Evaluation of a Seismic Collapse Assessment Methodology Based on the Collapsed Steel Buildings Data in Sarpol-e Zahab, Iran Earthquake

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Received: 25/07/2018

Accepted: 29/10/2018

ABSTRACT

The collapse evaluation of the seismically vulnerable structures is very important in any earthquake risk reduction program. There are several analytical methods currently available to assess the collapse capacity of structures under earthquake ground motions. Severe earthquakes in cities provides a unique opportunity to evaluate the effectiveness of the seismic collapse assessment methods. On November 21, 2017, an earthquake with the moment magnitude of 7.3 and the PGA of 0.69 g occurred in about 37 kilometers northwest of Sarpol-e Zahab region (Kermanshah, Iran). This earthquake caused the collapse of significant numbers of low and mid-rise steel structures. In this paper, an attempt is made to examine the efficiency of an approximate incremental dynamic analysis (IDA) method to estimate the collapse capacity of conventional steel structures. To this purpose, two partially collapsed steel structures are selected. Both two structures are comprised of an ordinary moment resisting frame system in one direction, and a braced frame system in other perpendicular direction. The dimensions and permanent displacements of these structures have been measured on-site. These buildings are modeled in a finite element program and analyzed by modal pushover analysis in two major directions, and the SDOF models are extracted. In the next step, the SDOF models are analyzed by the IDA method under the selected earthquake records. The median and dispersion of collapse capacity of the structures are calculated from the approximate IDA results. Finally, the collapse probability of these structures is calculated under the maximum considered earthquake (MCE), determining the uncertainties based on FEMA P695 relation and engineering judgments. The results show the development of simplified and inexpensive methods for collapse assessment is crucial to be implemented to identify existing killer buildings in cities prone to major earthquakes.

Keywords:
Sarpol-e Zahab (Kermanshah) earthquake, Steel buildings; Pushover analysis; Collapse capacity; Collapse probability

1. Introduction

The past earthquake experiences show that the factor causing the greatest direct and indirect financial loss and casualties is the collapse of buildings. In seismic design regulations, the buildings are designed in a way that the structure remains stable, and the casualties are minimized [1]. In addition to causing the greatest casualties and financial loss during the earthquakes, the collapse will increase the mortality rate after the earthquake by disrupting the relief process. This will be more significant in metropolises due to their high population density. For this reason, the main concern of decision-making and management centers in earthquake consequences is the casualties and financial losses caused by the collapse of existing buildings and its socioeconomic consequences for the city and the country.

Several approaches have been proposed so far to evaluate the collapse capacity of the existing structures. These methods include various approaches, such as simplification of the entire structure to an equivalent SDOF model, step-by-step analysis of the finite element model of the entire structure to record the sudden rise of the structure response and the incremental dynamic analysis (IDA) method [2]. The occurrence of severe earthquakes in cities allows the effectiveness of these methods to be evaluated in forecasting the building collapse.

Recently, a severe earthquake occurred at 7.3 moment magnitude in Sarpol-e Zahab of Kermanshah province located in western Iran, which had a major difference with the previous earthquakes occurred in Iran. Unlike previous events in which cities with often old and non-structure buildings were affected by earthquakes, in this earthquake, the cities containing buildings with seismic resistant systems were affected. In this city, like in the other cities of Iran, a significant percentage of conventional residential and commercial buildings are mid-rise and low-rise buildings with steel structure. A significant number of these buildings has been seriously damaged in this earthquake. This paper investigated the effectiveness of an approximate seismic collapse assessment of the existing buildings in comparison with the actual performance of some conventional collapsed steel buildings in Sarpol-e Zahab city.

The IDA is the most common method used by researchers to calculate the seismic collapse capacity of structures. However, it has not been widely used by engineers due to its complexity and time-consuming. Hence, the use of methods reducing the computational complexity using simplistic assumptions has taken the attention of researchers. One of the most commonly used methods is a nonlinear static (pushover) analysis. In these methods, the structure is loading with a constant or adaptive lateral load pattern until the control point location exceeds the target displacement or the structure collapses. Many investigations have been carried out on the ability of these methods to estimate the structural responses under a specified earthquake.

Han and Chopra [3] showed that the first mode SDOF equivalent structure accurately estimates the roof displacement of the steel moment resisting frames under a specified earthquake catalog. Various studies have also shown that first mode pushover-based IDA method can be used as an appropriate alternative approach for conventional buildings that are regular in both plan and elevation [4-5]. This method requires much less time for analyzing and presenting the results. It is a suitable method for collapse capacity assessment of a large number of buildings situated in a city, and the development of its application in various structural systems is justifiable and practicable. Here, an attempt is made to evaluate this approach on two structurally collapsed steel building during Sarpol-e Zahab earthquake.

In the method used, the structure is pushover analyzed in two perpendicular directions with the load pattern in accordance to its dominant elastic mode. Then, for each mode, the equivalent SDOF structures will be determined. These equivalent structures are subjected to earthquake records, and the approximate IDA curve of each record will be obtained.

Two regular 3-storey steel buildings of Sarpol-e Zahab have been selected as case studies to evaluate the results of this method. These buildings have a seismic resistant system and experienced full or partially collapse. The collapse capacity of these structures is derived from the proposed method. Next, the uncertainties required to estimate the probability of the collapse of these structures are determined by using the proposed values of the FEMA P695 instruction and applying the engineering judgment. Then, the collapse probability of these structures is calculated under maximum considered earthquake (MCE) spectrum. Finally, the results are compared with the actual response of the structures in the Sarpol-e Zahab earthquake and the results are investigated.

2. Characteristics of Sarpol-e Zahab Earthquake

This earthquake occurred at 21:48 (local time) on

Table 1. Characteristics of the Sarpol-e Zahab earthquake.

