Drift Based Damage Functions for Reinforced Concrete Moment Resisting Frames

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ABSTRACT

Determination of seismic performance for a structure is one of the most important topics that researchers have attended to. Most of regulations regarding performance-based design, introduce the drift as a criterion to determine global seismic performance of the structure. Recently, pushover analysis has widely been adopted as the primary tool for nonlinear analysis because of its simplicity and facility compared with dynamic procedures. The main objective of this research is to develop some relations to estimate damage to Reinforced Concrete Moment Resisting Frames (RCMRFs) based on drift criterion resulting from pushover analysis. For this purpose, by employing the Park-Ang damage index, damage analysis is performed on several frames subjected to various earthquake records. By comparing the amounts of damage and drift and evaluating correlation between two sets, some explicit damage functions are derived based on the pushover results. These functions can be applied to estimate the damage to the structures using a simple pushover analysis. The reliability of FEMA-273 acceptance limits on the drift criterion is discussed using the proposed drift based damage functions.

Keywords:
Damage index; Seismic Performance; Drift Criterion; Reinforced concrete frame

1. Introduction

Performance-Based Seismic Design (PBD) is a relatively new concept in structural engineering and is rapidly becoming widely accepted in professional practice. The growing acceptability of the performance-based design approach is reflected by a number of documents regarding seismic rehabilitation of existing buildings that have been published by the Federal Emergency Management Agency (FEMA), the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), California Universities for Research in Earthquake Engineering (CUREE), and SAC (a joint venture of SEAOC, ATC and CUREE). The concepts and principles laid out in these publications for seismic rehabilitation can also be applied for new building construction in the context of performance-based design [1-4]. This design method involves a set of procedures by which a building structure is designed in a controlled manner such that its behavior is ensured at predefined performance levels under earthquake loading. A nonlinear analysis tool is required to evaluate earthquake demands at the various performance levels. Pushover analysis is widely adopted as the primary tool for such nonlinear analysis because of its simplicity compared with dynamic procedures [5].

The main objective of the PBD is to control damage to the structure subjected to an earthquake. There is a correlation between each structural performance level and its corresponding damage to the structure [6]. For structural damage, local parameters such as shear distortions in joints and rotations at plastic hinges may be most relevant. In most cases, these local parameters can be deduced from story drifts [7]. In the PBD, for the Operational (OP) and Immediate Occupancy (IO) levels, the structure experiences no or minimal plastic deformation, and the emphasis is on maintaining elastic or minimal inelastic behavior. For the more severe Life Safety (LS) and Collapse Prevention (CP) performance levels, the emphasis is on controlling inter-story drift and inelastic deformation [8]. In addition, the inter-story drift is the criterion that is recommended by the seismic guidelines for evaluation of the global performance of the structure. For example, a framework that undergoes an overall drift of 1%, 2% and 4% of the building height is at the IO, LS and CP performance levels, respectively [5]. In recent years, researchers have attended this criterion as an engineering demand parameter. Krawinkler et al. quantified some relevant demand parameters such as inter-story drift for regular frame structures and illustrated how statistically representative relationships between these parameters and ground motion intensity measures can be established [9]. Erduran and Yakut developed three damage curves as a function of the drift ratio for three different levels of ductility [10]. These curves could be used in the evaluation and vulnerability assessment of reinforced concrete frame buildings. Their numerical results show that some performance-based acceptance criteria of ATC-40 [11] need to be revised. Lu et al. provided a simple alternative method for the prediction of the storey drift distribution and the critical drift concentration in a RC frame [12]. They introduced a new storey capacity factor to represent the combined effect of the storey strength and stiffness on the distribution of storey drift along the frame height. They could provide an appropriate estimation of the storey drift distribution. Ruiz-Garcia and Miranda presented the implementation of a probabilistic approach to estimate residual drift demands during the seismic performance-based assessment of existing multi-story buildings [13]. They showed that the relationship between transient (maximum) and residual (permanent) drift demands depends on the mean annual frequency of exceedance and the building's number of stories for a similar lateral load resisting system.

Considering the highly complicated and time-consuming calculations required in calculating various damage indices in time-history analysis, it is important to develop procedures that enable engineers to estimate the damage to a structure in the context of the PBD by a simple method. The main objective of this research is to find a correlation between the structural damage and overall drift of the structure on the basic of the numerical results of nonlinear analysis. A practical method based on the static pushover analysis is proposed to estimate the expected damage to RCMRFs when subjected to earthquakes. For this purpose, damage analysis is performed on several RCMRFs subjected to various earthquakes using Park-Ang damage index, and then the average damage is computed for each frame. On the other hand, pushover analysis is performed on each frame and overall drift ratio value is calculated at performance point using the capacity spectrum method. Furthermore, as a suitable solution for designers, a table corresponding with the drift criterion is presented to control the damage and to determine performance of the structure. The proposed index can provide a powerful and practical tool for design of RCMRFs with damage control.

2. Proposed Methodology

In this research, the damage in reinforced concrete elements is quantified with the Park-Ang damage index in order to evaluate the accuracy of the proposed damage criteria. The preference of this index is its conformity with experimental results and its simplicity and ranking proportion with observed damage. The index combines the maximum lateral displacement effects with the plastic dissipated energy at one end of the element according to the following relation [14]:

$$DI_{P\&A} = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta E_h}{M_y \theta_u} \tag{1}$$

where θ_m and θ_r are the maximum and yield rotation, respectively, and θ_u is the ultimate rotation capacity of the section. The ultimate rotation capacity θ_u is expressed through the ultimate curvature of the section as determined from a fiber model analysis of the cross-section. The incremental curvature that is applied to the section is continued until one of the following conditions is reached:

a) The specified ultimate compressive strain in the concrete is reached.
b) The specified ultimate strength of a rebar is reached.

