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Determination of Seismic Performance Factors for Buildings with Concentrically Braced Frame Systems under the Excitation of Near- and Far-fault Records

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Abstract

Performance coefficients are widely used in seismic design codes to achieve performance objectives. The values of these coefficients have significant importance in achieving pre-specified performance goals. Studies have shown that near-fault earthquakes decrease the ductility and increase the risk of failure in the structures; however, the current codes use the same performance coefficients to design structures against near- and far-fault records. In the present study, 1-, 5-, 10- and 15-story special concentrically braced frame (SCBF) structures designed in the region with high seismic hazard have been evaluated. Non-linear static, linear dynamic, and incremental non-linear dynamic analyses under the influence of two sets of near- and far-fault records extracted from FEMA-P695 have been used to calculate the performance coefficients. Furthermore, the fragility curves are calculated for three performance levels (IO, LS, CP) using a probabilistic assessment of the results derived from incremental dynamic analysis to investigate the relationship between obtained factors with the probability of exceedance from a specified level. According to the mean results of all records, the behavior factor for the steel special concentrically braced frame is 5.92. The mean behavior factor obtained for the near-fault records is 35% less than the far-fault records. Differences in the obtained behavior factor for structures under excitation of two types of earthquake records (near- and far-fault) are observed in the fragility curves related to the probability of exceedance from CP level. However, there is no significant correlation between the resulted behavior factors and the probability of exceeding IO and LS levels.

Keywords

non-linear dynamic analysis, near-fault earthquake, behavior factor, steel X-braced frame, fragility curve

1 Introduction

Structures may be allowed to enter the non-linear region under severe earthquakes so that the structure can absorb and dissipate the input energy by plastic deformations [1]. Nonlinear time history (NTH) analysis is the most accurate method to predict the seismic demands of structures; however, the results strongly depend on the selected ground motions and the scaling procedure [2]. On the other hand, equivalent linear analysis is applied in the codes to simplify the design procedure of structures and obtain the required forces of a structural component designed against seismic loads. Therefore, performance coefficients such as behavior factor and over-strength factor proportional to the seismic resistance system are presented to correlate the linear analysis results with the actual non-linear behavior of structures under the seismic loads. As these coefficients values are determined accurately, determining the required structural strength to achieve the minimum damage against design-based level earthquakes and preventing from collapse under rare earthquake is more accurate. The initial values proposed for these coefficients were based only on limited experiences and engineering judgment, and no analytical or numerical process was presented to calculate these coefficients as a symbol of physical reality [3]. In recent decades, more precisely, efforts have been made to obtain this factor and other performance coefficients. The first attempts to devise a calculation method of behavior factor are in the late 1970s. Since then, various methods have been developed to evaluate performance coefficients, and more parameters have been considered in their calculation. The results of these investigations are presented as a set of performance coefficients for different structural systems in the codes.

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However, some effective parameters, such as the difference in the effect of near- and far-fault earthquake records, have not been considered in deriving these factors in the codes. Unlike far-fault earthquakes, where cyclic loading often influences structural behavior under their excitation, the structure behavior under near-fault earthquakes is characterized by pulsed loading. For this reason, pulse-like near-fault earthquakes decrease the ductility and increase the risk of failure in the structures [4]. The difference in performance coefficients calculated for structures under two sets of near- and far-fault earthquake records is also mentioned in the FEMA-P695 [5]. Some codes [6–10] have applied the effects of near-fault earthquake records by presenting a series of amplification factors in the relationships of the code design spectrum. Given that the seismic requirement of different structural systems against near-fault earthquakes can be different, applying the same factor in the design spectrum to all structural systems cannot estimate this seismic requirement accurately. For this reason, one of the objectives of the present study is to consider the difference between the effects of the near- and far-fault earthquakes in calculating the performance coefficients of the studied structures.

As mentioned previously, the type of seismic resisting system is one of the effective parameters in calculating performance coefficients. Although studies on calculating performance coefficients for conventional structural systems have been carried out, more comprehensive studies on new seismic-resistant systems are needed. The special concentric brace is one of the recently introduced structural systems and is widely used to design structures against seismic loads. The non-linear behavior of this type of structural system under the strong seismic loads resulting from the yielding and buckling of the braces. On the other hand, accurate modeling of the non-linear behavior of structural components has significant importance in the accurate calculation of performance coefficients. As a result, accurate modeling of the non-linear behavior of the special concentric brace, particularly the post-buckling behavior of the braces, under the effects of reciprocal loading, can significantly impact the accuracy of evaluating this type of structural system under seismic loads. For this purpose, in the present study, a detailed numerical model of the cyclic behavior of braces has been developed using the results of laboratory studies.

Various methods have been developed to calculate performance coefficients. Generally, the primary methods proposed to calculate the performance coefficients, particularly the behavior factor, are American researchers (*R* factor) and European researchers (*q* factor). American researchers' methods generally have more straightforward foundations and theories but are also more applicable [11, 12]; while the method of European researchers has complex principles and theories and is challenging to use in practice for actual building frames, also it is impractical in some cases [11, 12]. The present study calculates the performance coefficients using incremental dynamic analysis presented in Mwafy and Elnashai [13] based on method of Yang with some modifications. Also, the fragility curves were extracted using incremental dynamic analysis results for three performance levels of IO, LS, and CP, to the probabilistic evaluation of a special concentric brace system under two different types of earthquake records, as well as to find the relationship between the behavior factor and the probability of structures damage.

The studied models, including 1-, 5-, 10- and 15-story structures with special concentrically braced frame systems designed based on the most recent seismic design codes, are located on the soil with medium shear velocity and situated in the high seismic hazard area. The considered records consist of two sets, including pulse-like near- and far-fault earthquakes, selected from the FEMA-P695.

2 Behavior factor

The first factor of performance coefficients presented in the codes is the behavior factor, which is still widely used in modern earthquake design codes. Other performance coefficients have been developed based on various parameters affecting the behavior factor.

