moment frame structures. To this purpose, three special moment frame structure with 5, 10 and 15 stories are designed according to the Iran's national building code for steel structures and the provisions for design of earthquake resistant buildings (2800 code). Incremental Nonlinear Analysis (IDA) is performed for 15 different ground motions, and responses of the structures are evaluated. For the immediate occupancy and the collapse prevention performance levels, the probability that seismic demand exceeds the seismic capacity of the structures is computed based on FEMA350. Also, fragility curves are plotted for three high-code damage levels using HASUS provisions. Based on the obtained results, it is evident that increase in the height of the frame structures reduces the reliability level. In addition, it is concluded that for the design earthquake the probability of exceeding average collapse prevention level is considerably larger than high and full collapse prevention levels.9.

**Keywords:** incremental nonlinear dynamic analysis; reliability of structures; fragility curves; seismic demand and capacity; performance level; confidence level

## 1. Introduction

While buildings are usually designed for seismic resistance using elastic analysis, most will experience significant inelastic deformations under large earthquakes (Khorramian *et al*. 2015, Khanouki *et al*. 2016). The increasingly advancements in computer technologies have provided the possibility of developing strength numerical methods for dynamic nonlinear analysis of structures (Arabnejad Khanouki *et al*. 2010, Arabnejad Khanouki *et al*. 2011, Shahabi *et al*. 2016). As such, the availability of high-quality software, such as OPENSEES, an open-source freely available software, provide the means for predicting structural response beyond the elastic range, including inelastic material property, hysteretic behavior, and panel zone modeling in steel 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, Jalali *et al*. 2012, Feizi *et al*. 2015, Tahmasbi *et al*. 2016, Toghroli *et al*. 2016) in that response history analyses of a given structure is calculated in a systematic manner which will be discussed later. This method is widely used for seismic evaluation of nonlinear response of structures subjected to a suite of severe strong motion (Niknam *et al*. 2007, Shariati *et al*. 2010, Jalali *et al*. 2012, Farahi and Mofid 2013, Azimi *et al*. 2015, Lee and Kim 2015, Shariati *et al*. 2015, Shah *et al*. 2016). In this article IDA approach together with fragility analysis are used to evaluate the seismic performance of special moment frames.

## 2. Incremental nonlinear dynamic analysis (IDA)

As mentioned earlier, IDA is an approach to assess the nonlinear behavior of a structure subjected to a suite of strong motions. The structure is repeatedly analyzed for each motion scaled for gradually increasing the applied strong motion time-history and calculating the corresponding certain damage measures (DM) and plotting against earthquake intensity measure (IM) to produce "IDA Curves". This concept is suggested by Bertero in 1977 (Kelly and Tasi 1984, Shariati *et al*. 2012, Metin Kose and Kayadelen 2013, Mohammadhassani *et al*. 2013, Mohammadhassani *et al*. 2014, Toghroli Ali *et al*. 2014, Safa *et al*. 2016) and developed further by several

∗Corresponding author, Ph.D.
E-mail: jalali@tabrizu.ac.ir
[a]Ph.D.
[b]Ph.D. Student


http://www.techno-press.com/journals/sem&subpage=7
ISSN: 1225-4568 (Print), 1598-6217 (Online)

Table 1 Cross sections for all members of models

| 15 Story Structure | | | 10 Story Structure | | | 5 Story Structure | | |
|---|---|---|---|---|---|---|---|---|
| Stories | Columns | Beams | Stories | Columns | Beams | Stories | Columns | Beams |
| 1,2,3 | Box 450×450×20 | IPE 450 | 1,2 | Box 420×420×20 | IPE 450 | 1,2,3 | Box 180×180×20 | IPE 360 |
| 4,5,6 | Box 380×380×20 | IPE 450 | 3,4,5,6 | Box 380×380×15 | IPE 450 | 4 | Box 160×160×16 | IPE 360 |
| 7,8,9 | Box 380×380×15 | IPE 450 | 7,8,9 | Box 280×280×10 | IPE 360 | 5 | Box 160×160×16 | IPE 300 |
| 10,11,12 | Box 280×280×10 | IPE 360 | 10 | Box 280×280×10 | IPE 330 | | | |
| 13,14,15 | Box 220×220×10 | IPE 300 | | | | | | |

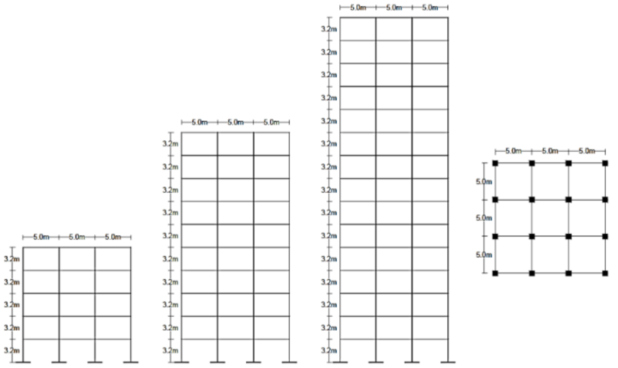



Fig. 1 Studied structural models

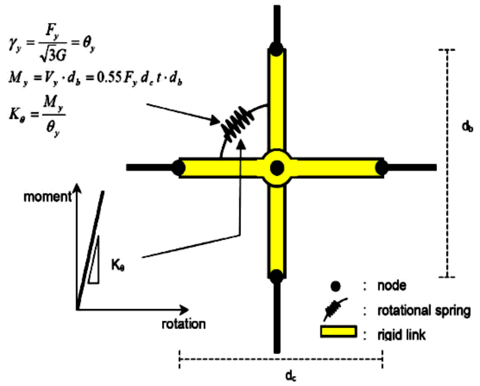



Fig. 2 Model of the panel zone

investigators such (Mohammadhassani *et al*. 2014, Kelly and Tasi 1984, Nassar and Krawinkler 1991, Bazzurro and Cornell 1994, 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*. 2013, Mohammadhassani *et al*. 2014). Traditionally, damage measure may be in the forms of maximum inter-story drift, maximum displacement, base shear, Park and Ang damage index, plastic hinge rotation and so on (Bazzurro and Cornell 1994, Bazzurro and Cornell 1994, Luco and Cornell 1998, Luco and Cornell 2000, Daie *et al*. 2011, Shariati *et al*. 2015, Shariati and Schumacher 2016).

