Verification of a Proposed Assessment Method Applied to Concrete Buildings Collapsed During Sarpol-e Zahab, Iran Earthquake

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ABSTRACT

Detecting the buildings experiencing collapse against future earthquakes is the most vital for seismic urban areas in Iran because of its irreparable consequences. Once again, the occurrence of Sarpol-e Zahab Earthquake (Mw= 7.3) reminded us of this necessity where structural collapses resulted in a large number of casualties. A simplified methodology is developed to assess the collapse of mid-rise concrete buildings during earthquakes in Iran. Besides, an attempt is made to verify this method through analyzing the recorded data of the collapsed buildings suffered from Sarpol-e Zahab earthquake of November 12, 2017, and considers whether the occurrence of the collapse could be anticipated or not. Three severely damaged buildings were selected, located in Sarpol-e Zahab, to verify this proposed methodology. They are 2 or 3 story buildings having moment resisting frames. The buildings are analyzed through nonlinear analysis. The well-calibrated nonlinear model is adopted for the nonlinear analysis. The intensity of damages are observed and recorded by the authors. The pushover analysis is conducted for them. Drifts evaluated by pushover analysis are compared to those recorded in the buildings. One of the buildings was a bare frame that its partitions and infill walls are not still constructed. This building had much better performance than two others and experienced less loss. Moreover, the results of the analysis show that the collapse criteria related to the seismic evaluation codes are non-conservative. The results of this survey imply that the proposed method can precisely forecast the collapse or non-collapse of the studied buildings. Therefore, it would be recognized as a reliable method for collapse assessment.

Keywords:
Seismic assessment; Reinforced-concrete buildings; Pushover analysis; Sarpol-e Zahab; Iran earthquake

1. Introduction

In the evening of November 12, 2017, Kermanshah province of Iran suffered from one of the most destructive seismic events during the past two decades. The Sarpol-e-Zahab Earthquake, with a magnitude of 7.3 (Mw) and PGA of 0.69 (g) caused extensive damages to buildings. Before the earthquake, most of the existing buildings were code-conforming ones. Some of the buildings underwent both partial and total collapse during the earthquake due to the lack of proper hazard estimation of the seismic area, changes of codes and deficiencies of construction. Therefore, it seems essential to propose methods for predicting the collapse or non-collapse of buildings. Quantifying the collapse probability plays a major role in any urban decision making for the earthquake-prone cities of Iran. The main challenges of collapse assessment are: 'Is it possible to forecast the collapse of existing buildings before the earthquake?'; 'What is the reliable method for collapse assessment?' To answer these questions, we should be aware of the deficiency of current methods. 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 whole structure to record the sudden rise of the structure response and the incremental dynamic analysis (IDA) method [1]. In this paper, the authors focus on the implementation of IDA methods to detect collapse in the behavior of structures under seismic loads. IDA plots IM (intensity measure) of ground motions (such as PGA or Sa) versus maximum EDP of structural response, while the location of maximum EDP in the structure is not clarified. Besides, the collapse mechanism in every structure intensively correlates to the distribution of plasticity and the locations of maximum response. Therefore, according to the results of this paper, the trend of IDA curves will not define the collapse capacity of the structure precisely. It should be pointed out that the IDA curve trend will be accurate for the equivalent single-degree-of-freedom system and some researches focus on the approach of capturing collapse mode through pushover and IDA analyses of SDOF (single degree of freedom) [2]. To avoid the limitations of SDOF systems and utilizing IDA curves for collapse evaluation, a more accurate approach, based on IDA is implemented in this study. A two-dimensional model developed for each archetype of RC frames using the OpenSees (2016) structural analysis software. Inelastic beams and columns are modeled through concentrated hinge developed by Ibarra et al. [3].

The sample buildings in this study are residential and have 2 and 3 stories. They have a 30-cm deck floor system that is conventional in the Iranian construction industry. They were designed according to the Iranian seismic code (Standard No. 2800). The buildings are divided into three types namely: case study 1, case study 2 and case study 3. This paper aims to quantify the collapse limit in each of these case studies, which are observed and investigated in the earthquake field by the authors. It is necessary to find out the probable collapse modes for this type of frames. The side-way collapse is recognized as a predominant collapse mode for this type of frames. As it was seen, there was no evidence of the vertical collapse mechanism and beam-column joint failure for the studied buildings. Therefore, the concentrated plastic hinges employed for nonlinear analyses would be valid. The collapse criteria in the proposed methodology are based on the capacity matrix. The results have indicated that the occurrence of the collapse was expected for studied buildings.

2. A Proposed Method for Collapse Assessment

The results of a study carried out by the authors imply that using the current procedure assessing collapse through IDA (Incremental Dynamic Analysis) leads to an overestimation of collapse capacities [4]. On the other hand, the IDA analysis is necessary to account for the uncertainty specifications of records in collapse assessment. Thus, a new approach is developed to evaluate collapse capacities more realistically. This study sheds light on several steps of collapse assessment, and it also focuses on the new approach of collapse determination through IDA. This new approach is more reliable than the current approach defining collapse. According to a newly proposed methodology by Zahedi and Eshghi [5], steps incorporated in collapse assessment are explained. As a beginning step, some archetypes are selected. Among mid-rise buildings from field surveys [5]. These surveyed buildings were designed according to the past revisions of the Iranian seismic code, and their configurations and details are extracted for defining archetype buildings. The calibrated concentrated hinges are employed to conduct a nonlinear analysis. Flowingly, pushover analysis is performed for each archetype to obtain collapse capacity for each story and to develop a capacity matrix. Subsequently, a series of IDA is done, and then the collapse criteria associated with the capacity matrix are applied to detect the collapse on each IDA curve. Finally, the collapse probability curves for the buildings are developed. The features of buildings in the study by Zahedi and Eshghi [5] are similar to the buildings damaged during Sarpol-e Zahab earthquake, especially since they are both code-conforming. Therefore, the same method adopted for collapse assessment has been applied.