In 1978, ATC-3-06 recommended the objective of the behavior factor, R, to reduce the values of design forces based on seismic hazard assessment of site and non-linear behavior of the structure. As mentioned previously, *R* factors were initially calculated based on limited experiments and engineering judgments.

In recent decades, efforts have been made to obtain more accurate analytical values of this factor. The first attempts to devise a method for calculating the behavior factor are related to Newmark's work. Newmark and Hall [14] performed comprehensive research on the frequency response spectrum and its effects on displacement, lateral force, and behavior factors. In this research, the range with a low and moderate frequency spectrum has almost the same deformations in elastic and inelastic analyses. In addition, in the spectrum range with very high frequency, the amount of lateral force applied to the system in elastic and inelastic conditions was approximately the same.

In 1982, Newmark and Hall [15] presented a method for creating a non-linear spectrum using a linear spectrum for one degree of freedom structures. Although this method has been developed for one degree of freedom structures, it is a great step in calculating the behavior factor of buildings. In the late 1980s, Freeman [11] and Uang [12] developed distinguished methods for calculating the behavior factor of R. One of the approaches known as the ductility factor method is the achievement of Uang [12], and the second method, known as the capacity spectrum, is the result of Freeman [11].

In the following, researchers presented different methods, considering more parameters in calculating the behavior factor. Some of these fundamental investigations and others have been applicable in this area. In the method of Nassar et al. [16], the effects of ductility, fundamental period of the structure, and strain hardening in the load-deformation model of material have been entered into the calculation of behavior factor.

Miranda and Bereto [17] conducted extensive and fundamental research on this coefficient. They divided the main parameters affecting this factor into three crucial factors: maximum bearing displacement of the system according to deformation, fundamental period of a structural system, as well as soil characteristics of a region; and in this regard, different numerical values for behavior factor are defined according to the type of soil and structure period.

Fajfar [18] has increased the range of parameters involved in the behavior factor and considered many factors such as earthquake characteristics, soil characteristics of the region, system damping, load-deformation model of material, period of the structure, and the coefficient of ductility in calculating the behavior factor.

It should be noted that the studies mentioned above have calculated the behavior factor of multi-degrees of freedom structure based on a single degree of freedom structure. Other approaches used for calculating the behavior factor of multi-degrees of freedom structure are presented more precisely by other researchers. It is referred such as Mwafy and Elnashai [13], Elnashai and Broderick [19], Kappos [20], Grecea, and Dubina [21], in which the damage limit is selected based on a damage index such as the inter-story drift ratio or the values of plastic hinge rotation.

Mahmoudi and Zaree [22] also conducted applicable research about current steel concentrically braced frame systems. This study considers the effects of ductility and overstrength, and the hysteresis curves of bracing members are investigated. The behavior factor of studied frames with respect to the number of bracing bays is estimated.

Some other researchers have used non-linear dynamic analysis to calculate the behavior factor. Karavasilis et al. [23] used a non-linear dynamic analysis to calculate the behavior factor of steel moment frame for different levels of performance. Fanaie and Ezzatshoar [24] calculated the behavior factor of gateway braces by incremental dynamic analysis. Asgarian and Shokrgozar [25] used linear and non-linear analysis to calculate the BRBF behavior factor.

Despite the extensive research about the effect of earthquake records on the behavior factor of structures, a limited number of studies have been conducted to evaluate behavior factors with considering pulse-like near-fault records.

3 Calculation of behavior factor

3.1 Basis of calculating of behavior factor

One of the remarkable approaches to calculating the behavior factor is method of Uang [12]. In Uang's method, the non-linear behavior of the structure is presented using a bilinear curve, as shown in Fig. 1. In this figure, (V_y) is yielding base shear force and (V_e) is maximum base shear force of the elastic structure. The base shear force (V_e) reduces to (V_y) due to ductility and non-linear behavior of the structure. The reduction coefficient due to ductility, R_μ is calculated from the following equation [12]:

$$R_\mu = \frac{V_e}{V_y} . \quad (1)$$

The ratio of the base shear corresponding to the structure yielding (V_y) to the base shear corresponding to the first plastic hinge occurrence in the structure (V_s) is the overstrength coefficient, defined as [12]:

$$R_s = \frac{V_y}{V_s} . \quad (2)$$

Finally, the behavior factor value is calculated from the Eq. (3) [12]:

$$R = R_\mu \times R_s . \quad (3)$$

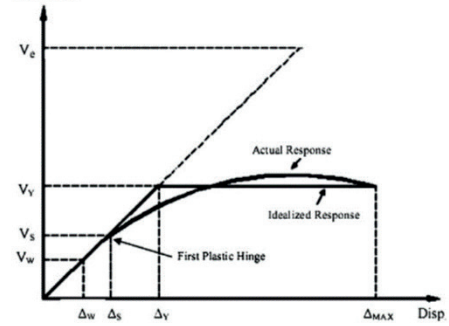


Fig. 1 Base shear–roof displacement relationship of a structure [24]

3.2 Calculating behavior factor by incremental non-linear dynamic analysis

In the following, the method of using the non-linear incremental dynamic analysis and linear dynamic analysis to calculate the behavior factor based on Uang's method is presented.

In this method, developed by Mwafy and Elnashai [13], a non-linear incremental dynamic analysis is used to obtain the maximum base shear, and the ratio of the ultimate base shear, $V_{b(Dyn,u)}$ to the equivalent base shear of the first yielding, $V_{b(Dyn,y)}$ is introduced as over-strength factor. To obtain $V_{b(Dyn,u)}$, the spectral acceleration of the earthquake record at fundamental period of structure is increased up to the step that mechanism (instability) occurs, or damage limit of collapse happens in the structure. The acceleration that results in the mechanism formation is accepted as the ultimate limit, and its equivalent base shear is obtained [13].