## 3. Appropriate selection of IM and DM

The 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 (T1, 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), $\theta_{max}$ (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 (T1, 5%) and $\theta_{max}$ are selected as IM and DM.

## 4. Structural models

Table 2 Uniform live and dead loads

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

In this study, three 5, 10 and 15 story steel building are modeled. The structural system of these buildings is special moment frame. Fig. 1 and Table 1 shows the structures and their plan. They have three 5 meters' spans in both directions, and the story height is 3.2 meters.

The buildings are loaded gravitationally based on the 6th volume of Iran's national building code. Table 2 lists intensities of uniform dead and live loads applied to stories of the structures.

Due to the importance of the panel zone and its characteristic in the seismic behavior of structures, its effects are also modeled in this work. Fig. 2 demonstrate the utilized model and Fig. 3 related shear-rotation curves.

## 5. Analysis

According to the OpenSees an open-source freely available software, accuracy in analysis time history and pushover, modeling in this program is done. In the archive materials presented in the software, there are two types of steel material.

Material Steel 01 supplier elastoplastic curve by consider hardening can be made a model of behavior but according to the Fig. 4(a) transition zone between elastic and plastic phases is steep angle and fracture.

Table 3 Natural periods of structures in seconds

| Mode No. | 5 Story building | 10 Story building | 15 Story building |
|---|---|---|---|
| 1st mode | 1.219 s | 2.000 s | 2.574 s |
| 2nd mode | 0.441 s | 0.747 s | 1.047 s |

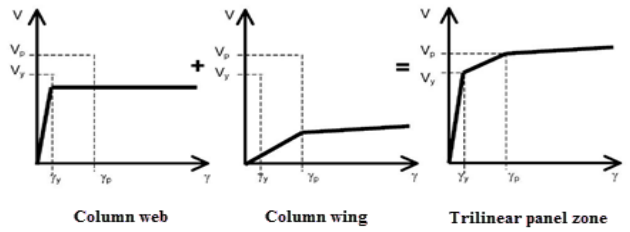



Fig. 3 Shear force-rotation curves

Table 4 Pushover data

| | $V_{\max}$ (KN) | $0.8V_{\max}$ (KN) | $\delta_{y.eff}$ (m) | $\delta_u$ (m) | $\Omega = \frac{V_{\max}}{V}$ |
|---|---|---|---|---|---|
| 5 Story Structure | 295.0 | 236.0 | 0.25 | 0.67 | $\Omega_5$=7.1 |
| 10 Story Structure | 278.0 | 222.4 | 0.61 | 0.99 | $\Omega_{10}$=7.5 |
| 15 Story Structure | 173.0 | 134.8 | 0.65 | 2.45 | $\Omega_{15}$=8.4 |

What material Steel 02 Distinguished from other material Steel 01, is transition area from elastic phase to plastic phase with a soft curve.This advantage is not only for the behavior of material closer to the real behavior of steel, but also will be effects positive in the process of structural analysis and received reasonable results, Fig. 4(b). To increase the accuracy of calculations of structural members and frames are divided into smaller pieces modeled by nonlinear displacement-based beam-column elements.

### 5.1 Modal analysis

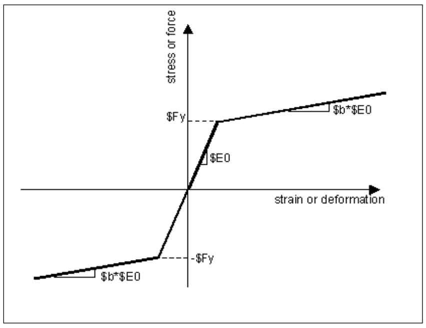
The purpose of modal analysis is to determine natural periods of vibration of the structures. The results for the first two modes of vibration are presented in Table 3.

### 5.2 Nonlinear static analysis (Pushover)

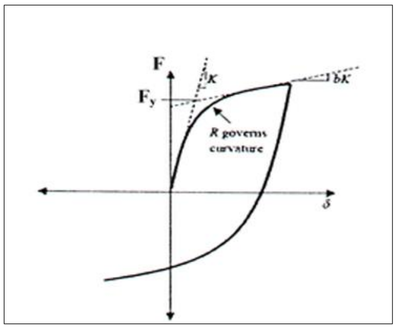
A simpler option to assess the performance of structures is pushover analysis or simplified nonlinear static analysis. This method assumes that the response of a structure can be predicted by the first or the first few modes of vibration. It involves the incremental application of loading that follows some predetermined load pattern until the failure modes of the structure can be identified thus producing a force-displacement relationship or capacity curve, which gives a clear indication of the nonlinear response. Figs. 5,6,7 and Table 4 show the pushover curves for the three 5, 10, and 15 story steel structures.

As seen in Table 4, ultimate values roof displacement and over strength for 15 Story structure is far more values of 5 story structure. (FEMA 695).

### 5.3 Incremental nonlinear dynamic analysis (IDA)




(a) Steel01




(b) Steel02

Fig. 4 Nonlinear materials in OpenSees

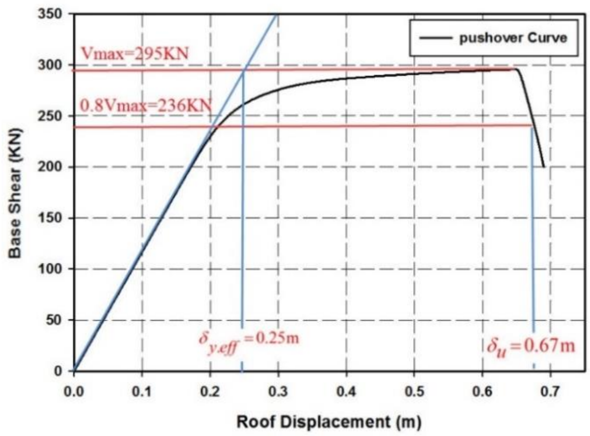



Fig. 5 Pushover diagrams 5 story structure

#### 5.3.1 Selected ground motion

To perform incremental nonlinear analysis, 15 different strong motion records are selected as listed in Table 5. The site soil types (soil type II based on the Iranian seismic code termed code No. 2800), the closest distances to the causative event, the magnitudes of strong motions in terms of Richter scale, their components, and their PGAs are shown in the Table 5. The selected three steel structures are subjected to these ground motions following the IDA procedures which will be explained later.