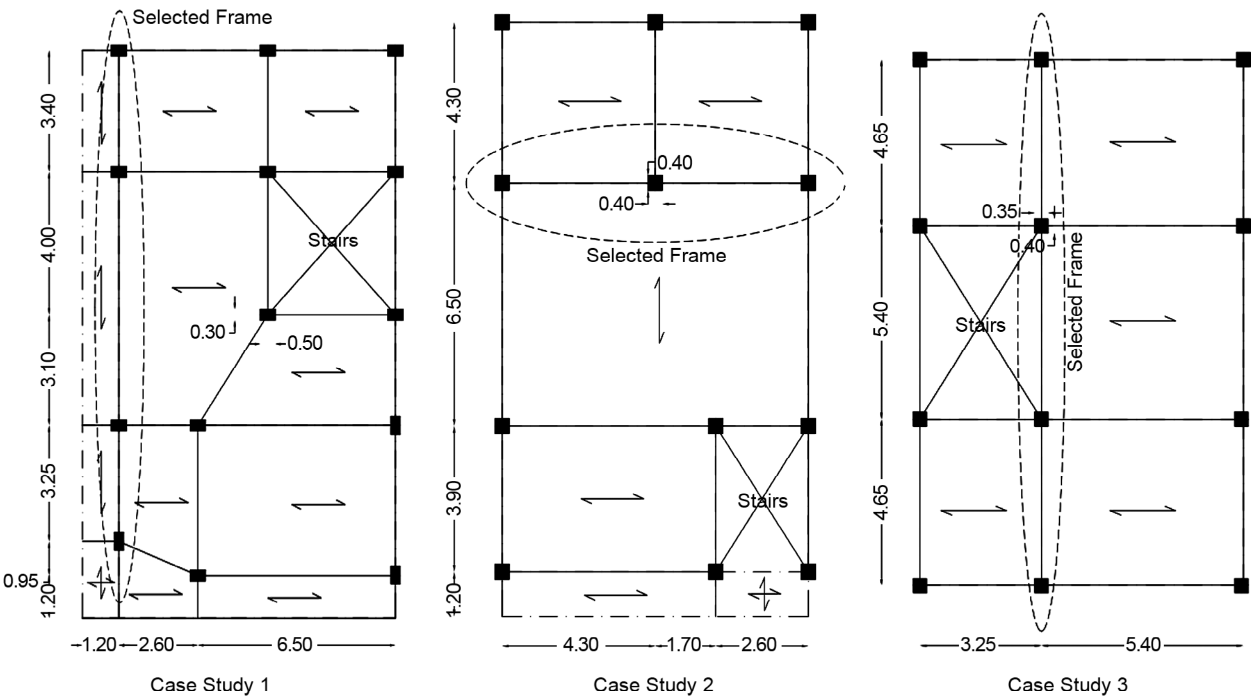
3. Three Collapsed Buildings During Sarpol-e Zahab Earthquake

The sample buildings in this study are residential and have 2 and 3 stories. They have a 30-cm deck floor system, which is conventional in the Iranian construction industry. They were designed according to the Iranian seismic code (Standard No. 2800). The buildings are divided into three types namely: case study 1, case study 2 and case study 3. They have a plan area of 10.6 m by 13 m, 9 m by 16.1 m and 9 m by 15.1 m for case 1, case 2 and case 3, respectively. The first story of case 1 has 4.5 m high and for two other cases have 3.25 m high, all other stories for all cases have 3.25 m high. The structures are designed for the high seismic hazard of Sarpol-e Zahab zone according to the Iranian seismic code (Standard No. 2800). These buildings are designed to withstand the dead and live load and the seismic load and their combinations. It is assumed that these structures conform to seismic codes detailing requirements such as transverse confinement in the beam-column region, seismic hook and lap splice. They also fulfill other requirements of RC moment frames, including maximum and minimum rein-forcement ratios, maximum hoop spacing.

Three frames are selected from three sample buildings. Figures (1) and (2) show the location of extracted frames in plans and studied sample buildings. Figure (3) exhibits general configurations of selected frames. The case study frame 1 and 3 has three bays. The case study frame 2 has two bays. The total length of case 1, 2 and 3 are 14 m, 8.60 m and 14.70 m, respectively. The selected RC moment frames are modeled without infill walls and are regular in plan, without major strength or stiffness irregularities. Figure (3) also presents the dimensions of beams and columns. There are three types of columns in the frames having a size of 0.30×0.5, 0.4×0.4 and 0.35×0.4 (all dimensions are in meters). Figure (1) displays which columns are rectangular or square. The sizes of the beams' sections are all 0.4×0.4. These are code-conforming structural elements and designed to resist design base shear relating to the Iranian seismic code.