$V_{b(Dyn,y)}$ is the base shear equivalent to the spectral acceleration at fundamental period of structure that causes the formation of the first plastic hinge at just one point of the whole structure. If there is a gradual increase in spectral acceleration, only one plastic hinge at a time in the structure may form. And if the spectral acceleration increases to the point where the second hinge is about to form, there is still only one plastic hinge in the whole structure. It cannot be accurately stated how much of the spectral acceleration of the first mode causes the first yield in the structure, and the corresponding base shear cannot be calculated. Accordingly, the method presented by Mwafy and Elnashai [13] is modified in accordance with the results obtained based on Masoumi [26]. In the procedure stated by Masoumi, the equivalent shear of the first plastic hinge formation resulted from non-linear static analysis, $V_{b(st,y)}$, is used as the base shear of the first yield in the structure to determine the over-strength factor. This means that the ending of the linear area in the push-over curves of a structure, which is equivalent to the formation of the first plastic hinge in the structure, can be considered as the same point for nonlinear static and dynamic analyses [26]. Thus, the over-strength factor can be obtained as the ratio of the dynamic base shear that results in the formation of a mechanism (instability) in the structure or causes the structure to reach the damage limit of collapse, $V_{b(Dyn,u)}$, to the static base shear equivalent to the formation of the first plastic hinge, $V_{b(st,y)}$, in the structure according to Eq. (4).

$$R_s = \frac{V_{b(Dyn,u)}}{V_{b(st,y)}} \tag{4}$$

To obtain the behavior factor due to the ductility, a dynamic analysis is performed by assuming the elastic behavior of the structure under dynamic spectral acceleration at the fundamental period of structure leading to the mechanism formation (instability) in the structure or causing the structure to reach damage limit of collapse, and maximum linear base shear, $V_{b(Dyn,el)}$ is calculated. Thus, the behavior factor resulted from ductility, R_μ, is obtained using the results of the incremental dynamic analysis as well as a linear dynamic analysis according to the Eq. (5):

$$R_\mu = \frac{V_{b(Dyn,el)}}{V_{b(Dyn,u)}} \tag{5}$$

Finally, the behavior factor of studied frames under the selected earthquake records is calculated using Eq. (5). In the present study, the mean base shear values of seismic records were used to calculate the performance coefficients. To compare between far- and near-fault records, the mean base shear values for two sets of records are calculated separately. Finally, the mean of the performance coefficients obtained for two sets of records is presented as the overall coefficient.

4 Damage states

Various codes, such as ASCE/SEI 41-06 and rehabilitation standards [27], have suggested different criteria for determining different damage levels in structures. In this study, the concept of inter-story drift ratio has been used as damage criteria. In this way, damage levels are divided into two categories:

4.1 Drift of stories

The maximum inter-story drift ratio corresponding to the damage levels in ASCE 41-06 is in Table 1 [27].

4.2 Instability and structure mechanism

Suppose there is instability during non-linear dynamical analysis or exceeding of force values from the control force values in the force-control components. In that case, the maximum base shear values obtained from the last scaled earthquake are selected as the ultimate limit state.

Table 1 Building performance levels for steel braced frame structure according to ASCE 41-06

Performance Levels	Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
Drift	0.5%hs	1.5%hs	2%hs

5 The studied models

In this study, the structures consist of 1-, 5-, 10-, 15-stories frames with steel special concentrically braced frame systems that are designed firstly in the form of three-dimensional models and on the soil type II corresponding to Iranian 2800 standard (medium shear velocity) [6] to correspond more precisely with the behavior of actual existing structures. Then, by extracting one of the frames from each structure, the models are created in a two-dimensional state to analyze and obtain behavior factors. In each structure, stories height equals 3.2 meters, and each frame has three bays.

The frames distance is assumed to be five meters. The bracing system in both directions is placed symmetrically in the middle bays. The plan view and location of the braces are shown in Fig. 2. It should also be noted that beam to column connections and the connections of brace to beam-column intersections are pinned, and the connection of the columns to each other along structure height is fixed. The designed sections for columns, beams, and braces are box, I-shape, and double-channel, respectively.

Beam and column elements have the characteristics of St 37 steel. Dead load is 4.4 kN/m^2, and live load is 3.50 kN/m^2. The live load of the roof is 1.50 kN/m^2. The exterior wall load is 7 kN/m^2. Seismic lateral loading for designing the studied models is calculated based on Iranian 2800 standard.

The design base acceleration and building importance coefficient are $A = 0.3$ g and $I = 1$, respectively. The behavior factor for designing structures is assumed to be $R = 5.5$. The redundancy factor equals 1.2 based on Iranian 2800 standard. The AISC360-10 [28] are used to control the design criteria for steel structures.

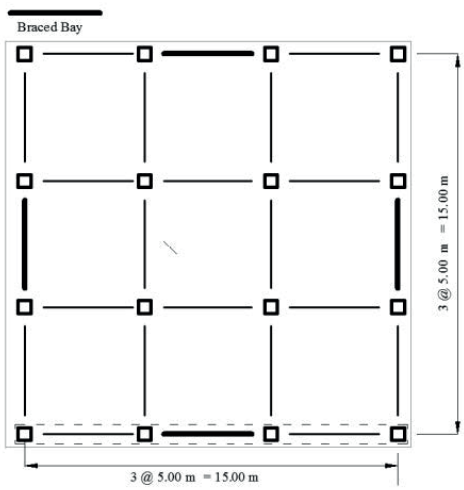


Fig. 2 Plan view of structures

6 Non-linear modeling

Perform 3D [29] was used to conduct non-linear analyzes. The non-linear modeling of steel components in this study is considered based on the backbone (Push) model presented in ASCE 41-13 [30] (Fig. 3). Values of a and b are directly derived from ASCE 41-13. The curve presented in Fig. 3 is also assumed to be elastic-perfectly plastic. The simple steel bar element of bar/tie/strut type with buckling steel material is used to model the braces.