#### 5.3.2 Limit states based on the IDA curves

Within recent years, three performance levels (limit states) have been introduced representing damage levels corresponding to three hazards leveled target events, i.e., Immediately occupancy (IO), Life safety (LS), and Collapse prevention (CP).

Table 5 Ground motion records used for IDA

| No. | Event | Station | Soil1 | R2 (km) | M3 | $\phi^{o4}$ | PGA (g) |
|---|---|---|---|---|---|---|---|
| 1 | 1994Northrige, | Castaic-Old Ridge Route24278 | B | 22.6 | 6.7 | 090 | 0.568 |
| 2 | 1994Northrige, | LA-116th St School14403 | B | 41.9 | 6.7 | 090 | 0.208 |
| 3 | 1994Northrige, | Malibu-Point Dume Sch24396 | B | 35.2 | 6.7 | 090 | 0.130 |
| 4 | 1994Northrige, | LA-Obregon Park24400 | B | 37.9 | 6.7 | 090 | 0.355 |
| 5 | 1971San Fernando, | Palmdale Fire Station262 | B | 25.4 | 6.6 | 210 | 0.151 |
| 6 | 1971San Fernando, | Pasadena-CIT Athenaeum80053 | B | 31.7 | 6.6 | ... | 0.088 |
| 7 | 1971San Fernando, | Upland-San Antonio Dam287 | B | 58.1 | 6.6 | 015 | 0.058 |
| 8 | 1971San Fernando, | Wrigjtwood-6074Park Dr290 | B | 60.3 | 6.6 | 025 | 0.061 |
| 9 | 1979Imperial Valley, | Cerro Prieto6604 | B | 26.5 | 6.5 | 147 | 0.169 |
| 10 | 1989Loma Prieta, | Fremont-Mission San Jose57064 | B | 43.0 | 6.9 | ... | 0.124 |
| 11 | 1989Loma Prieta, | SAGO South-Surface47189 | B | 34.7 | 6.9 | 261 | 0.073 |
| 12 | 1994Northrige, | Lnglewood-Union Oil14196 | B | 44.7 | 6.7 | ... | 0.091 |
| 13 | 1989Loma Prieta, | Belmont-Envirotech58262 | B | 49.9 | 6.9 | ... | 0.108 |
| 14 | 1989Loma Prieta, | Berkeley LBL58471 | B | 83.6 | 6.9 | ... | 0.057 |
| 15 | 1989Loma Prieta, | Golden Gate Bridge1678 | B | 85.1 | 6.9 | 270 | 0.233 |

Table 6 Summary of the performance capacity levels of the three structures

| | 5 Story Structure | | | | | | 10 Story Structure | | | | | | 15 Story Structure | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sa (T1,5%) | | | $\theta_{max}$ | | | Sa (T1,5%) | | | $\theta_{max}$ | | | Sa (T1,5%) | | | $\theta_{max}$ | | |
| | IO | CP | GI | IO | CP | GI | IO | CP | GI | IO | CP | GI | IO | CP | GI | IO | CP | GI |
| 16% | 0.20 | 0.95 | 2.50 | 0.02 | 0.050 | $+\infty$ | 0.28 | 1.10 | 1.30 | 0.02 | 0.100 | $+\infty$ | 0.18 | 0.29 | 2.00 | 0.02 | 0.049 | $+\infty$ |
| 50% | 0.19 | 0.92 | 2.00 | 0.02 | 0.052 | $+\infty$ | 0.23 | 0.64 | 1.10 | 0.02 | 0.061 | $+\infty$ | 0.10 | 0.30 | 1.20 | 0.02 | 0.070 | $+\infty$ |
| 84% | 0.18 | 0.72 | 1.53 | 0.02 | 0.043 | $+\infty$ | 0.15 | 0.73 | 1.00 | 0.02 | 0.100 | $+\infty$ | 0.08 | 0.20 | 1.00 | 0.02 | 0.060 | $+\infty$ |

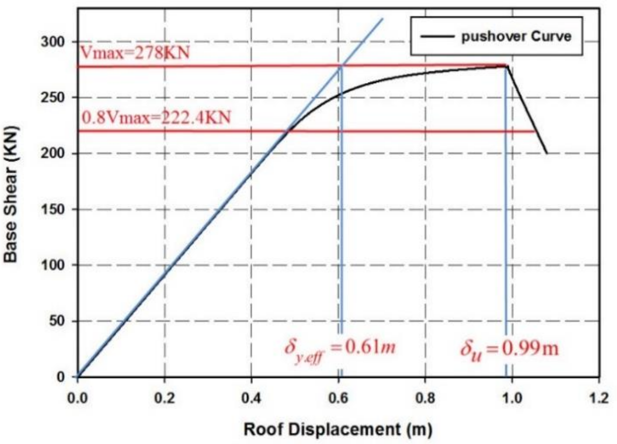



Fig. 6 Pushover diagrams 10 story structure

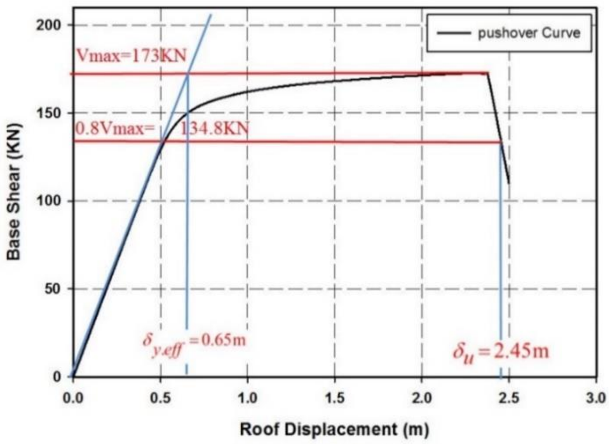



Fig. 7 Pushover diagrams 15 story structure

Today, the three seismic performance levels of structures against the three hazard leveled events i.e., strong motion with 50% probability of exceedance (PE) in 50 years for IO, PE of 10% in 50 years for LS, and PE of 2% in 50 years for CP may be evaluated using the IDA approach.