Event Date	Location	Depth (km)	Magnitude (Mw)	PGA (g) 0°	PGA (g) 90°	PGA (g) Vertical
2017/11/12 18:18	34.77N 45.76E	18	7.3	0.69	0.58	0.34

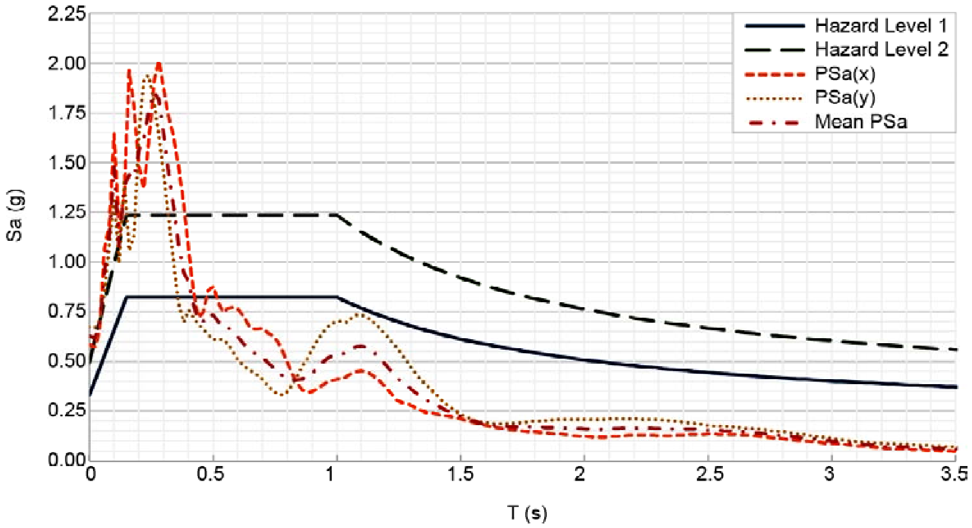

Figure 1. Design spectrum (hazard level 1) and MCE spectrum (hazard level 2) of the Sarpol-e Zahab region according to the fourth edition of 2800 code and the Sarpol-e Zahab earthquake spectra.

Aban 21, 1396, corresponding 18:18 (universal time) on November 12, 2017, with 7.3 moment magnitude in a 15 km distance from Ezgeleh and about 37 kilometers northwest of Sarpol-e Zahab city in Kermanshah Province, located on Iran-Iraq border [6]. The focal point of this earthquake was determined by the Geophysics Institute of Tehran University at 34.11 degrees north latitude and 41.16 degrees east longitude and at a depth of 18 km [6]. Table (1) summarizes the characteristics of this earthquake.

3. Specifications of the MCE Spectrum

The seismic collapse probability of the structure is estimated based on the assessed MCE spectrum for the building site [7]. In lieu of a seismic hazard analysis, this spectrum can be assumed about 1.5 times the design-based earthquake (DBE) spectrum, based on the Iranian Instruction for Seismic Rehabilitation of Existing Buildings [8]. These spectra are presented in Figure (1). In addition, according to the micro-zonation map presented in Figure (2), the soil of the case study buildings location is in the IV type of the Iranian Seismic Code 2800 [9]. The area's seismic hazard is determined as a region with relatively high seismic hazard in the code. The mean acceleration spectrum of Sarpol-e Zahab earthquake (derived from the geometric mean of the orthogonal two-dimensional spectra) is also presented in Figure (1).

4. Collapse Assessment Using the Approximate IDA Analysis Method

As mentioned in the introduction, in the approximate IDA method, the IDA analyses are performed on the equivalent SDOF structure resulting from the pushover analysis results. The IDA method includes a series of nonlinear dynamic time history analyses for each earthquake record so that each record would be scaled to multiple values of seismic intensity levels. Consequently, this method involves a complete range of structural behavior from elastic to non-elastic nonlinear, and finally general dynamic instability [10]. The result of these analyses is an IDA curve for an earthquake. This curve is a diagram of seismic intensity measure (IM) against the engineering demand parameter (EDP). In the technical literature, the corresponding spectral acceleration of the elastic structure's first period $S_a(T_1)$ has been widely used as the seismic intensity parameter [3]. Besides, the maximum roof drift has been used as the seismic demand parameter [3]. In this paper, the same parameters are used.