The strength and stiffness degradation in Perform 3D software are compared with the experimental results shown in Fig. 4 [31]. This hysteresis curve follows the backbone curve (push) shown in Fig. 3. Also, at two ends of the braces, the rigid connection area is considered 5% of the total length of the brace. The tensile and compressive strength of the braces is calculated according to ASCE 41-13 [30]. The post-buckling compressive strength is calculated based on the bracing member's slenderness. According to ASCE 41-13 [30], the effective length factor is half the total length of the brace (excluding the length of the rigid connection area). The effect of tension stretch due to the increment of buckling deformations in a cycle is considered using a stretch factor equal to 0.05.

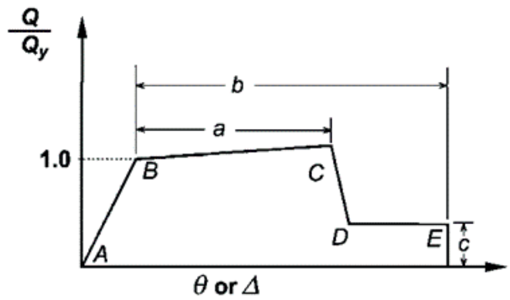


Fig. 3 Force - Displacement curve in ASCE 41-13 [30]

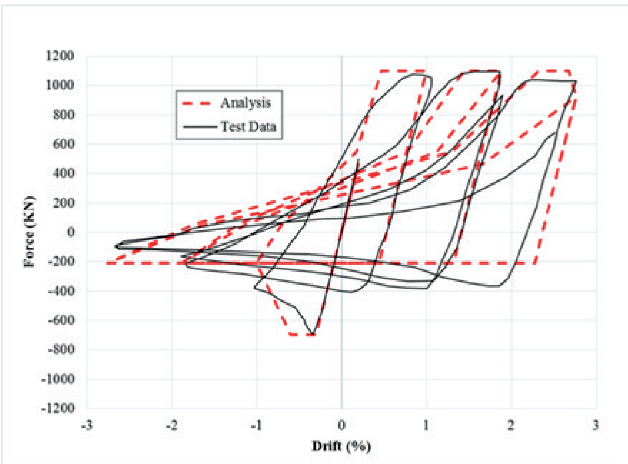


Fig. 4 Brace model calibration using test data from Fell et al. [31]

The non-linear behavior of the beams and columns is also considered to be concentrated plastic hinges at two ends of the element.

The mass in the structures is concentrated at the end nodes of column element in a two-dimensional model. The Rayleigh model with 5% critical damping has been used for considering damping in linear and non-linear dynamic analysis. Furthermore, P-delta effects are considered in linear and non-linear analyses.

7 Ground motions

The non-linear response requirements of near-fault earthquakes are more significant than those of far-fault earthquakes. In some cases, the designed spectrum is modified to take into account the near fault earthquakes effects; however, studies have shown that applying methods such as Root Sum of Squares and absolute sum of values for predicting the non-linear response of structures under near fault records in dynamic response spectrum may result in non-conservative results [32, 33]. Limited studies have been carried out on the effects of near-fault records on the behavior factor of structures [34].

Due to the high seismic demand of near-fault earthquakes, structures designed following the ordinary base forces presented in current seismic codes cannot provide near-fault effects. Therefore, the necessity of investigation and recognition of near-fault records and incorporation of these records effects in the seismic codes and improvement of the structure's capacity for high seismic needs from near-fault earthquakes were research issues in recent decades. Given that the seismic requirement of different structural systems against near-fault earthquakes can be different, applying the same coefficient in the design spectrum to all structural systems cannot estimate this seismic requirement accurately. Therefore, the impact of near-fault earthquakes on the design of different structural systems must be considered in their performance coefficients.

To compare the behavior factor of the studied structures, seven pulse-like near-fault records and seven far-fault records have been used for linear and non-linear time history analysis. Characteristics of the records are presented in Tables 2 and 3. Records have been selected from the FEMA-P695 [5].

The selected seismic intensity measure in this paper is the 5% Damped Spectral Response Acceleration at the fundamental period of the structure [$S_a(T_1,5\%)$]. This parameter is suitable because the vibration of the structures with short and medium heights is dominant in the first mode [35].

Table 2 Puls-like near-fault accelerograms

Name	Station	M	Year	PGA (g)	Epicentral Distance(km)
Cape Mendocino	Petrolia	7.1	1992	0.63	4.5
Chi-Chi	TCU102	7.6	1999	29	45.6
Kocaeli	Izmit	7.5	1999	0.22	5.3
Landers	Lucerne	7.3	1992	0.79	44
Loma Prieta	Saratoga-Aloha	6.9	1989	0.38	27.2
Northridge-01	Sylmar-Olive View	6.7	1994	0.73	16.8
SuperStatition Hills-02	Parachute Test Site	6.5	1987	0.42	16

Table 3 Far-fault accelerograms

Name	Station	M	Year	PGA (g)	Epicentral Distance(km)
Chi-Chi	TCU045	7.6	1999	0.44	10
Friuli	Tolmezzo	6.5	1976	0.35	15
Hector Mine	Hector	7.1	1999	0.34	10.4
Kobe	Nishi-Akashi	6.9	6.9	0.51	7.1
Kocaeli	Arcelik	7.5	1999	0.22	10.6
Manjil	Abbar	7.4	1990	0.51	12.6
Northridge	Beverly Hills-Mulhol	6.7	1994	0.52	9.4

An algorithm should be used to scale up the seismic intensity measure for incremental dynamic analysis. For this purpose, the search and completion algorithm Hunt and Elephant [36] have been used. In this method, in the first step for the scale of the seismic intensity criteria, a very small amount for the seismic intensity parameter (spectral acceleration of the first mode), which involves the linear response of the structure, is selected.