4. Structural Model and Simulating the Collapse

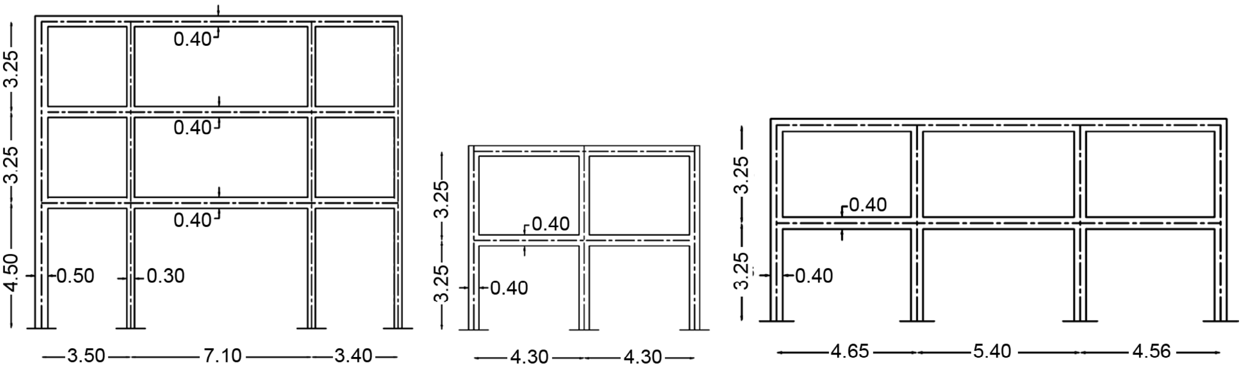
As the Figure (1) shows, there is no irregularity in plans of selected buildings, also, the authors attempted to avoid the complexity of three-



Figrue 1. Plans of sample buildings.

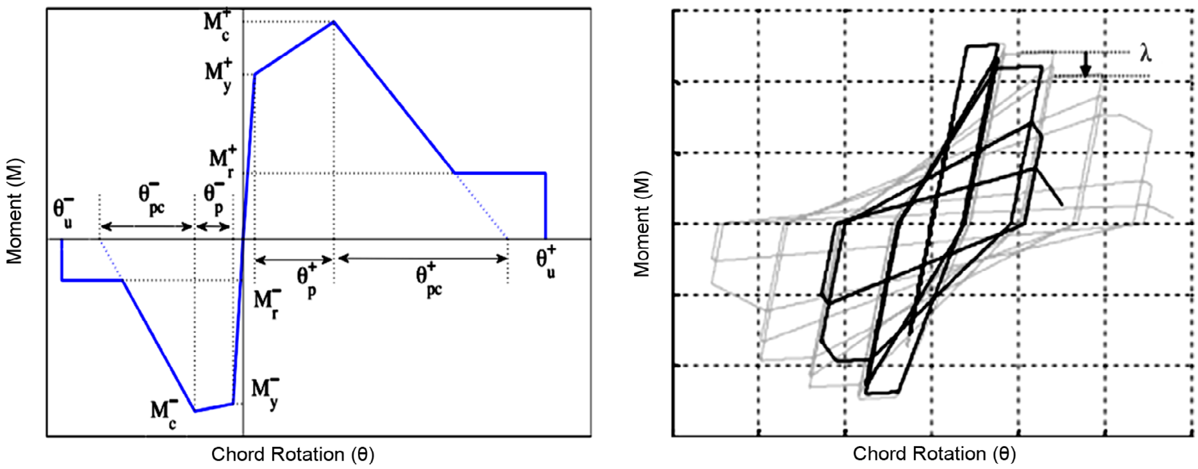
Case-1 Case-2 Case-3

Figrue 2. The three heavily damaged buildings after the earthquake.



Selected Frame of Case Study 1 Selected Frame of Case Study 2 Selected Frame of Case Study 3

Figrue 3. General configurations of selected frames.



(a) Backbone Curve of Modified Ibarra-Medina-Krawinkler (b) Cyclic Deterioration

Figrue 4. A peak- oriented model for concentrated plastic hinge [3].

dimensional analysis. Therefore, a two-dimensional model developed for each archetype of RC frames using the OpenSees [6] structural analysis software. Inelastic beams and columns are modeled through concentrated hinge developed by Ibarra [3]. The nonlinear hinges modeled as zero-length elements at two points for beam-column elements. Figure (4) illustrates the backbone and hysteretic models of the nonlinear hinge model. As depicted in Figure (4), Ibarra [3] model captures the essential modes of monotonic and cyclic deterioration that precipitates sideway collapse [7].

The properties of nonlinear hinges of beam-column elements are extracted from a set of calibrated parameters according to the experimental tests of beam-columns, as described by Haselton [8]. This model includes eight parameters. The first two parameters are M_y and θ_y, which are defined according to Fardis [9] equation. Moreover, the concrete cracking occurs at low-level deformation; therefore, the initial elastic stiffness of hinge is very significant to simulate the response at low-level deformation. In this study, according to Ibarra [3], the initial stiffness of all members (zero-length and elastic) are defined to account for cracking and to model the full range of behavior appropriately [8]. The residual strength of the hinge (M_r) is determined equal to twenty percent of the yield moment.