Each of these performance level evaluations could give some useful information on structural damage level, underlying assumptions and application limits In this study, FEMA350 requirements are used to calculate and evaluate the performance levels of the selected structures.

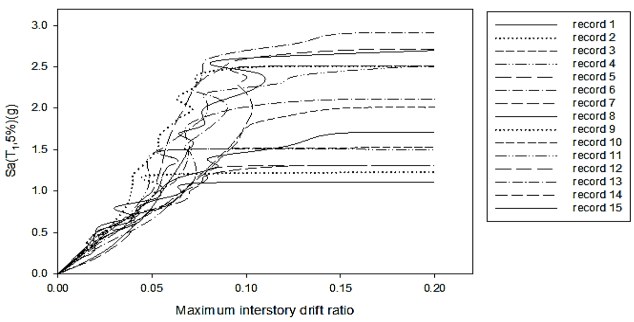



Fig. 8 IDA curves 5 story structure

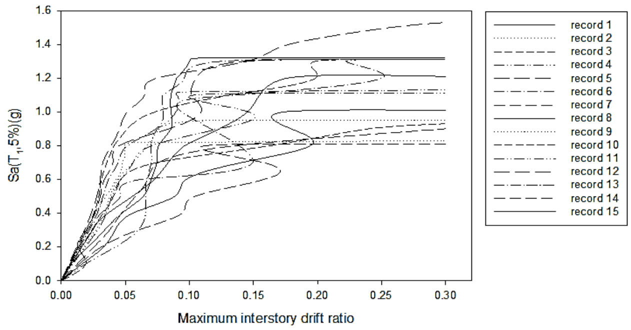



Fig. 9 IDA curves 10 story structure

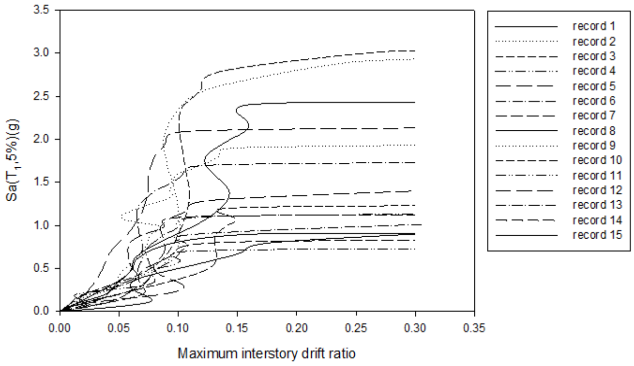



Fig. 10 IDA curves 15 story structure

Figs. 8, 9, 10 present the calculated IDA curves for the three steel structures.

IDA curves depict wide range of structural behavior, and considerable dispersion exists for multiple ground motions as seen in the figures. Therefore simplification methods are required to reach the compact responses permitting their uncertainties to be valuable (Niknam and Eskandari 2010). For this purpose, the three statistical-based curves 16%, 50%, and 84% levels as the result of applying the selected suite of strong motion over each of the three structures are extracted from the plotted IDA curves (Vamvatsikos and Cornell 2002, Yun *et al*. 2002) and the results are depicted as shown in Figs. 11, 12, 13.

The criteria identifying each of the abovementioned structure's performance levels, (IO), (LS), and (CP), in terms of spectral acceleration at the first mode period Sa(T1,5%)g, presented by FEMA350 are used to evaluated the seismic performance of the three structures. Based on FEMA 350 the maximum structure's capacity corresponding to the immediate occupancy, the collapse prevention, and the ultimate capacity performance levels for special moment frame structures are 2%, 10%, and 20% respectively. These limit states are calculated using the prepared compact IDA curves and summarized as listed in Table 6.

For example, as seen in Table 6, the collapse performance level (CP) of the 5 story structure, in term of spectral acceleration at T1 with five percent dumping, identified by the median compact curve (Fig. 11) are Sa(T1, 5%)=0.92 g or the maximum rotation of $\theta_{max}$=0.052 Radian.

In other words, at these values, a record equal to the 50

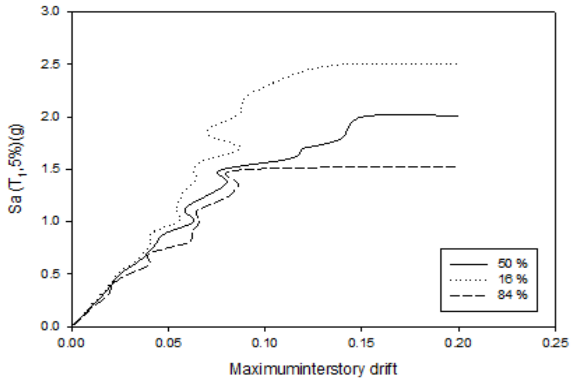



Fig. 11 Compact IDA curves 5 story structure

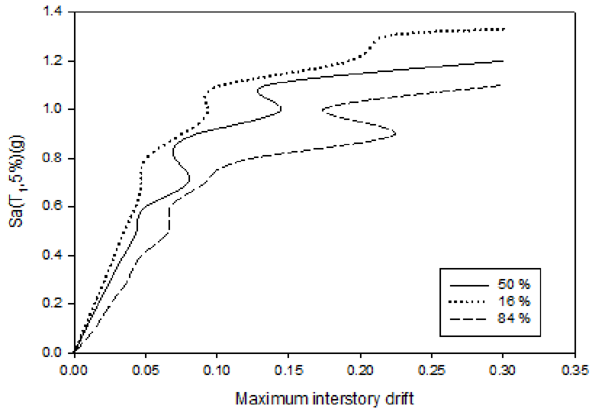



Fig. 12 Compact IDA curves 15 story structure

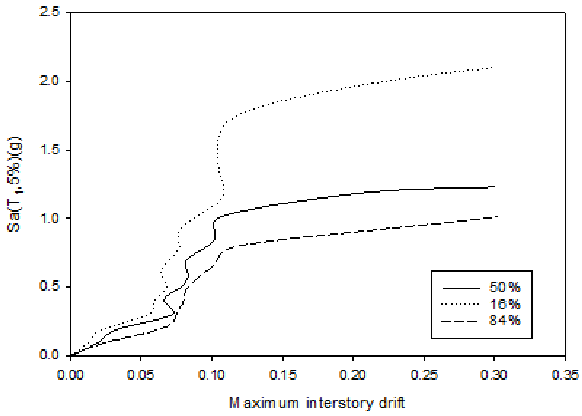



Fig. 13 Compact IDA curves 10 story structure

percent of records brings the 5 story building to collapse prevention level. These limits for IO level are Sa(T1, 5%)=0.19 g or $\theta_{max}$=0.02 Radian and GI level occurs at Sa(T1, 5%)=2.00 g.