Then, in the following steps, the seismic intensity increases progressively according to Eq. (6) in each step. The value of the α coefficient in this study is equal to 0.05.

$$S_a\left(T_1,5\%\right)_i = S_a\left(T_1,5\%\right)_{i-1} + \alpha \times \left(i-1\right), \tag{6}$$

where i is step number and $S_a(T_1,5\%)$ is 5 % damped spectral response acceleration of ground motion at fundamental period of structure.

8 Results

8.1 Non-linear static analysis

Nonlinear static analysis so-called pushover analysis has been developed in seismic design codes [30] in the last two decades as a practical tool in seismic design and in recent years several advanced pushover methods have been proposed considering the higher modes and changes in vibration characteristics during inelastic response [37–39]. However, these advanced pushover methods have not yet been included in seismic design codes, so in this research, the conventional nonlinear static procedure based on the first mode is used, assuming that the responses of the low and mid-rise buildings are not affected much by higher modes.

Push-over curve of frames with lateral load pattern based on the first mode of structures are presented in Fig. 5. The static base shear, equivalent to the formation of the first plastic hinge in the structure, is extracted from these graphs.

8.2 Incremental nonlinear dynamic analysis

In Fig. 6, the results of the dynamic incremental analysis of structures based on the maximum inter-story drift against the spectral acceleration corresponding to the first vibration mode of the structure are presented. As shown, structures under near-fault earthquakes enter non-linear regions with less spectral acceleration than far-fault earthquakes. Also, increasing the structure's height reduces the spectral acceleration corresponding to the specified damage limit for both kinds of records.

It is worth mentioning that the weak story behavior is one of the main reasons for the failure of the studied frame, especially under high seismic hazard level ground motions. As a result of weak story behavior, the damage is concentrated on the weak story, leading to a decrease in ductility and, consequently, behavior factor. Some methods have been proposed to enhance the designs by preventing the occurrence of weak stories [40, 41]; however, the primary purpose of this article is to calculate the performance coefficients of the concentrically braced frame designed according to the current seismic design approach by taking into account all probable failure mechanisms.

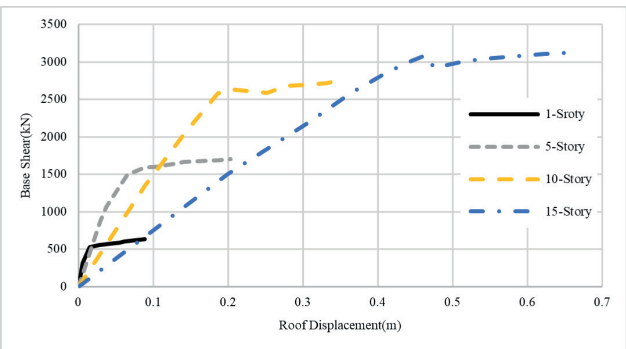


Fig. 5 Push-over curves

It would be the next step to design the SCBf based on new approaches to verify that the frame will achieve the desired level of ductility under near and far field earthquakes.

8.3 Dynamic linear analysis

The maximum elastic base shear resulted from linear dynamic analysis $V_{b(Dyn,el)}$ is obtained under the scaled records based on the collapse prevention limit state determined by the incremental dynamic analysis for near- and far-fault records. Increasing the structure's height reduces the base shear ratio of near-fault to far-fault records.

8.4 Seismic performance factors

Based on the obtained results, the values of the overstrength coefficient, the behavior factor resulting from ductility are calculated and presented for the near- and far-fault records in Tables 4 and 5. The mean values of the behavior factor obtained for structures under far-fault records are greater than the values of near-fault records. These results indicate a high demand for the structure under near-fault records so that if structures were designed against the near-fault records, they need to consider the greater design base shear. This requirement for steel special concentrically braced frame under near-fault records is 56%, more than far-fault records.