The seismic collapse capacity of the structure is

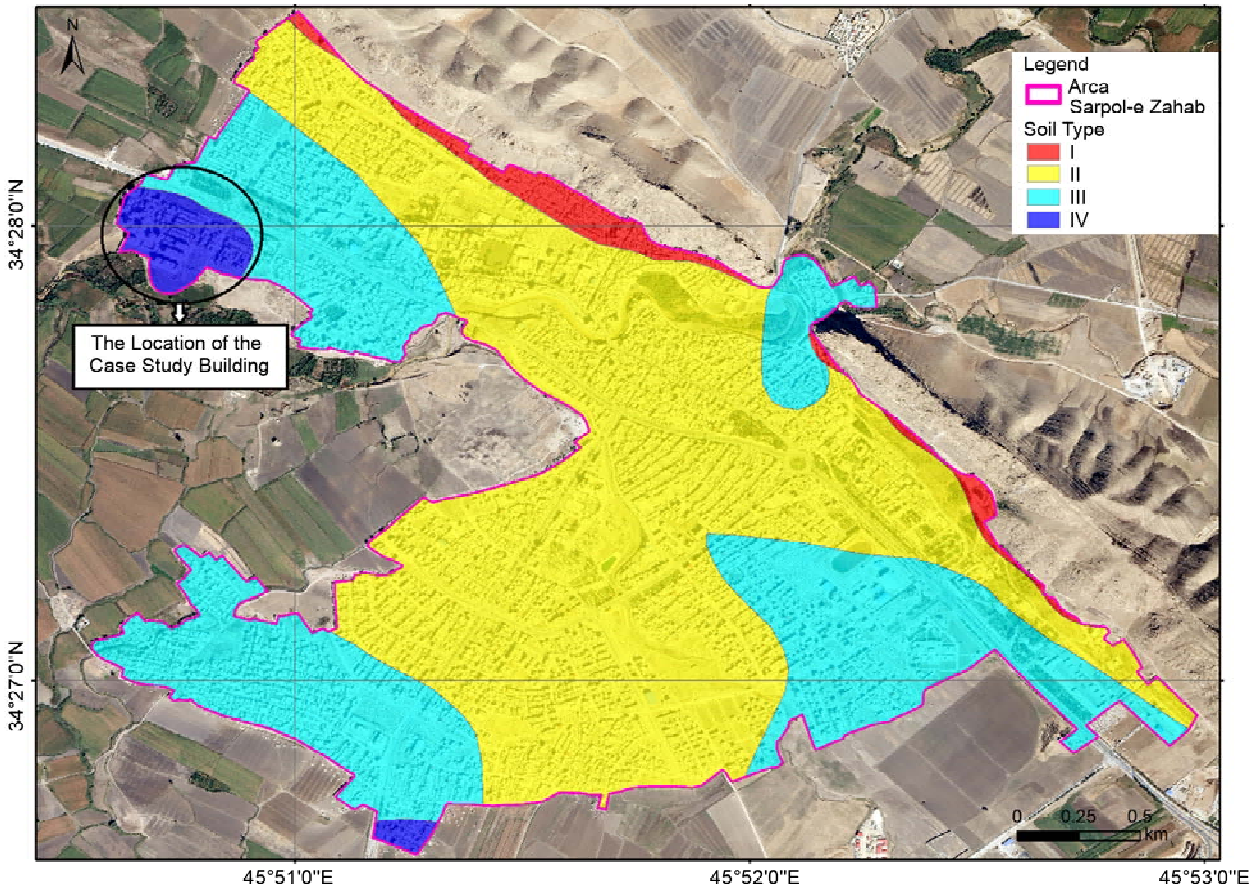

Figure 2. The micro-zonation map of the Sarpol-e Zahab based on the IIEES report [9].

estimated from the results of the IDA analysis. Structural collapse capacity is defined as the spectral acceleration $S_a(T_1)$ in which the structure gets dynamically unstable and collapses due to a deterioration in the structural stiffness and strength of components or the effects of the p-delta [2].

In the approximate method used in this paper, IDA analyses are applied to equivalent SDOF structures resulted from pushover method. The details of this method are presented step-by-step in the next section.

4.1. Approximate IDA Approach Steps

1. The software modeling of buildings is carried out in a finite element program.
2. The modal analysis of the elastic structure is performed, and the natural vibration frequencies of the structure and modal shapes of the first mode $\varphi_{1(x,y)}$ are determined in two perpendicular directions.
3. Plastic hinge parameters of each building members are assigned to the components according to the regulations ASCE 41-13 [11].
4. The corresponding loading vector of the first mode in each direction S_x, S_y is obtained according to the Equation (1), and the pushover analysis is performed based on each of these vectors.

$$S_x = \pm[M](\{\varphi_{1x}\} \pm \lambda.\{\varphi_{1y}\})$$
$$S_y = \pm[M](\{\varphi_{1y}\} \pm \lambda.\{\varphi_{1x}\}) \qquad (1)$$

The λ coefficient is considered to take into account the corresponding force to 30% displacement due to the earthquake in the orthogonal direction.

5. The transformation factor based on elastic mode is determined using Equation (2) to convert the multi-degree-of-freedom (MDOF) structure to the equivalent SDOF:

$$\Gamma_i = \frac{\sum_{j=1}^{n} m_j \varphi_{i,j}}{\sum_{j=1}^{n} m_j \varphi_{i,j}^2} \qquad (2)$$

In both equations, n is the number of stories, $\varphi_{i,j}$ is the shape of the i^{th} elastic mode in the j^{th} story, and m_j is the mass of the j^{th} story, and $i = x,y$ directions.

6. For each mode, the pushover curve in coordinate of D_{Roof}-V_{Base} is idealized by regulations ASCE 41-13 proposed method.
7. The force-deformation curve of the equivalent SDOF structure for each mode is obtained from the division of the idealized curve to the corresponding transformation factor (Γ_i). The mass of the equivalent SDOF equation is equal to $\sum_{j=1}^{n} m_j \varphi_{i,j}$.
8. The displacement corresponding to the structure collapse will be determined through pushover analysis. In the pushover analysis, the structure collapses once the pushover curve slope gets negative due to the strength and stiffness deterioration and P-delta effect. The structural collapse criteria in static nonlinear analyses are the first step in which one of the following two criteria occurs.
   8.1. The base shear in pushover curve reaches 0.8 maximum base shear [5].
   8.2. The total drift reaches 5% at the moment resisting frame direction and 2% in the bracing direction [12].
9. The equivalent SDOF structure resulted from step 7 is modeled and analyzed in OpenSEES [13] by the modified IMK (Modified Ibarra-Medina-Krawinkler Deterioration Model with Bilinear Hysteretic Response) model (Bilin material) presented by Lignos and Krawinkler [14] for a steel structure. The parameters proportional to this model are extracted from the idealized curve according to Figure (3).

The parameter Du is obtained by dividing the displacement corresponding to the structural collapse (obtained from step 8) by the corresponding transformation factor (Γ_i). This value should be revised in time history analyses to consider the effects of cyclic deteriorations. For this purpose, the approach proposed by Fajfar [15] is used in this paper. In this method, the structure deformation capacity derived from pushover analysis should be reduced to an average of 0.91 for steel structures. Therefore, the parameter D_u obtained from the equivalent SDOF structures will be multiplied by a coefficient of 0.91.

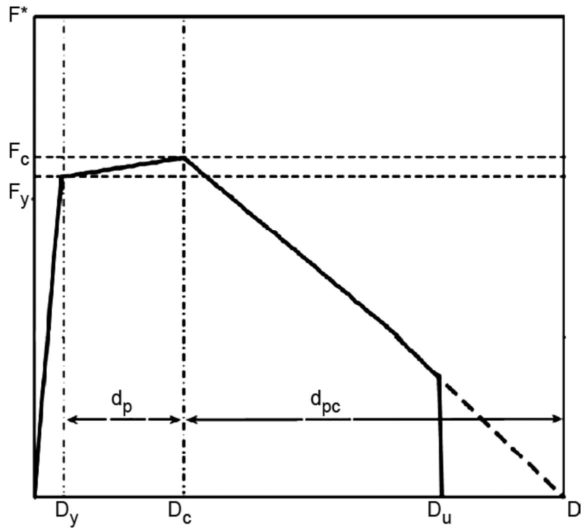

Figure 3. Apply the IMK model for SDOF structure.