## 6. The confidence limits of the structures

Unfortunately, it is not possible to calculate capacity and seismic demand of structures deterministically, because of the existing uncertainties in prediction of ground motions, structural response and damage resistance capacity of structures (Tavakoli and Ghafory-Ashtiany 1999).

Table 7 Computed capacities based on relative displacements

| | Structure | Collapse Prevention Level at 2/50 Hazard Level | Immediate Occupancy Level at 50/50 Hazard Level |
|---|---|---|---|
| Global | 5 Story Structure | 0.100 | 0.020 |
| | 10 Story Structure | 0.100 | 0.020 |
| | 15 Story Structure | 0.085 | 0.020 |
| Local | 5 Story Structure | 0.070 | 0.020 |
| | 10 Story Structure | 0.070 | 0.020 |
| | 15 Story Structure | 0.070 | 0.020 |

Prediction of seismic performance of structures is very complex. This complexity is not only due to various affecting parameters but also because of physical behavior and available uncertainties. Some of the major causes of available uncertainties are Inability to exactly model physical behavior, lack of accurate information to define structural characteristics or to predict future ground motions. Especially, prediction of future ground motions based on previous ones introduces large uncertainties to the estimated seismic demand of structures.

The structural characteristic may also be different with assumptions of designer or change during earthquakes. Also, Analysis methods could not be able to model structural behavior because of their simplifications and approximations exactly. To take these possible uncertainties into account for evaluation and prediction of the seismic behavior of structures, one can compute reliability coefficients as follows

$$\lambda = \frac{\gamma . \gamma_a . D}{\varphi . \mathrm{C}} \tag{1}$$

In this equation, $C$ and $D$ are the median predicted capacity and demand of the structure, respectively. These parameters are derived from analysis of the structure. $\gamma$ is the uncertainty coefficient of the seismic demand due to uncertainty in ground motion and structural response prediction. $\gamma_a$ is the uncertainty coefficient of the analysis method and is a function of analysis method and ground motion intensity. Finally, $\varphi$ is the over-strength coefficient that stands for uncertainties in the capacity prediction (Venture 1999).

Table 8 Computed demands based on relative displacements

| Structure | Collapse Prevention Level at 2/50 Hazard Level | Immediate Occupancy Level at 50/50 Hazard Level |
|---|---|---|
| 5 Story Structure | 0.052 | 0.0081 |
| 10 Story Structure | 0.061 | 0.0060 |
| 15 Story Structure | 0.070 | 0.0093 |

### *6.1 Capacity of the structures*

The capacity of the modeled structures for two performance level, namely IO and CP, and in the global and local level are evaluated and presented in Table 7.

### *6.2 Seismic demand of the structures*

The seismic demand of structure is computed for the IO and CP performance levels by taking 2-50 and 50-50 hazard levels, respectively. The 2-50 hazard level means that likelihood of exceeding the certain ground motion intensity in 50 years is 2 percent. For the 50-50 hazard level, this probability is 50 percent. In the computation of the demand, median IDA curve and seismic hazard curve for Tehran (Tavakoli and Ghafory-Ashtiany 1999) are used. The obtained results are listed in the next table:

### *6.3 Computation of the confidence level (C.L)*

Confidence level of the structure is computed based the calculated capacities and demands based on Table A-1 of FEMA350 code (Venture 1999). It should be noted that the derived values belong to the median IDA curve.

In this table C.L values depend on $\lambda$ and $K, \beta_{ut}$. Values of $\lambda$ is obtained based on the formula (1) and $\beta_{ut}$ calculating from tables 4-11, 4-13 FEMA350.

$K$ parameter (logarithmic slope of the hazard curve) calculate according to FEMA 350 code

$$k = \frac{Ln\left(\dfrac{H_{S1(10/50)}}{H_{S1(2/50)}}\right)}{Ln\left(\dfrac{S_{1(2/50)}}{S_{1(10/50)}}\right)} = \frac{1.65}{Ln\left(\dfrac{S_{1(2/50)}}{S_{1(10/50)}}\right)} \tag{2}$$

Table 9 Derived confidence level for CP performance level based on relative displacements

| Collapse prevention performance level at 2-50 hazard level | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Structure | | $C$ | $D$ | $\gamma$ | $\gamma_a$ | $\phi$ | $\lambda$ | $k$ | $\beta_{ut}$ | C.L % |
| Global | 5 Story | 0.1 | 0.052 | 1.2 | 1.06 | 0.85 | 0.77 | 2.5 | 0.40 | 91 |
| | 10 Story | 0.1 | 0.061 | 1.2 | 1.06 | 0.85 | 0.91 | 2.2 | 0.40 | 75 |
| | 15 Story | 0.085 | 0.070 | 1.5 | 1.10 | 0.75 | 1.80 | 2.0 | 0.50 | 26 |
| Local | 5 Story | 0.07 | 0.052 | 1.5 | 1.06 | 0.85 | 1.10 | 2.5 | 0.35 | 56 |
| | 10 Story | 0.07 | 0.061 | 1.2 | 1.06 | 0.85 | 1.30 | 2.2 | 0.35 | 35 |
| | 15 Story | 0.07 | 0.070 | 1.5 | 1.10 | 0.75 | 2.20 | 2.0 | 0.40 | 10 |