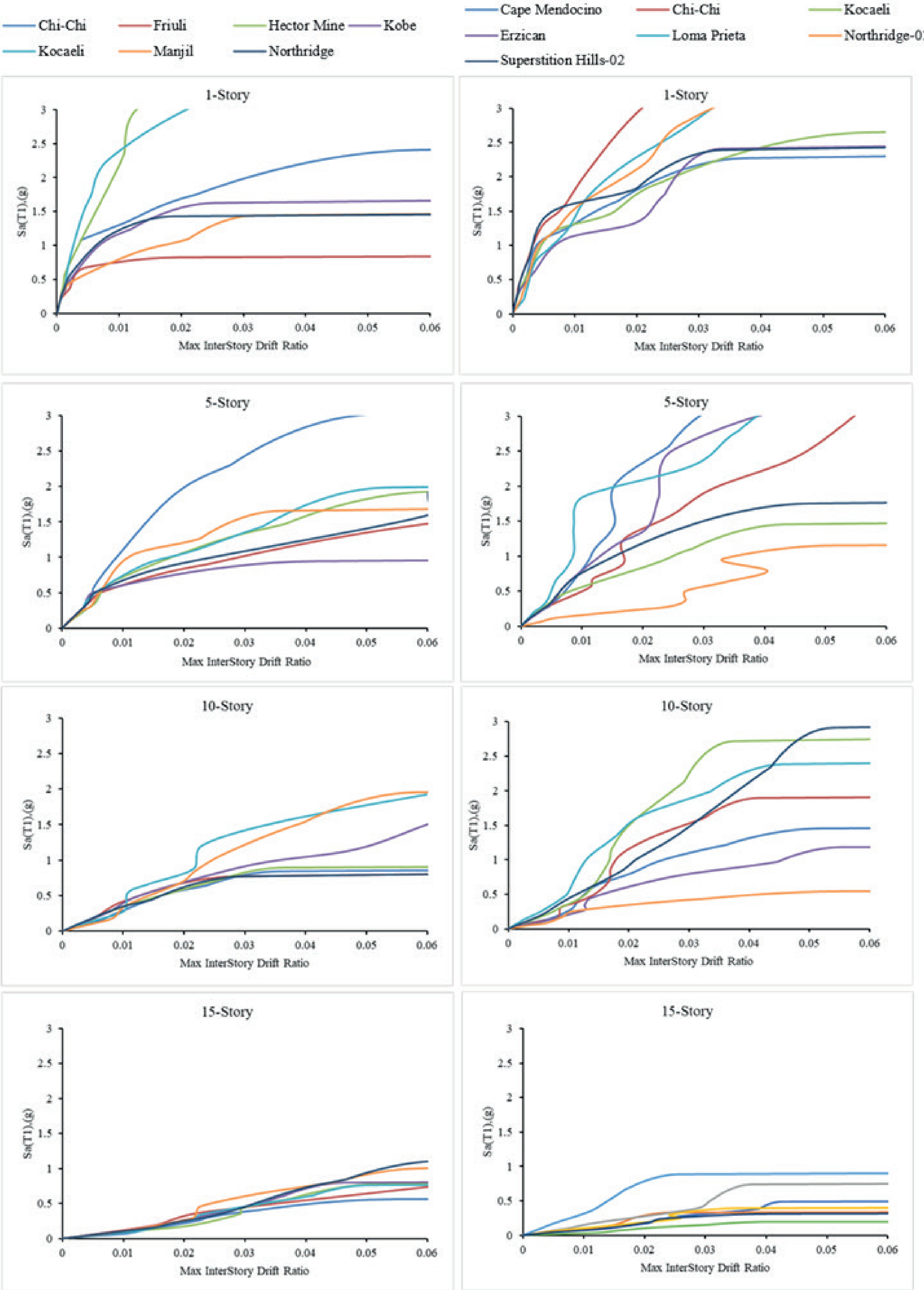
The mean values of the overstrength coefficient for two sets of records are close. However, with increasing the frame height, the frame's behavior factor reduces. This change decreases the behavior factor due to the steel structure ductility with increasing height.

Table 4 Seismic Performance Factors (near-fault records)

Near-Fault Record			
No. of Story	R_s	R_μ	R
1	1.94	2.67	5.18
5	1.87	2.67	4.99
10	1.80	2.81	5.06
15	1.77	1.85	3.27
Average	1.85	2.50	4.63

Table 5 Seismic Performance Factors (far-fault records)

Far-Fault Record			
No. of Story	R_s	R_μ	R
1	2.04	4.71	9.61
5	1.91	4.00	7.64
10	2.16	3.53	7.62
15	2.39	1.64	3.92
Average	2.13	3.47	7.20



(a) Near-fault Records (b) Far-fault Records

Fig. 6 Non-linear Incremental Dynamic Analysis results of 1-5-10 and 15 story Structures (a) Near-fault (b) Far-fault

In the behavioral curve of short structures with more stiffness and less period than those with taller buildings, the gradient of the elastic zone is also greater, and these structures achieve their yielding point or the same yielding deformation in lesser lateral displacements; while high-rise structures tolerate more displacements until they reach their yielding point. Considering that in the pushover curve of the structure, the damage state is assumed to be constant, therefore, with increasing the height of the structure, the ductility decreases, which results in a decrease in the behavior factor due to the ductility and behavior factor of the structure.

In all structures, the behavior factor under the near-fault record is less than that of far-fault records. This difference is more in single-story structure. This difference is more due to the significant difference in the behavior factor due to the ductility in the one-story structure under the far- and near-fault records. In the pulse-like near-fault records, the input energy releases quickly, and doesn't allow the bracing system to dissipate the earthquake input energy; hence, this reduces the ductility and consequently decreases the behavior factor.

8.5 Fragility curves

In seismic design codes, various criteria are used to achieve predetermined objectives. For example, in ASCE 07-16, the objective of the seismic design of structures classified at risk levels 1 and 2 is to achieve the collapse probability of less than 10% under maximum considered earthquake (MCE, 2% / 50 Years) [42]. Also, the objective of Iranian 2800 standard for structures with intermediate importance is to seismic design structures that have not suffered major structural and non-structural damage under design level earthquakes (10% / 50 Years) and have minimal casualties. To achieve these goals, the values of the performance coefficients must be correctly determined and included in the seismic loading calculations. Therefore, one way of accurately assessing the values of performance coefficients in the seismic codes is to examine the probability of exceeding different performance levels of designed structures based on those codes.

In the present study, the behavior factor presented for the structure with special concentrically braced frame system in Iranian 2800 standard has been evaluated by developing fragility curves for two sets of far- and near-fault records. Iranian 2800 standard does not provide a specific probability of exceeding the intended performance level of this code (life safety) and only mentions the minimization of casualties. The present study considered 10% probability of exceeding the performance level of life safety as a quantitative measure of this regulation.

Also, to evaluate the relationship between the obtained behavior factor (with the method presented in the present study) for far- and near-fault records and probability of exceeding structures from different performance levels, the fragility curves for three IO, LS, and CP performance levels were derived. Thus, the results of the fragility curves can be used for probabilistic evaluation of the studied structures under two different types of earthquake records. In the present study, fragility functions are extracted from the parameters obtained from IDA curves. The Log Normal Cumulative Distribution Function is used to define the fragility function of IDA, as suggested by Baker and Cornell [43].

$$P\left(PL \mid IM = x\right) = \Phi\left[\ln\left(x / m_R\right) / \beta_R\right] \tag{7}$$