4.1. Analytical and Occurred Drifts of Case Study Frames

The studied buildings are modeled in ETABS software. The buildings are analyzed for all the dead, live and seismic loads. The seismic loading on buildings is defined and applied according to Iranian Standard No. 2800. Iranian buildings are designed to carry load combinations accounted for design base earthquake in Iranian Standard No. 2800. There are proposed drifts for buildings the estimated drifts of which must not exceed them. In Iranian design code procedure, the method of analysis is elastic, then the elastic drifts transformed to equivalent inelastic drifts through deflection amplification factor (C_d). These values are evaluated for studied buildings suffering Sarpol-e Zahab earthquake. Table (1) shows these values and as reported, the occurred drift for case study 1 and 2, are 2.15% and 3.15%, respectively. The occurred drift is not measured for case study 1, while there is no observation of permanent displacement.

Table 1. Analytical and real drift of the studied buildings.

Case Study	#1	#2	#3
Maximum Elastic Drift in X Direction Under Design Base Earthquake (Iranian Standard No. 2800)	0.00311	0.00114	0.0023
Maximum Elastic Drift in Y Direction Under Design Base Earthquake (Iranian Standard No. 2800)	0.0033	0.00124	0.00213
Allowable Drift	0.0046	0.0046	0.0046
Deflection Amplification Factor (Iranian Standard No. 2800)	4.5	4.5	4.5
Maximum Occurred Drift	-	0.0215	0.0315

4.2. Pushover Analysis

The nonlinear static analyses conducted for three models using an inverted triangular pushover load distribution. Figure (5) shows three graphs for three case study models. Comparison of design shear and ultimate base shear shows that the ultimate base shear of the models is lower than design base shear. This result implies that the selected buildings do not meet Iranian seismic code requirements. While the overstrength factor (Ω) for case 3 was near 1.8, it was 1.25 for both case 1 and case 2. The results showed that the case 3 frame has approximately 1.44 times higher overstrength than the other two cases.

Figure (5) also represents the ultimate roof drift ratio (RDR_{ult}) for three case study models. The ultimate RDR is defined in which the ultimate strength has been decreased by 20% [10]. It is clear from the Figure (5) that the RDR_{ult} of the case 3 structure is nearly 1.5 times larger than the RDR_{ult} of case 1 and case 2 frames. The RDR_{ult} values are 1.2% for case 3 and 0.8% for case 1 and case 2. The source of these results depicted in Figure (6). As schematic signs for all of the hinges in beams and columns described (Figure 7), the distribution of nonlinearity is vaster in Case 3 than case 1 and case 2 frames.

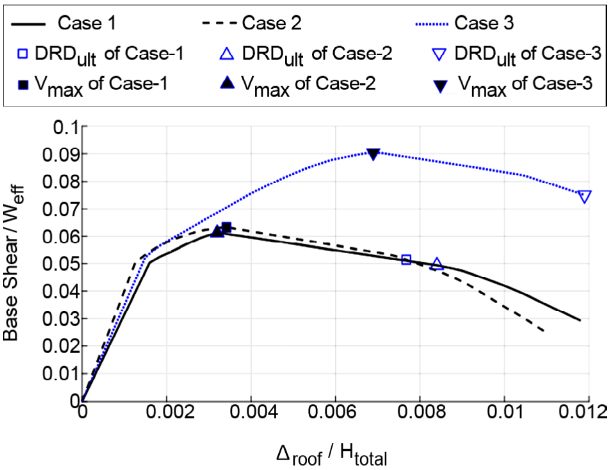


Figure 5. Static pushover curves using an inverted triangular load pattern for the case study frames.

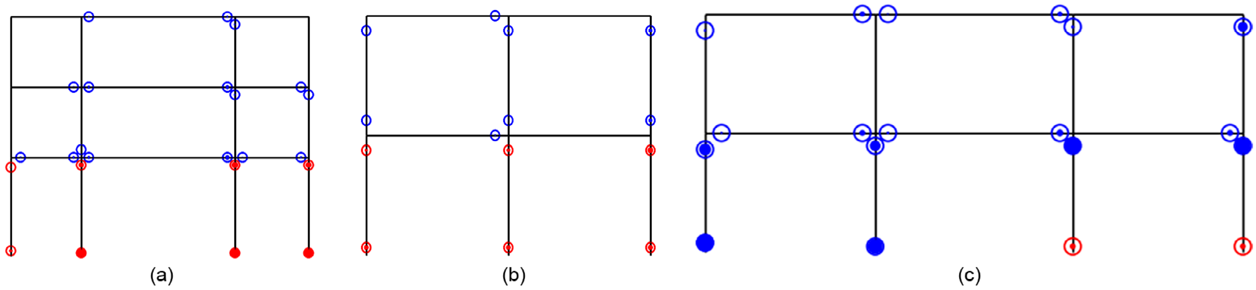


Figure 6. Pushover deformations of case study frames (a) Typ. 1 (b) Typ. 2 (c) Typ. 3.

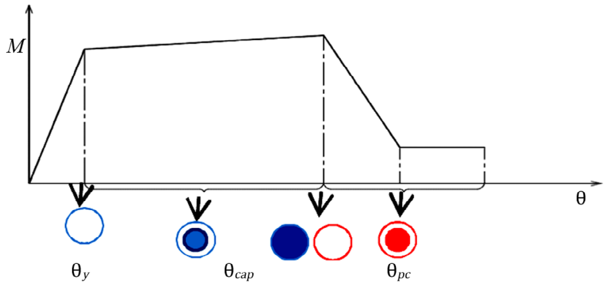


Figure 7. Schematic signs for different stages through non-linear plastic hinge formation.