10. The IDA analysis of equivalent SDOF structures is performed under specified earthquake records, and the SDOF IDA curves in the $S_a(T_1)$-D_{SDOF} coordinate will be obtained. From the multiplying D_{SDOF} to Γ_i / H, the approximate IDA curve of the original structure is achieved in the coordinate of $S_a(T_1)$-RDR in each direction (H is total building height).
11. The collapse capacity for each record will be equal to the spectral acceleration corresponding to the first step after which the maximum roof drift exceeds 5% in the bending frame direction and 2% in the bracing direction or the IDA curve slope gets lower than 20% of the initial curve slope. The structure collapse capacity (SC) will also be equal to the median of collapse capacities $S_{a-50\%}(T_1)$.
12. The required uncertainties include record-to-record uncertainty (β_{RTR}), design requirements uncertainty (β_{DR}), test data uncertainty (β_{TD}), and modeling uncertainty (β_{MDL}) determined. The total uncertainty is obtained using Equation (3). Given that there is no reference for the calculation of these parameters in Iran, the parameters are obtained based on the engineering judgment and from proposed values of the FEMA P695 instruction [8]:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \tag{3}$$

14. The collapse fragility curves will be obtained considering a log-normal distribution for collapse capacities, using the total uncertainty obtained

from the previous step. The structure collapse probability is calculated under MCE hazard level. If the collapse probability is obtained to be more than 10%, it is not acceptable, and the building is considered subject to the collapse.

5. The Case Study Buildings

Given that in this earthquake, most collapsed steel structures were buildings under 3-storey, this group of buildings was selected as target buildings. Therefore, two regular 3-story buildings with severe structural damage located in the south-west of Sarpol-e Zahab were selected. These two models have experienced significant drifts in both directions and structurally have been collapsed. Age of these buildings is estimated to be about three years. These buildings are named S-1 and S-2. Location of the buildings is provided in Figure (4).

Steel buildings in this city, often have a moment resisting and a braced frame in two orthogonal directions. The case study buildings were also executed similarly. Both of two buildings had a moment resisting frame system in one direction. In the other direction, one of the buildings had concentric X-bracing system, and another building had an eccentrically bracing system. The roof of both two buildings is block joist with clay heel.

5.1. Building S-1

This building with three stories, a residential land use and a height of about 11.15 meters is located in the southwest of Sarpol-e Zahab city in geographical coordinates of 45°50'42.2"E and 34°28'00.4"N. This building has a moment resisting frame in one direction and a concentric X bracing frame (CBF) in the orthogonal direction. The elevation and plan view of this building and its measured details are presented in Figures (5) and (6), and Tables (2) and (3).

Table 2. S-1 Building beams and columns.

	Beams		Columns
	Frame Dir	Brace Dir	
Story 1	G2	G1	According to Figure (4)
Stories 2 to 3			C1
+1.00 m	-	G1	-
Balconies	G1	G3	-

Table 3. S-1 Building structural sections.

Element	Name	Details
Beams	G1	IPE16
	G2	2IPE22
	G3	IPE22
Columns	C1	2IPE16 c/c8cm – Pl 10×20@40cm
	C2	3IPE16– Pl 10×20@40cm
Braces	BR	IPE14 End Gusset Plate: 75×35×0.8cm Center Gusset Plate: 40×40×0.8cm

Figure 4. The location of the case study buildings.

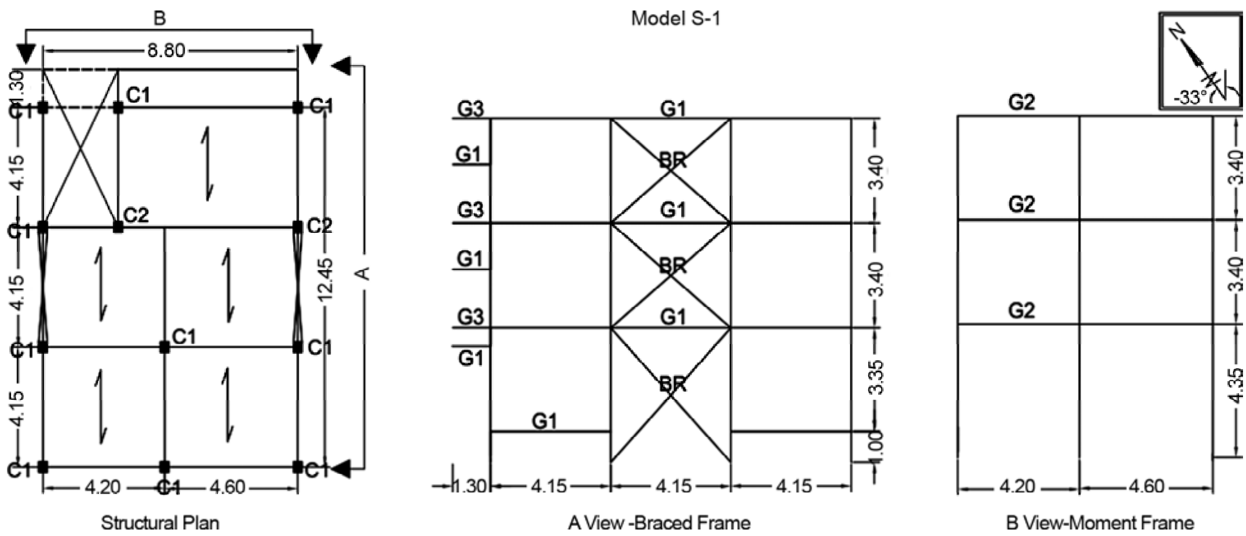

Figure 5. Elevation and plan view of S-1 building.