Table 10 Derived confidence level for IO performance level based on relative displacements

| Collapse prevention performance level at 2-50 hazard level | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Structure | | $C$ | $D$ | $\gamma$ | $\gamma_a$ | $\phi$ | $\lambda$ | $k$ | $\beta_{ut}$ | C.L % |
| Global | 5 Story | 0.02 | 0.0081 | 1.4 | 1.02 | 1.0 | 0.57 | 3.0 | 0.2 | 0.99 |
| | 10 Story | 0.02 | 0.0060 | 1.4 | 1.02 | 1.0 | 0.42 | 2.8 | 0.2 | 0.99 |
| | 15 Story | 0.02 | 0.0093 | 1.4 | 1.04 | 1.0 | 0.67 | 2.6 | 0.2 | 0.98 |
| Local | 5 Story | 0.02 | 0.0081 | 1.4 | 1.02 | 1.0 | 0.57 | 3.0 | 0.3 | 0.99 |
| | 10 Story | 0.02 | 0.0060 | 1.4 | 1.02 | 1.0 | 0.42 | 2.8 | 0.3 | 0.99 |
| | 15 Story | 0.02 | 0.0093 | 1.4 | 1.04 | 1.0 | 0.67 | 2.6 | 0.3 | 0.95 |

Table 11 Limiting values of relative displacement for various damage levels based on HASUS

| Structure | Structure type | High-code values for various levels | | | |
|---|---|---|---|---|---|
| | | Low | Moderate | Extensive | Complete |
| 5 Story | S1M* | 0.0040 | 0.0080 | 0.020 | 0.0533 |
| 10 Story | S1H* | 0.0030 | 0.0060 | 0.015 | 0.0400 |
| 15 Story | S1H* | 0.0030 | 0.0060 | 0.015 | 0.0400 |

* S1: Steel moment frame
* M: Mid-rise
* H: High-rise

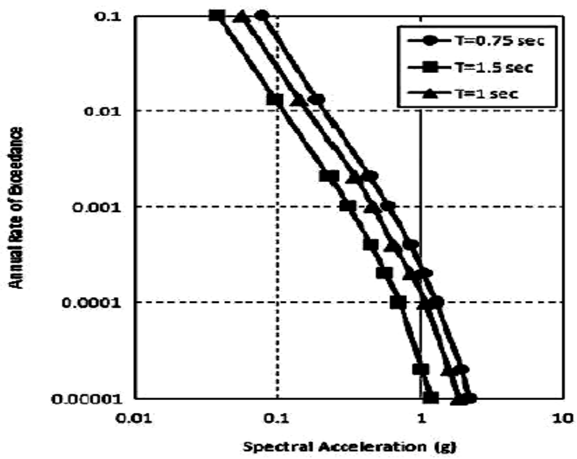



Fig. 14 Probabilistic seismic hazard curve in Tehran

Due to the location of structures in Tehran; Probabilistic Seismic Hazard curve has been used in (Zolfaghari), Fig. 14. The results are presented in Tables 9 and 10.

According to the FEMA350 and its proposed confidence levels, in the immediate occupancy performance level, the performance of the structures is acceptable. But in the case of collapse prevention, an increase in the height of structure reduces its confidence level considerably. Therefore, it seems that 2800 code need revision for mid-rise and high-rise structure at CP performance level.

## 7. Evaluation of fragility curves based on HASUS requirements

Fragility curves are assumed in the form of log-normal probability distribution functions. To plot these functions only two parameters, namely mean and standard deviation, are needed. After determination of mean and standard deviation for each limit state, fragility curve can be plotted using the following equation

$$P\left[LS / x_i\right] = \phi\left[\frac{\ln x_i - ln\bar{x}}{\beta}\right] \qquad (3)$$

In this relationship, $p$[], stands for the probability of a performance level (hear life safety performance level or LS). Xi represents one the earthquake parameters such as spectral acceleration (Sa). $\bar{x}$ is average of relative displacement at desired spectral acceleration. $B$ is the standard deviation and $\varphi$ is the log-normal distribution function.

In this study, based on HASUS three damage level namely moderate, extensive and complete are considered. HASUS takes relative displacement of stories as a quantitative criterion for the performance level of the

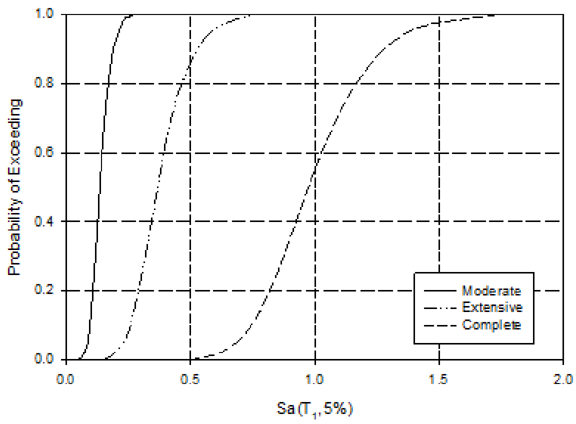



Fig. 15 Fragility curves 5 story structure

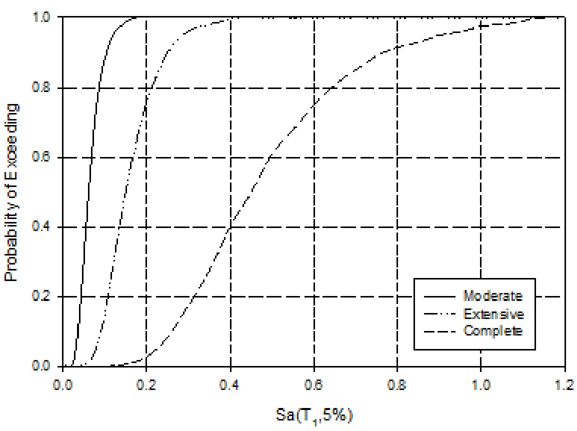



Fig. 16 Fragility curves 10 story structure

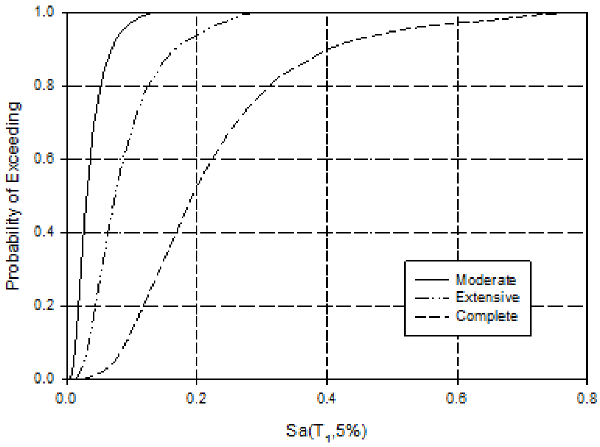



Fig. 17 Fragility curves 15 story structure

structure. Table 9 present HASUS high-code values of this criteria.