Where $P(PL \mid IM = x)$ is equal to the probability that an earthquake with $IM = x$ (seismic intensity criterion) will exceed the desired performance level (e.g., PL: IO, LS, CP); $\Phi()$ is the standard normalized cumulative distribution function (CDF); m_R is median of the fragility function (IM level with 50% probability of exceeding the given PL), and β_R is logarithmic standard deviation. Therefore, to calibrate the above equation for the studied structures, the results of non-linear dynamic analysis are used to estimate the parameters m_R and β_R . In the present study, m_R and β_R measure the aleatoric uncertainty in the seismic capacity of the structure. Sources of uncertainty additional to the above in capacity estimation include epistemic uncertainties (based on knowledge) that comes from the formulated hypotheses of analysis and because of database limitations. To consider both aleatoric and epistemic uncertainties, the value of β_R given in Eq. (8) is replaced by the square root of the sum of squares β_{RR} and β_{RU}, where β_{RR} is the aleatoric component of uncertainty and β_{RU} is modeling uncertainty (epistemic). The relation used to define β_{RR} equals as follows:

$$\beta_{RR} = \left(\beta^2_{D(D|Sa)} + \beta^2_C\right)^{0.5}. \tag{8}$$

Where β_C is indeterminacy in capacity and depends on the PL; and $\beta_D(D|S_a)$ indeterminacy seismic demand. In the present study, the values employed by Ellingwood et al. [44] for β_C and β_{RU} are assumed to be $\beta_C = 0.25$ and $\beta_{RU} = 0.20$. Indeterminacy sources are not considered in the structural characteristics of materials. In order to calculate the indeterminacy seismic demand, $\beta_D(D|S_a)$, N produces pairwise values $\{(IM_i, \delta_{max,i}), i = 1, ..., N\}$ for each defined performance level (i.e., IO, LS, CP) defined by the IDA curves. In this paper, N is the number of selective earthquakes. Due to the higher non-linear nature and high response dispersion caused by record to record variation, non-linear regression analysis using Eq. (9) was used to estimate $\beta_D(D|S_a)$.

The logarithmic transformation of Eq. (9) is converted to the form presented in Eq. (10); where a and b are constant, which can be obtained by using simple linear regression analysis:

$$\delta_{max} = aS_a^b , \tag{9}$$

$$\ln\delta_{max} = \ln a + b\ln S_a . \tag{10}$$

Accordingly, the obtained fragility curves are presented in Fig. 7. As you can see, in 1-story structure, the probability of exceeding all three damage levels in all seismic intensities for the near-fault records is more than the far-fault records. In the 5-story structure for damage state of IO, the structural performance under near-fault records is better than far-fault records. However, in this structure, the fragility curve for the damage state of CP under far-fault records is below the near-fault records. At the LS performance level for the 5-story structure, the fragility curves of the two sets of records are close to each other. In the 10-story structure, the probability of exceeding the LS and CP levels under far-fault records is less than the near-fault records, and at the damage state of IO, this probability is close to each other for the two sets of records. In the 15-story structure, similar to 1-story structure, the probability of exceeding all three damage levels in all seismic intensities for near-fault records is more than far-fault records.

To better understand the results of the fragility curves, the probability of exceeding the performance level specified for the spectral acceleration corresponding to the hazard levels of 2% / 50 years, 10% / 50 years, and 50% / 50 years are given in Table 6. The spectral acceleration corresponding to the 10% / 50 years hazard level equals design spectral acceleration during the first period of the structure. Spectral acceleration of 2% / 50 years and 50% / 50 years are 1.5 and 0.5 times the spectral acceleration of 10% / 50 years, respectively. Table 6 indicates that the probability of exceeding or reaching a given damage state for the 50% / 50 years hazard level is less because its excitations level is low. Furthermore, the probability of exceeding or reaching damage states for all hazard levels in the 15-story structure is greater than the other structures, so the probability of exceeding damage state of CP for a 15-story structure at a hazard level of 2% / 50 years is greater. Except for damage state of IO in the 5- and 10-story structures, at the remaining damage states and hazard levels, the probability of exceeding structures under near-fault records is more than far-fault records.

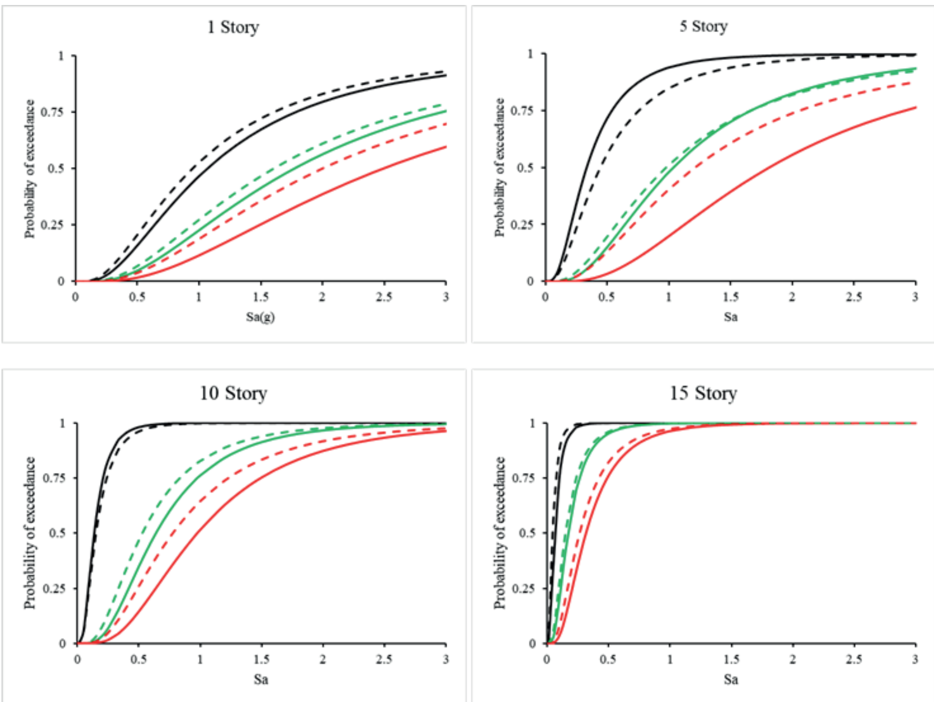


Fig. 7 Fragility curves

In general, the greatest difference between the far- and near-fault records is the probability of exceeding damage state of CP, which decreases with increasing height. Also, by comparing the results of the fragility curves and behavior factors, it can be concluded that the difference in the behavior factor obtained for the structures under two types of near- and far-fault earthquake records are observed as the difference in the probability of exceeding the structures from the damage state of CP in fragility curves so that the behavior factor obtained for the structure under the far-fault records is more than the near-fault records. As a result, the fragility curve at the CP level for far-fault records was lower than the near-fault records. However, there is no significant relationship between the resulting behavior factors and the observed difference in the probability of exceeding other damage states under two sets of earthquake records.