Figure 6. Real view of S-1 building after Sarpol-e Zahab earthquake.

5.2. Building S-2

This building, located in the southwest of Sarpol-e Zahab, has three stories, 11.45 meters' height, and residential land use. The building is located on 45°50'50.4"E and 34°27'55.7"N geographical coordinates and has a moment resisting frame in one direction and an eccentric bracing frame (EBF) in the orthogonal direction. The link beam length of EBF is 0.8 m, and according to ASCE 41-16 instruction, its plastic hinge parameters is the same as beams. The view and plan of this building and its measured dimensions and details are presented in Figures (7) and (8), and Tables (4) and (5).

6. Required Parameters

6.1. Determining the Modeling Parameters

When modeling the buildings, the specifications of the steel and the concrete used were the conventional values for buildings in Iran. The St-37 steel was assumed to have an elastic modulus of 200 GPa and yield stress of 235 MPa. The concrete was assumed to have an elastic modulus of 25 Gpa and a compressive strength of 250 Mpa. Gravity loading was performed in accordance with the

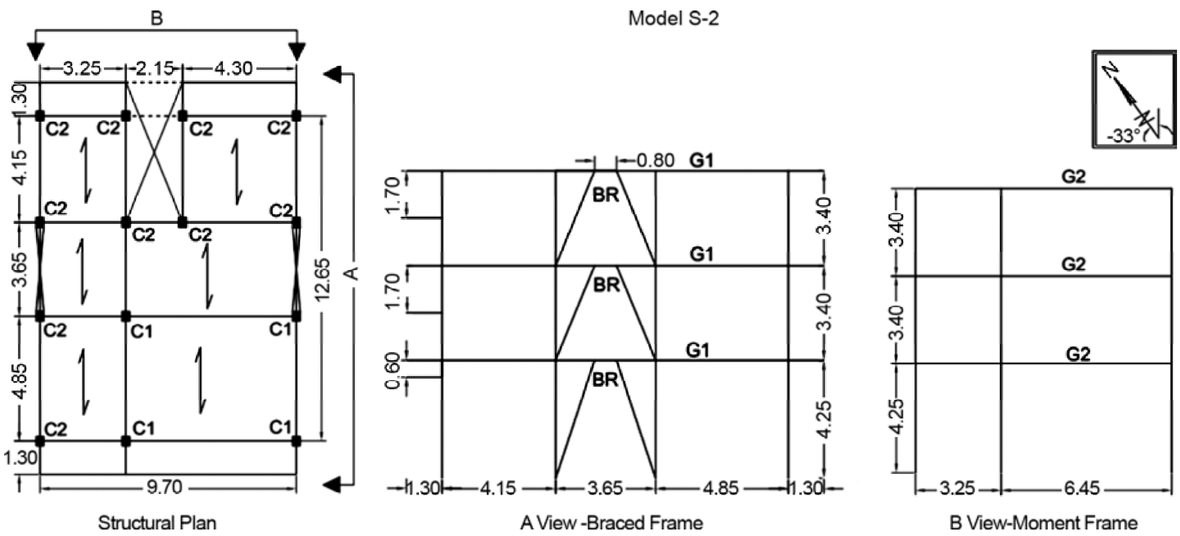

Figure 7. Plan and view of S-2 building.

Figure 8. Real view of S-2 building after Sarpol-e Zahab earthquake.

Table 4. S-1 Building structural sections.

	Beams		Columns
	Frame Dir	Brace Dir	
Stories 1 to 3	G2	G1	According to Figure (4)
Stories 2, 3	G2	G1	C2
Balconies	G1	G1	-

Table 5. S-2 Building beams and columns.

Element	Name	Details
Beams	G1	IPE16
	G2	2IPE20
	G3	IPE22
Columns	C1	2IPE16 c/c8cm – Pl 20×1cm
	C2	2IPE16 c/c8cm – Pl 10×20@40cm
Braces	BR	2UNP10 /1cm Link Beam Length: 80cm Free length: 90 cm End Gusset Plate: 75×35×0.8cm Center Gusset Plate: 40×40×0.8cm

Iranian regulations for loads on buildings [16] as shown in Table (6). For collapse assessment, gravity loading was done using the 1.05 dead + 0.25 live [8] combination.

6.2. Determining the infill walls parameters

The infilled walls of the studied buildings were comprised of 10cm hollow clay blocks. The properties of the walls are shown in Table (7). A model by using an equivalent diagonal strut is made to consider the effects of an infilled wall, based on the proposed method of ASCE 41-13 [11] .

6.3. Determination of Earthquake Catalog

The recent earthquake in Sarpol-e Zahab has a directivity effect, which is one of the characteristics

Table 6. Loading classes details.

Usage	Dead	Live
Residential	650 Kg/ m^2	200 Kg/ m^2
Roof	600 Kg/m^2	150 Kg/ m^2
Exterior Walls	600 Kg/m	-

Table 7. The properties of infills.

Type	Dimensions	Elastic Modulus	Compressive Strength
Hollow Clay Block	$20^{cm}\times20^{cm}\times10^{cm}$	5.6 GPa	8 MPa

of near-fault earthquakes. However, according to the fault map of Iran, the city is considered far-fault, and it is rational to use the far-fault earthquakes catalog to evaluate the collapse of the structures in this city. Therefore, 22 pairs of FEMA P695 far-fault catalog are used in this paper.