The high-code values are taken because according to 2800 code, the structures are located in high earthquake hazard area (Tehran). The resulted fragility curves are depicted in Figs. 14, 15, 16.

## 8. Conclusions

In this paper, seismic behavior of three special moment frame steel structure with 5, 10 and 15 stories are evaluated. These structures are designed based on the Iran's national building code for steel structures and the provisions for design of earthquake resistant buildings (2800 code). Due to the importance of the panel zone in the behavior of the moment frame structures, a nonlinear model which takes effects of the panel zone into account is utilized. Using nonlinear incremental dynamic analysis, the seismic demand and capacity of the structures under 15 different earthquakes are estimated. Also, confidence limits of the structures are determined based on FEMA350, and also fragility curves are presented according to the HASUS requirements. Needless to say, the obtained results are dependent on the selected ground motion records, structural characteristics, and other assumptions. Therefore, change in these parameters may vary the results.

From the derived median curves of the structures, it is evident that increase in the structure height leads to reduce in the equivalent elastic stiffness and the structure becomes more flexible. Comparing seismic demand and capacity of the structures at various performance levels, it is concluded that all of the structures provide an acceptable confidence level for the immediate occupancy performance at 50-50 hazard level. But in the collapse prevention level, only the 5 stor building exhibits sufficient reliability for the 2-50 hazard level. It necessitates revising requirement s of 2800 code for high-rise buildings at the collapse prevention performance level. Finally, the fragility curves reveal (for the design earthquake based on 2800 code), the probability of damage at the moderate level in more than extensive and complete damage levels. In future research, the study aims to research on advanced concrete and composite structure for further investigations, as preliminary findings have been presented by other researchers (Mohammadhassani *et al*. 2014, Abdul Awal 1988, Abdul Awal 1992, Hossain and Awal 2011, Sinaei *et al*. 2011, Shehu and Awal 2012, Hafizah *et al*. 2014, Shariati *et al*. 2014, Muhammad *et al*. 2015, Roslan *et al*. 2016).

## Acknowledgments

The authors would like to acknowledge the support from the Faculty of Civil Engineering, University of Tabriz, Iran, under the Grant No. 102-2500.

## References

Abdul Awal, A.S. (1988), "Failure mechanism of prepacked concrete", *J. Struct. Eng.*, **114**(3), 727-732.

Abdul Awal, A.S. (1992), "Creep recovery of prepacked aggregate concrete", *J. Mater. Civil Eng.*, **4**(3), 320-325.

Arabnejad Khanouki, M.M., Ramli Sulong, N.H. and Shariati, M. (2011), "Behavior of through beam connections composed of CFSST columns and steel beams by finite element studying", *Adv. Mater. Res.*, **168**, 2329-2333.

Arabnejad Khanouki, M.M., Ramli Sulong, N.H. and Shariati, M. (2010), "Investigation of seismic behaviour of composite structures with concrete filled square steel tubular (CFSST) column by push-over and time-history analyses", *Proceedings of the 4th International Conference on Steel & Composite Structures*, July, Sydney, Australia.

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", *Latin Am. J. Solid. Struct.*, **12**(4), 787-807.

Bazzurro, P. and Cornell, C.A. (1994), "Seismic hazard analysis of nonlinear structures. I: Methodology", *J. Struct. Eng.*, **120**(11), 3320-3344.

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

Daie, M., Jalali, A., Suhatril, M., Shariati, M., Arabnejad Khanouki, M.M., Shariati, A. and Kazemi Arbat, P. (2011), "A new finite element investigation on pre-bent steel strips as damper for vibration control", *Int. J. Phys. Sci.*, **6**(36), 8044-8050.

Farahi, M. and Mofid, M. (2013), "On the quantification of seismic performance factors of Chevron Knee Bracings, in steel structures", *Eng. Struct.*, **46**, 155-164.

Feizi, M.G., Mojtahedi, A. and Nourani, V. (2015), "Effect of semi-rigid connections in improvement of seismic performance of steel moment-resisting frames", *Steel Compos. Struct.*, **19**(2), 467-484.

Hafizah, N.A.K., Bhutta, M.A.R., Jamaludin, M.Y., Warid, M.H. Ismail, M., Rahman, M.S., Yunus, I. and Azman, M. (2014), "Kenaf Fiber Reinforced Polymer Composites for Strengthening RC Beams", *J. Adv. Concrete Tech.*, **12**(6), 167-177.

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

Hazus, M. (2008), "Earthquake loss estimation methodology, Technical Manual", National Institute of Building Sciences for

Federal Emergency Management Agency, Washington, DC, MR3 Edition.

Hossain, M. and Awal, A.A. (2011), "Experimental validation of a theoretical model for flexural modulus of elasticity of thin cement composite", *Constr. Build. Mater.*, **25**(3), 1460-1465.

Jalali, A., Daie, M., Nazhadan, S.V.M., Kazemi-Arbat, P. and Shariati, M. (2012), "Seismic performance of structures with pre-bent strips as a damper", *Int. J. Phys. Sci.*, **7**(26), 4061-4072.

Kelly, J. and Tasi, H. (1984), Structural Engineering and Structural Mechanics, Report No. UCB/SESM-84/17.

Khanouki, M.M.A., Ramli Sulong, N.H., Shariati, M. and Tahir, M.M. (2016), "Investigation of through beam connection to concrete filled circular steel tube (CFCST) column", *J. Constr. Steel Res.*, **121**, 144-162.