The obtained values for the probability of exceeding from the performance level of life safety at the design earthquake hazard level show that 1-, 5-, and 10-story structures with special concentrically braced frame systems designed according to the Iranian 2800 standard with behavior factor equal to 5.5, somehow were able to achieve the design goal under the far-fault records, so that the mean of this probability for three mentioned structures are 16%. Also, the mean probability of exceeding the performance level of life safety at the design earthquake hazard level for these structures under the near-fault earthquake record is 22%, which is 6% higher than the far-fault record.

The probability of exceeding the performance level of life safety at the design earthquake hazard level for the 15-story structures under far- and near-fault earthquake records are 73% and 74%, respectively, indicating that with an increment of structures height to 15 stories, this probability significantly increases for both sets of records.

According to the results obtained for structures with special concentrically braced frame systems, with different stories number, in this section, and the behavior factors obtained in Sections 4–5, it can be concluded that providing a constant behavior factor for structures with different heights has abundant approximation different heights has abundant approximation, and this distinction needs to be considered in the codes.

Table 7 presents the probability of exceeding the life safety performance level at the design earthquake hazard level along with the corresponding behavior factors for the studied structures. As can be seen, although the behavior factors obtained for 1-, 5-, and 10-story structures under far-fault records are higher than the initial design behavior factor (5.5), the probability of exceeding the performance level of life safety at the design earthquake hazard level indicates that these structures must be designed for lower behavior factor to achieve the desired performance level of regulation.

This indicates that the behavior factor obtained from conventional methods cannot estimate the actual need of structures to achieve the desired performance level of regulation. Therefore, the differences in the obtained results

Table 6 Damage probabilities for different hazard levels

No.Story	Record Type	50% / 50 yrs			10%/ 50 yrs			2% / 50 yrs		
		IO (%)	LS (%)	CP (%)	IO (%)	LS (%)	CP (%)	IO (%)	LS (%)	CP (%)
1	Far	9	2	1	32	13	6	53	31	17
	Near	12	3	2	38	17	11	59	36	26
5	Far	43	4	0	79	21	6	92	41	15
	Near	30	6	4	64	26	18	81	45	35
10	Far	61	2	0	92	14	4	98	34	14
	Near	54	4	1	87	24	10	96	46	25
15	Far	82	27	6	98	73	36	100	86	55
	Near	92	36	12	99	73	42	100	89	65

Table 7 Probability of exceedance from Life Safety damage state

Records Type	Far Fault					Near Fault				
No. of Story	1	5	10	15	Average	1	5	10	15	Average
R	9.60	7.69	7.65	3.96	7.23	5.19	5.04	5.07	3.28	4.65
Probability of exceedance from LS (10%/ 50 yrs)	13	21	14	73	30	17	26	24	73	35

for far- and near-fault records in both parts of the behavior factor calculation and probabilistic evaluation, as well as the different results for structures with various heights in these two sections, indicate a high approximation in using a unique behavior factor for structures with special concentrically braced frame systems differ from site and geometry specifications. Also, the results show that the behavior factor used in the seismic design codes, to consider the non-linear effects, cannot satisfy the final performance goals of the codes, and non-linear and probabilistic analyses should be added to the codes ensure that the goals are met.

7 Conclusions

This paper evaluates the behavior factor, the behavior factor caused by ductility, and the overstrength coefficient of the steel special concentrically braced frame system under the influence of near- and far-fault records. To calculate the behavior factor of this system, non-linear static, linear dynamic, and non-linear incremental dynamic analyzes have been used. The earthquake records, including seven pulse-like near-fault and seven far-fault records, have been selected from the FEMA-P695. Also, the fragility curve of the structures has been extracted for three performance levels of IO, LS, and CP.

Regarding the studied models and the type of soil considered with mean shear velocity, the results obtained for medium-rise structures in areas with high seismic hazard level and semi-soft soil can be cited. The summary of the results is as follows:

1. The overstrength coefficient for the near-fault record equals 1.85, and for the far-fault record is 2.13. On average, for all records is 1.99. This coefficient is considered 2 in Iranian 2800 standard and ASCE 7-16.
2. The behavior factor due to ductility for near- and far-fault records equals 2.50 and 3.47, respectively. On average, for all records is 2.99.
3. The behavior factor for near- and far-fault records equals 4.63 and 7.20, respectively. This factor in Iranian 2800 standard and ASCE 7-16 equals 5.5 and 6, respectively.
4. With increasing structure height, the behavior factor of the steel concentrically braced frame decreases.
5. Required values for design base shear based on the obtained behavior factor for the steel special concentrically braced frame for the near-fault records is 56% greater than the far-fault records.
6. Differences in the behavior factor obtained for structures under two types of near- and far-fault earthquake records is corresponding to the probability of exceeding structures from the CP damage level in fragility curves. But there is no direct relationship between the obtained behavior factor and other damage levels.
7. The behavior factor used in Iranian 2800 standard, to consider non-linear effects in linear analysis, cannot meet the code's final performance goals, and non-linear and probabilistic analyses should be added to the codes to ensure that the goals are met.

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