6.4. Determination of Uncertainties

As previously mentioned, in this paper, the FEMA P695 values are used to determine uncertainties by engineering judgment. In this section, these values are calculated:

- Record-to-record uncertainty (β_{RTR}): it is considered 0.4 according to the recommended instruction.
- Design requirements uncertainty (β_{DR}): this value is determined based on the compliance rate of the studied buildings with the provisions of seismic regulations. The buildings investigated this study have a seismic resistance system, and according to the age of the building, they should be designed according to the third edition of the Iranian seismic loading regulations (2800). Nevertheless, the field observations show that during the construction period, significant changes have been made to the buildings, contrary to the regulations. An increase height of about 1 meter in the ground story columns, the use of 3IPE sections in some columns of buildings S-1 and S-2, the change of the bracing profile from double UNP to the single IPE profile in the building S-1, and the lack of stiffeners in link beam of the building S-2 are among the most important changes. Therefore, based on engineering observations and judgments, these buildings fall into the "poor" category and the values of 0.5 considered for their uncertainty.
- Test data uncertainty of (β_{TD}): In modeling of these buildings, the proposed values of ASCE 41-16 instruction are used for nonlinear parameters of plastic hinges. On the other hand, there are some sections in the structural elements of the buildings whose nonlinear parameters are not provided in this instruction and the sections should be equated. Therefore, "good" category with a value of 0.2 for these buildings is considered.
- Modeling Uncertainty (β_{MDL}): Regarding the 3D nonlinear modeling of buildings and the use of an approximate IDA analysis based on the pushover method, these buildings falls into "good" category with an uncertainty of 0.2.

A summary of the uncertainty values of these buildings and total uncertainty is presented in Table (8).

7. Providing the Results

7.1. S-1 Building

The pushover curves of this building in two directions along with their idealized curves and equivalent SDOF model curve are presented in Figure (9). Table (9) also presents the parameters of the equivalent SDOF structure.

Table 8. The uncertainties of case study buildings.

Model	β_{RTR}	β_{DR}	β_{TD}	β_{MDL}	β_{TOT}
S-1	0.4	0.5	0.2	0.2	0.70
S-2					

Table 9. SDOF Parameters of S-1 building.

Direction	T_{e1} (s)	Γ	d_p (m)	d_{pc} (m)	D_u (m)	Mass (kg)
Moment Frame	1.5167	1.268	0.030	0.609	0.72	229710
Braced Frame	0.6137	1.263	0.143	0.346	0.47	241710

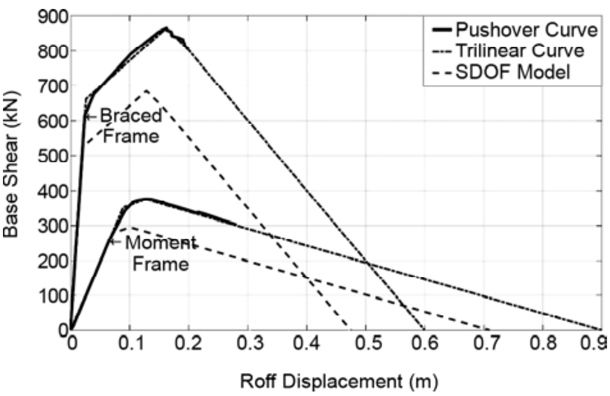

Figure 9. Pushover, Trilinear and equal SDOF of S-1 building.

Table 10. Collapse parameters of the S-1 building.

	Moment Resisting Frame				Braced Frame			
	S_C (g)	S_{MCE} (g)	β_{Tot}	% $P_{collpase}$	S_C (g)	S_{MCE} (g)	β_{Tot}	% $P_{collpase}$
μ-σ	0.36	0.91	0.7	91	1.38	1.23	0.7	44
μ	0.24			97	0.95			65
μ+σ	0.17			99	0.68			80

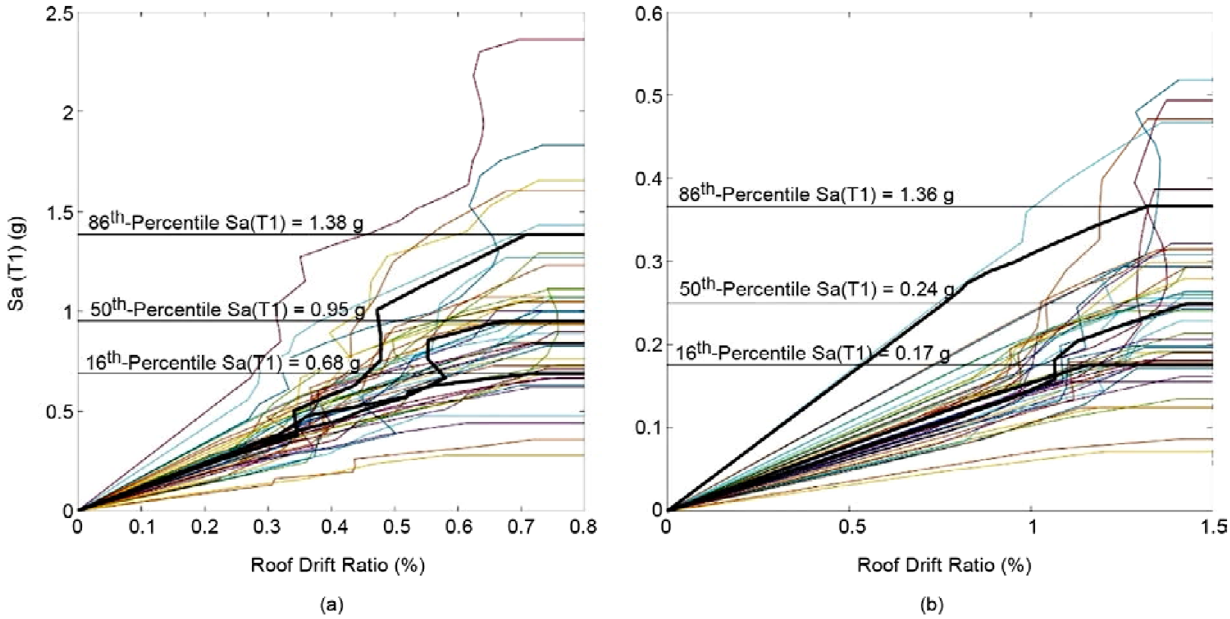

Figure 10. Approximate IDA curves of S-1 buildings, a) moment resisting frame direction, b) braced frame direction.