4.3. Collapse Mode Detected Using Pushover Analysis

As explained in previous sections, the sideway collapse is the most frequently observed collapse mode for this type of archetypical frame, owing to detailing and other reasons such as the ratio of V_P to V_n for these configurations, which were not larger than 0.35 for both frames (FEMA-P750, Collapse indicator). Figure (6) makes clear the plasticity spread throughout all members of the frames in a way that the probable sideway collapse mode can be predicted and determined in the analysis. Figure (6) shows the collapse mechanism in frame structures. Therefore, in this study, the collapse mode captured and recognized as a beam and story mechanism. Thus, the designs and detailing of archetypical frames of this study were based on modern seismic codes, and the frames were ductile, it is not expected that the story mechanism would occur in these frames. It should be mentioned that any significant deficiencies of construction could challenge this assumption. While the effects of deficiencies are out of the scope of this study, there is no evidence of collapse mode except for sideway collapse in real damaged structures in Sarpol-e Zahab zone.

The relative size of inner and outer circles indicates the ratio of $\theta_{max}/\theta_{cap}$ in the blue region and θ_{max}/θ_{pc} in the red region.

4.4. Detecting Collapse Drift Through Drift Capacity Matrix

When pushover analysis is conducted for a structure, the capacity curve plot for shear story versus drift can be extracted for each story. This graph represents the structural behaviour of the story under a lateral load and is a suitable and simplified method for defining capacity drift. Collapse can be defined in terms of an acceptable story drift limit occurring after the formation of a failure mechanism in the structure. A capacity matrix is derived to contain *N* drift capacities and can be employed to determine the collapse [11-12]. Figures (8) to (10) show the calculated drift

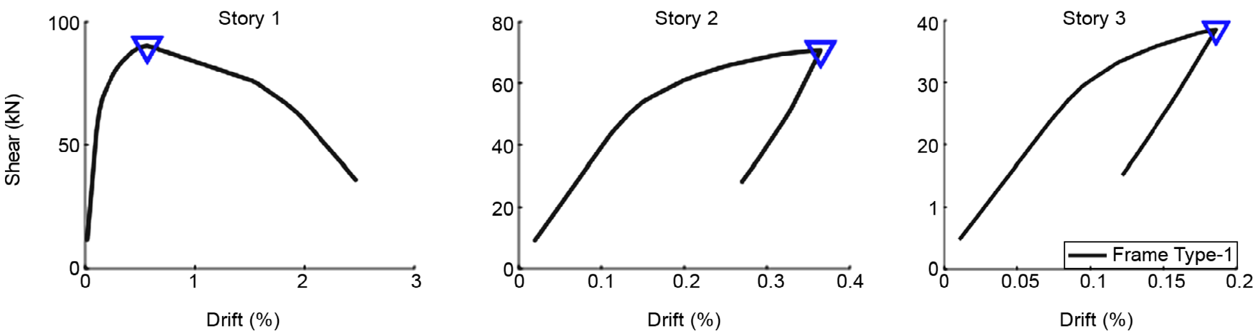


Figure 8. Storey shear vs. drift of each story for the frame Case-1 triangular pushover load pattern (the inverted triangles indicate drift capacity).

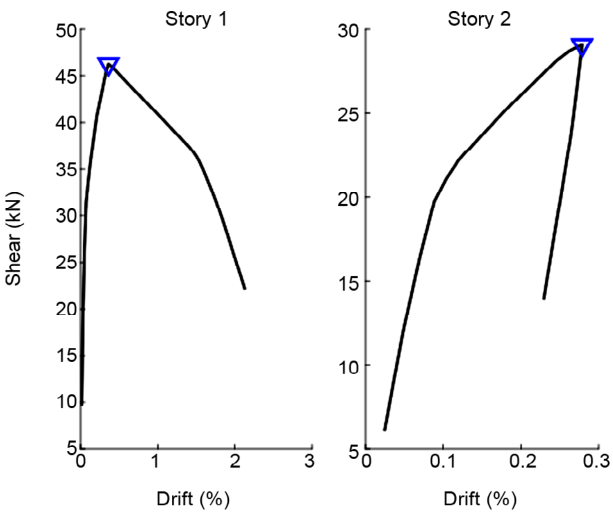


Figure 9. Storey shear vs. drift of each story for the frame Case-2 triangular pushover load pattern (the inverted triangles indicate the drift capacity).

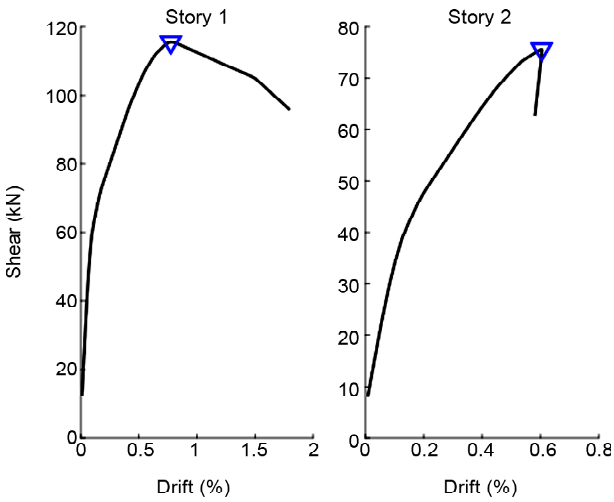


Figure 10. Storey shear vs. drift of each story for the frame Case-3 Triangular pushover load pattern (the inverted triangles indicate drift capacity).

capacities for all case study frames. The ultimate drift in each curve for each story is named capacity drift of that story.