Khorramian, K., Maleki, S., Shariati, M. and Ramli Sulong, N.H. (2015), "Behavior of Tilted Angle Shear Connectors", *PLoS ONE*, **10**(12), e0144288.

Lee, J. and Kim, J. (2015), "Seismic performance evaluation of moment frames with slit-friction hybrid dampers", *Earthq. Struct.*, **9**(6), 1291-1311.

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.

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.

Metin Kose, M. and Kayadelen, C. (2013), "Effects of infill walls on RC buildings under time history loading using genetic programming and neuro-fuzzy", *Struct. Eng. Mech.*, **47**(3), 401-419.

Mohammadhassani, M., Akib, S., Shariati, M., Suhatril, M. and Khanouki, M.A. (2014), "An experimental study on the failure modes of high strength concrete beams with particular references to variation of the tensile reinforcement ratio", *Eng. Failure Anal.*, **41**, 73-80.

Mohammadhassani, M., Nezamabadi-Pour, H., Suhatril, M. and Shariati, M. (2013), "Identification of a suitable ANN architecture in predicting strain in tie section of concrete deep beams", *Struct. Eng. Mech.*, **46**(6), 853-868.

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.

Muhammad, N.Z., Keyvanfar, A., Majid, M.Z.A., Shafaghat, A. and Mirza, J. (2015), "Waterproof performance of concrete: A critical review on implemented approaches", *Constr. Build. Mater.*, **101**, 80-90.

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. and Eskandari, M. (2010), "Application of incremental dynamic analysis on reinforced concrete frame structures", *3rd National Congress on Strengthening and Urban management*, Iran. (in Persian)

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)

Roslan, N.H., Ismail, M., Abdul-Majid, Z., Ghoreishiamiri, S. and Muhammad, B. (2016), "Performance of steel slag and steel sludge in concrete", *Constr. Build. Mater.*, **104**, 16-24.

Safa, M., Shariati, M., Ibrahim, Z., Toghroli, A., Baharom, S.B., Nor, N.M. and Petković, D. (2016), "Potential of adaptive neuro fuzzy inference system for evaluating the factors affecting steel-concrete composite beam's shear strength", *Steel Compos. Struct.*, **21**(3), 679-688.

Shah, S., Sulong, N.R., Shariati, M., Khan, R. and Jumaat, M. (2016), "Behavior of steel pallet rack beam-to-column connections at elevated temperatures", *Thin Wall. Struct.*, **106**, 471-483.

Shahabi, S., Sulong, N., Shariati, M. and Shah, S. (2016), "Performance of shear connectors at elevated temperatures-A review", *Steel Compos. Struct.*, **20**(1), 185-203.

Shariati, A. and Schumacher, T. (2016), "Eulerian-based virtual visual sensors to measure dynamic displacements with subpixel accuracy of structures and mechanical systems", *J. Struct. Control Hlth. Monit.*, DOI: 10.1002/stc.1977.

Shariati, A., Schumacher, T. and Ramanna, N. (2015), "Eulerian-based virtual visual sensors to detect natural frequencies of structures", *J. Civil Struct. Hlth. Monit.*, **5**(4), 457-468.

Shariati, A., Shariati, M., Ramli Sulong, N.H., Suhatril, M., Arabnejad Khanouki, M.M. and Mahoutian, M. (2014), "Experimental assessment of angle shear connectors under monotonic and fully reversed cyclic loading in high strength concrete", *Constr. Build. Mater.*, **52**, 276-283.

Shariati, M., Ramli Sulong, N., Suhatril, M., Shariati, A., Arabnejad Khanouki, M. and Sinaei, H. (2012), "Fatigue energy dissipation and failure analysis of channel shear connector embedded in the lightweight aggregate concrete in composite bridge girders", *Fifth International Conference on Engineering Failure Analysis*, July, The Hague, The Netherlands.

Shariati, M., Ramli Sulong, N.H. and Arabnejad Khanouki, M.M. (2010), "Experimental and analytical study on channel shear connectors in light weight aggregate concrete", *Proceedings of the 4th International Conference on Steel & Composite Structures*, July, Sydney, Australia.

Shariati, M., Ramli Sulong, N.H., Shariati, A. and Khanouki, M.A. (2015), "Behavior of V-shaped angle shear connectors: experimental and parametric study", *Mater. Struct.*, 49(9), 3909-3926.

Shehu, I. and Awal, A. (2012), "Mechanical properties of concrete incorporating high volume palm oil fuel ash", *Adv. Mater. Res.*, **599**, 537-540.

Sinaei, H., Jumaat, M.Z. and Shariati, M. (2011), "Numerical investigation on exterior reinforced concrete Beam-Column joint strengthened by composite fiber reinforced polymer (CFRP)", *Int. J. Phys. Sci.*, **6**(28), 6572-6579.

Tahmasbi, F., Maleki, S., Shariati, M., Ramli Sulong, N.H. and Tahir, M.M. (2016), "Shear capacity of C-shaped and L-shaped angle shear connectors", *PLOS ONE*, **11**(8), e0156989.

Tavakoli, B. and Ghafory-Ashtiany, M. (1999), "Seismic hazard assessment of Iran", *Ann. Geophys.*, **42**(6), DOI: http://dx.doi.org/10.4401/ag-3781.

Toghroli, A., Mohammadhassani, M., Suhatril, M., Shariati, M. and Ibrahim, Z. (2014), "Prediction of shear capacity of channel shear connectors using the ANFIS model", *Steel Compos. Struct.*, **17**(5), 623-639.

Toghroli, A., Suhatril, M., Ibrahim, Z., Safa, M., Shariati, M. and Shamshirband, S. (2016), "Potential of soft computing approach for evaluating the factors affecting the capacity of steel-concrete composite beam", *J. Intell. Manuf.*, 1-9.

Vamvatsikos, D. and Cornell, C.A. (2002), "Incremental dynamic analysis", *Earthq. Eng. Struct. Dyn.*, **31**(3), 491-514.

Venture, S.J. (1999), "Seismic design criteria for new moment resisting steel frame construction (50% Draft)", Report No. FEMAXXX, January.

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.

*CC*