Through the method presented in section 5, the approximate IDA curves of this building were obtained in two main directions in the $S_a(T_1)$ - RDR coordinates. These curves, along with the median, 16th and 84th percentile collapse capacities are presented in Figure (10).

The collapse fragility curves of this building are presented in Figure (11) and are obtained with 0.70 uncertainty and collapse capacity of S_C = 0.24 g for moment resisting frame direction and S_C = 0.95 g for bracing frame direction. Table (10) also presents the para-meters related to the collapse capacity and the collapse probability of this building.

As can be seen, at the moment resisting frame direction, the acceleration corresponding to the building collapse is very small, and the collapse is almost imminent (collapse probability about 97%). Even the low-bound of collapse probability is about 91%.

The collapse probability in bracing frame direction is not within acceptable limits too (about 65%). Therefore, the structural collapse of this building was predictable under a severe earthquake. In fact, this building has structurally collapsed in Sarpol-e Zahab earthquake, so that it has been experienced a roof drift ratio of 0.025 at the moment resisting frame direction, and a roof drift of 0.027 in the bracing frame direction.

7.2. S-2 Building

The pushover curve, the idealized curve and equivalent SDOF curve of this building are presented in Figure (12). The parameters of the equivalent SDOF structure are shown in Table (12). Besides, Figure (13) shows the median, 16th and 84th percentile of collapse capacities of this building.

In Figure (14), the collapse fragility curves of this building are presented in two directions considering the uncertainty to be about 0.7 and collapse capacity

Table 11. SDOF Parameters of S-2 building.

Direction	T_{el} (s)	Γ	d_p (m)	d_{pe} (m)	D_u (m)	Mass (kg)
Moment Frame	1.68	1.23	0.217	0.487	0.81	244840
Braced Frame	1.45	1.257	0.177	0.696	0.90	294620

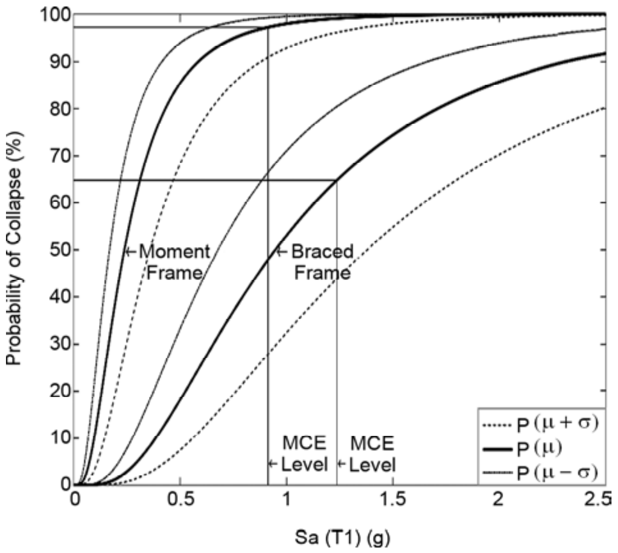

Figure 11. Collapse fragility curves of S-1 building.

to be S_C = 0.41 g for moment resisting frame direction and S_C = 0.32 g for EBF bracing frame direction. Table (12) also presents the parameters related to the collapse capacity and the collapse possibility of this building.

This building has a very high collapse probability in EBF direction (94%). In moment resisting frame direction, the collapse probability is high too (about 85%). So, it is classified as a high-risk building from seismic collapse viewpoint. The building has also undergone a seismic structural collapse in the Sarpol-e Zahab earthquake. That is because it has experienced roof drift of 0.026 in the bending frame direction and 0.027 in the EBF bracing frame direction.

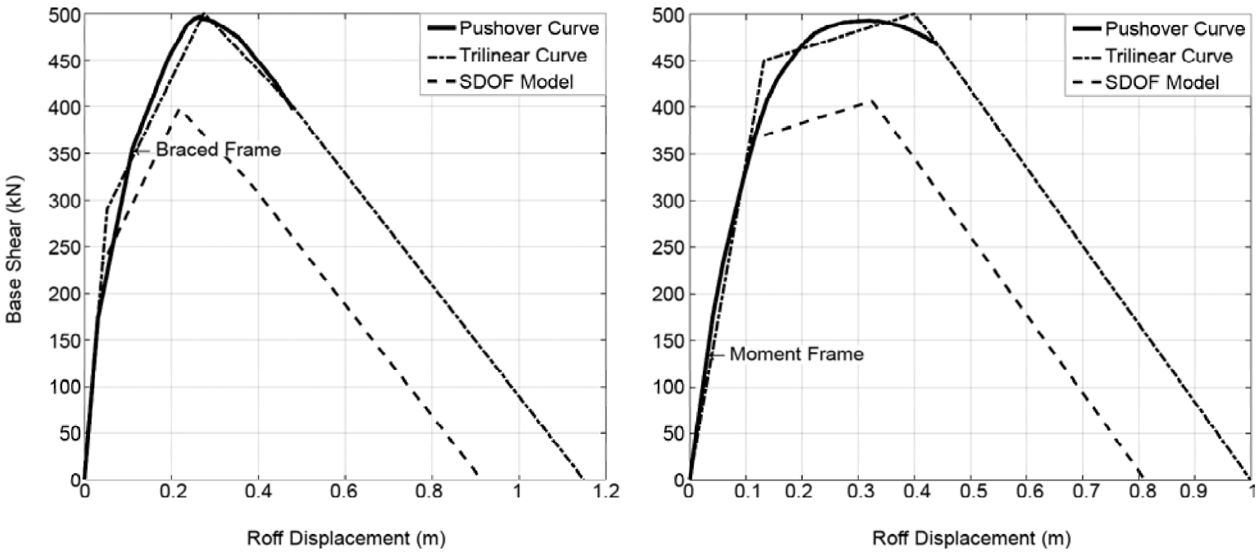

Figure 12. Pushover, Trilinear and equal SDOF of S-2 building.