5. Incremental Dynamic Analysis (IDA) and Collapse Assessment

Figures (11) and (12) show the IDA analysis for three frames. The IDA analyses conducted for seven ground motion record pairs. The records are selected from the far-field set of FEMA-P695. The list of records is tabulated in Table (2). The increasing of record intensities is continued up to the sideway collapse. Several analyses are carried out to capture the global collapse of frames, respectively. Thus, pushover analysis of frames shows the formation of beam and story mechanism

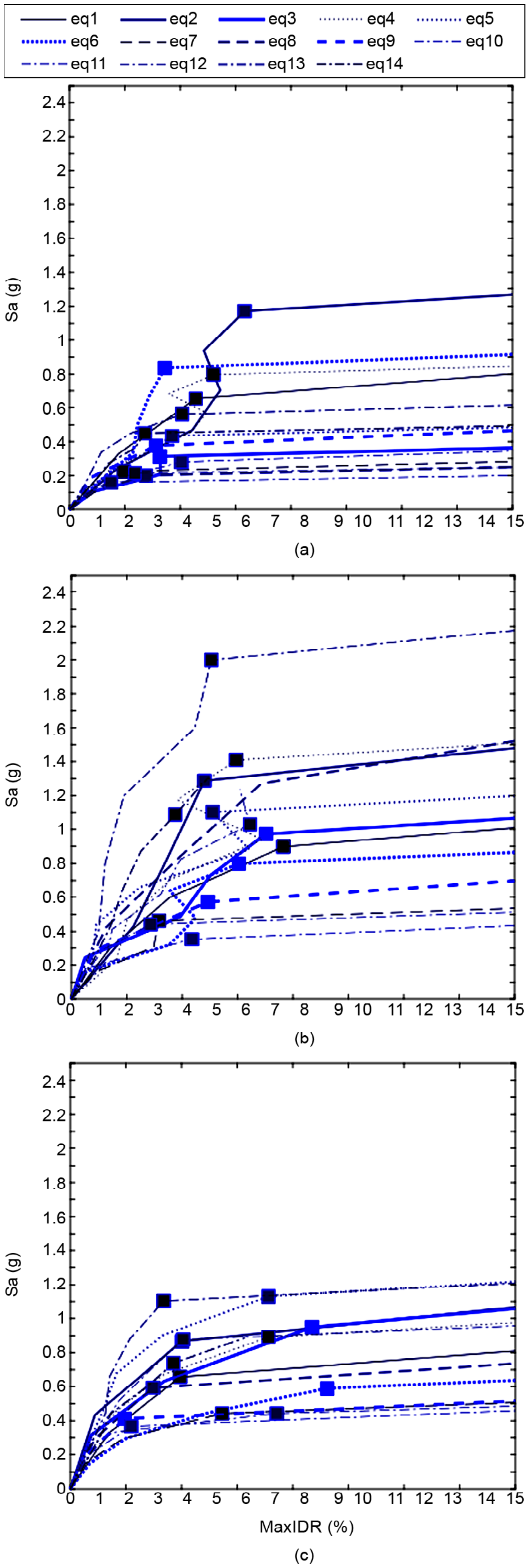


Figure 11. The IDA curves and the associated collapse drifts (Approach #1) for the selected frames of Kermanshah buildings (a) Case-1, (b) Case-2, (c) Case-3.