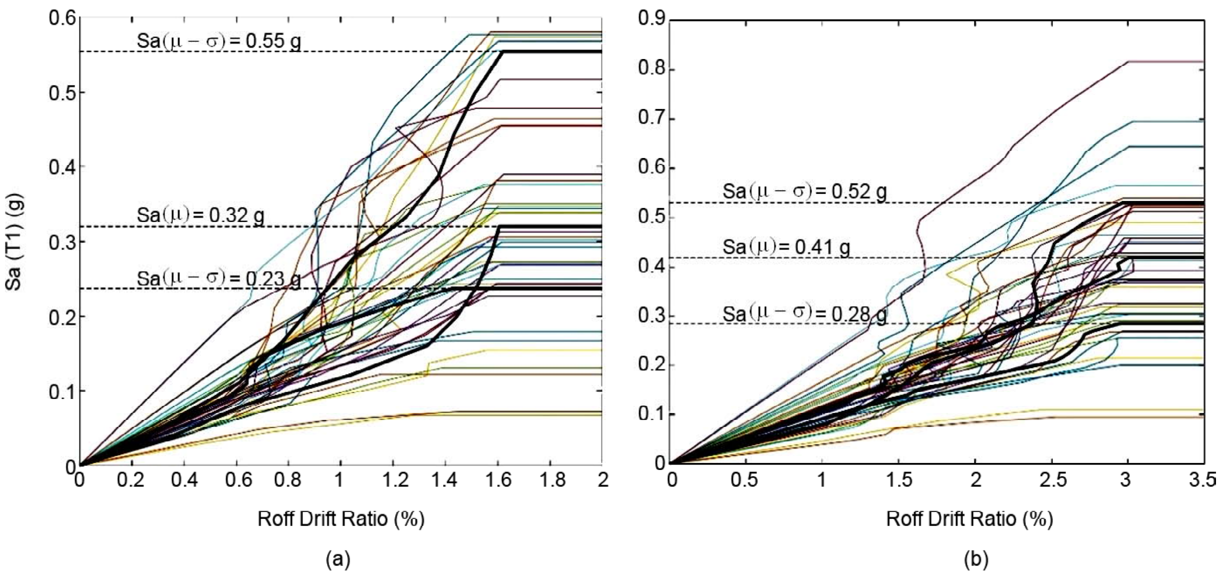

Figure 13. Approximate IDA curves of S-2 buildings, a) moment resisting frame direction, b) braced frame direction.

Table 12. Collapse parameters of the S-2 building.

	Moment Resisting Frame				Braced Frame			
	S_{CT} (g)	S_{MCE} (g)	β_{Tot}	% $P_{collpase}$	S_C (g)	S_{MCE} (g)	β_{Tot}	% $P_{collpase}$
μ-σ	0.28			94	0.23			98
μ	0.41	0.85	0.7	85	0.32	0.94	0.7	94
μ+σ	0.53			75	0.55			78

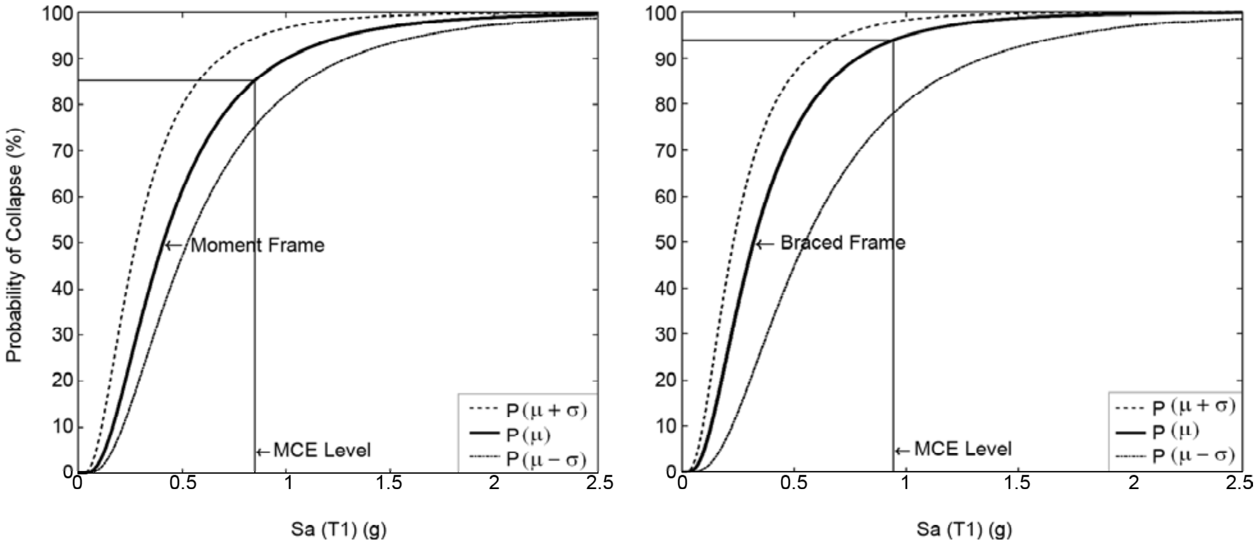

Table 14. Collapse fragility curves of S-2 building.

8. Conclusion

Seismic collapse assessment of the existing buildings is a very important issue in urban seismic risk management. Due to the vastness of the cities, the use of rapid and simplified assessment methods is highly practical. In this paper, the approximate IDA method based on the results of the first-mode pushover analysis was evaluated as a simplified method for seismic collapse capacity assessment. That is, two 3-story steel buildings structurally collapsed in the recent Sarpol-e Zahab earthquake were selected and evaluated using the mentioned method. The results showed that this method could predict the building collapse with an acceptable accuracy before the earthquake.

Besides, by using this method, the engineering demand parameters (EDPs) of the structure could be determined under desired earthquake catalog. This method only uses the first-mode pushover results in the assessment process. Therefore, it can be used for conventional regular buildings in which the first mode is the dominant mode. In high-rise or irregular buildings, it is necessary to consider the effect of higher modes on the assessment process.

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