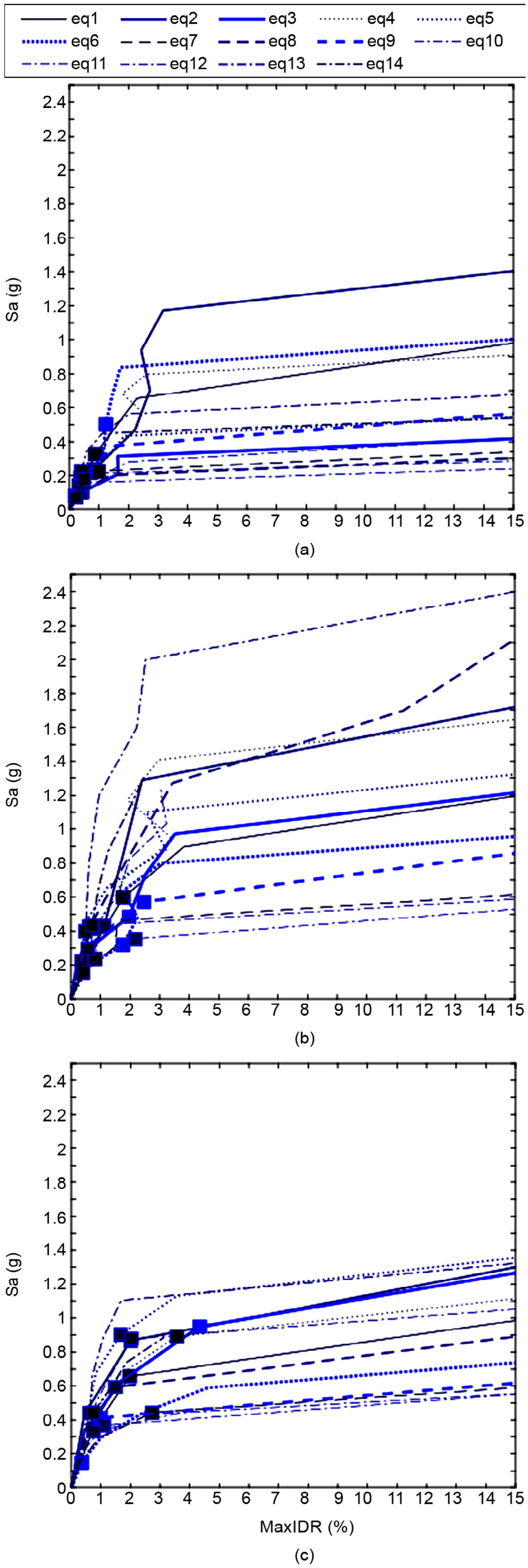


Figure 12. The IDA curves and the associated collapse drifts (Approach #2) for the selected frames of Kermanshah buildings (a) Case-1, (b) Case-2, (c) Case-3.

in the frame resulting in a global sideway collapse in structures. A building's global drift capacity is considered to the maximum story drift ratio at which the maximum story drift ratio versus spectral acceleration curve becomes flat or the maximum story drift ratio at which this curve reaches a slope equal to 20% of the slope in the elastic region of the curve. Hence, for each record, the IDR of collapse is assigned in curves through this approach (this approach = approach #1) (Figure 11).

The results of the IDA analyses clarify that IDR and Sa parameters for three frames all follow the lognormal distribution. The median computed collapse values are 0.0168, 0.0254 and 0.0201 for IDR in all frames of the case 1, case 2 and case 3, respectively (Figure 11). The median $S_a(T_1)$ capacities calculated as 0.404 g, 1.00 g and 0.6967 g for case 1, case 2 and case 3, respectively. The value of the record-to-record variability (σ_{lnRTR}) are estimated as 0.367 for the case 1 frame, 0.271 for the case 2 frame and 0.142 for the case 3. Now, the cumulative density function of $S_a(T_1)$ parameters is plotted. Figure (13) presents the probability curves of the frames using IDA analyses. The results of global collapse capacities marked in Figure (11). These capacities contradict those indicated in Figure (12). To be more precise, the global drift capacities do not correlate well with partial capacities and unquestionably violate them. To deal with this challenge, a new approach (new approach=approach#2) is applied. In this approach, the capacity matrix is incorporated into the assessment of collapse through IDA curves

Table 3. Means and standard deviations values of collapse limits for two methods extracted from different IDA curves.

Case.	1	2	3
$\mu_{lnx}{}^{*}$	-0.9118	-0.1091	-0.3936
$\sigma_{lnx}{}^{*}$	0.3669	0.2706	0.1418
$\mu_{lnx}{}^{**}$	-1.7872	-1.0267	-0.6311
$\sigma_{lnx}{}^{**}$	0.3049	0.1452	0.2769
Mean of CD*	0.0168	0.0254	0.0202
Mean of CD**	0.0042	0.0077	0.0157

μ_{lnx}* and σ_{lnx} *: Mean and Standard deviation of X respectively, which denotes the S_a and having the log-normal distribution, the S_a evaluated according to the conventional collapse determination.

μ_{lnx}** and σ_{lnx}**: Mean and Standard deviation of X respectively, (X denotes the S_a), the Sa calculated according to the new approach of collapse determination.

CD* and CD**: Values of the Collapse Drift based on conventional and new method respectively.

Table 2. Far-field records set applied for analysis [13].

ID No.	PEER-NGA Record Information				Recorded Motions	
	Record Seq. No.	Lowest Freq (Hz.)	File Names – Horizontal Records: Component 1	File Names – Horizontal Records: Component 2	PGA_{max} (g)	PGV_{max} (cm/s.)
1	953	0.25	NORTHR/MUL009	NORTHR/MUL279	0.52	63
2	960	0.13	NORTHR/LOS000	NORTHR/LOS270	0.48	45
3	1602	0.06	DUZCE/BOL000	DUZCE/BOL090	0.82	62
4	1787	0.04	HECTOR/HEC000	HECTOR/HEC090	0.34	42
5	169	0.06	IMPVALL/H-DLT262	IMPVALL/H-DLT352	0.35	33
6	174	0.25	IMPVALL/H-E11140	IMPVALL/H-E11230	0.38	42
7	1111	0.13	KOBE/NIS000	KOBE/NIS090	0.51	37

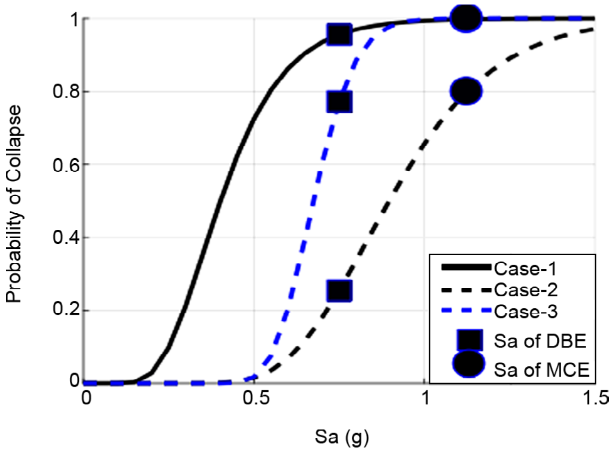


Figure 13. The collapse probability curves extracted from approach #1 for selected frames of Kermanshah buildings.

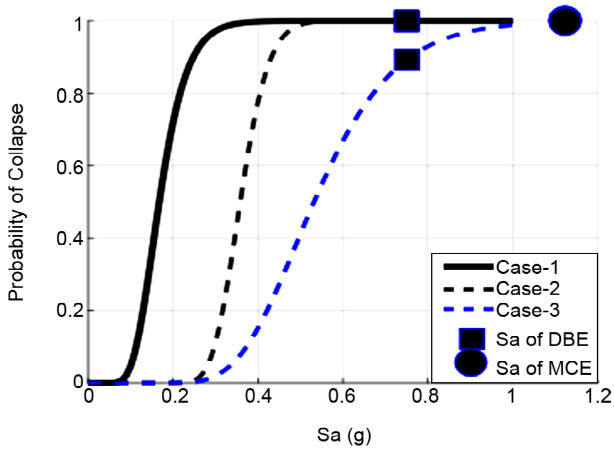


Figure 14. The probability curves extracted from approach #2 for selected frames of Kermanshah buildings.

(Figure 12). In this study, a new definition of the collapse criteria is presented regarding a capacity drift matrix resulted from a pushover analysis (Figures (8) to (10)). The collapse can be detected in a way that, as soon as the first story exceeded its capacity drift, it would be marked as an indicator of structural collapse through an IDA analysis. Accordingly, the collapse limit state through each IDA analysis was defined (Figure 12).Therefore, the collapse probability curve is also estimated for approach #2 (Figure 14). It is observed that the procedure of collapse criteria strongly affects the collapse probability. For instance, the median drift calculated from IDA analyses of case 1 varies from 1.68% to 0.42% for the approach # 1 and the approach # 2, respectively. These data demonstrate the importance of collapse criteria. The logic diagram of approach #2 is shown in Figure (15).

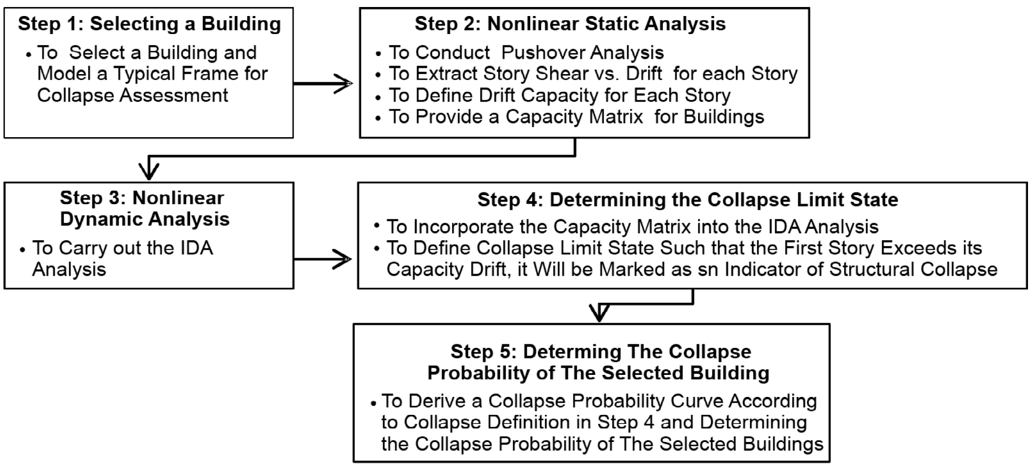


Figure 15. The process logic diagram for the proposed method (approach #2).

6. Conclusion

The collapse criteria were carried out through IDA curves in these two approaches. In the approach #1, the global drift capacity was considered to the maximum story drift ratio at which the maximum story drift ratio versus spectral acceleration curve becomes flat or the maximum story drift ratio at which this curve reached a slope equal to 20% of the slope in the elastic region of the curve. The collapse criteria in the proposed methodology (approach #2) are based on the capacity matrix. The results have been indicated that the occurrence of the collapse was expected for studied buildings. The results imply that the collapse probability curve in Figure (14) correlates to the damage observed in the field better than Figure (13). In this study, the authors attempted to propose an accurate and simplified method for collapse assessment. The results observed from the damages due to this earthquake proved that the revision in detailing for masonry infill walls are necessary for residential buildings. Besides, the results of the analysis show that the collapse criteria related to the Iranian seismic code are non-conservative.

The new collapse assessment method proposed by the authors, in this study, is not time-consuming. The results of this survey imply that the proposed method can precisely forecast the collapse or non-collapse of studied buildings. Therefore, it would be recognized as a reliable method for collapse assessment based on Sarpol-e Zahab damaged buildings. It is worth mentioning that the major advantage of this method is its simple model and its simplicity to be used by practicing engineers. This method can mark 'killer RC buildings' in a residential area. Adopting a good policy in emergency management will essentially be required to detect the killer buildings. Thus, applying this method can remarkably decrease casualties.

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