The attained curvature of the section when reaching either of the two conditions is recorded as the ultimate curvature. It is obvious that the ultimate curvature is dependent of the cross-section and its reinforcement. Accordingly, amounts of the ultimate rotation capacity for various elements of the structure are not the same. M_y is the yield moment and β is a constant parameter, which depends on structural characteristics and history of inelastic response. A value of 0.1 for the parameter β has been suggested for nominal strength deterioration [14]. This value has been used to calculate the damage in the dynamic analysis in this research. E_h is the hysteretic energy includes cumulative effects of the repeated cycles of inelastic response.

A well-defined damage index is a normalized quantity that will be zero if the structure remains elastic (i.e., no significant damage is expected) and will be unity if there is a potential of failure. Structural performance states (such as operational, life-safety, collapse prevention, etc.) correspond to values of *DI* between zero and unity. The park and Ang damage index has been calibrated with observed structural damage. Table (1) presents the calibration damage index with the degree of observed damage in structure. A more detailed description of these terms is found in [14].

To determine the performance of the structure using damage index, referring to introduced details for different performance levels in references such as ATC40, FEMA273, one can approximately consider Operational Level (OP), Immediate Occupancy (IO) level, Life Safety (LS) level and Collapse Prevention (CP) level in correspondence with negligible damage, low damage, moderate damage and severe damage in Park-Ang criterion, respectively.

Table 1. The relation between Park-Ang damage index and dam age state.

Degree of Damage	Damage Index	Physical Appearance	State of Building
Collapse	>1.0	Partial or Total Collapse of Building	Loss of Building
Severe	0.4-1	Extensive Crashing of Concrete; Disclosure of Buckled Reinforcement	Beyond Repair
Moderate	0.25-0.4	Extensive Large Cracks; Spalling of Concrete in Weaker Elements	Repairable
Minor	0.1-0.25	Minor Cracks; Partial Crushing of Concrete in Columns	Repairable
Slight	<0.1	Sporadic Occurrence of Cracking	Repairable

As mentioned before, the main objective of this research is to develop a drift-based damage function for RCMRFs. The drift criterion is a popular index that is employed to determine the global performance of the structure. In addition, this criterion is recommended by existing seismic guidelines such as FEMA273 and ATC40 for evaluation of the performance level of the structure. In this study, the index is obtained from pushover analysis using the following relation:

$$(DI)_{Drift} = \frac{\Delta_m}{H} \tag{2}$$

where Δ_m is the target displacement at the performance level under consideration and *H* is the height of the structure. To calculate this index, monotonically increasing lateral loads along with constant gravity loads are applied to the frame until the control node (usually referred to the building roof) sways to a predefined 'target' lateral displacement. Consequently, the relationship between the base shear and roof displacement, known as the capacity curve and is the fundamental product of the pushover analysis, is determined. Then, intersecting the capacity spectrum and inelastic demand spectrum, the performance point is obtained [11]. Therefore, the nonlinear responses such as displacement are determined at the performance point. Now, having the value of the overall drift at the performance point, static damage index can be computed from Eq. (2).

To develop reliable damage-drift relation, a number of nonlinear dynamic time history analysis and nonlinear static analysis are carried out for reinforced concrete moment resisting frames. Then, by comparing the results of the damage index from Eq. (1) and the drift criterion from Eq. (2) and fitting a curve, which has the best fit to a series of data points (damage and drift), the correlation between these criteria is obtained. It must be noticed that overall drift of building, which is average value of all drifts of stories, has been used in Eq. (2). This criterion is a function of all inter-story drifts. Although, the dynamic damage index, the relationship of which with overall drift has been determined in this research, is computed based on damage indices of all the elements and stories. Accordingly, global damage of the structure is estimated based on the average value of inter-story drifts. The proposed damage index is a global criterion that can predict the overall performance of the building, but it is not applicable to predict local damages.

3. Design of Frame Models

To obtain a database of sufficient size, fourteen reinforced concrete frames with various numbers of stories and bays, as shown in Figure (1), have been considered [15]. These frames can include most of low-rise to mid-rise buildings. The numbers of stories are assumed to be one, three, five, seven, and nine in the two-bay frames; five, eight, twelve and fifteen in the four-bay frames; and two, four, six, eight and ten in the five-bay frames. The height of each storey is 3.2 meters and the length of each bay is 4 meters in all frames. It is assumed that all frames lie on rock site. These frames are loaded based on Iranian seismic code 2800 [16] for zone of intermediate relative seismic hazard. Building importance factor for all of them is considered to be one. The distributed dead and live loads of 29822 N/m and 7848 N/m are applied to the beams at all the stories. The concrete is assumed to have the cylinder strength of 30 Mpa, a modulus of rupture of 3.45 Mpa, a modulus of elasticity of 27386 Mpa, a strain of 0.002 at maximum strength and an ultimate strain of 0.003. The steel has the yield strength of 300 Mpa and the modulus of elasticity of 200000 Mpa. The frames are designed based on ACI provisions. Some characteristics of these frames have been briefed in Table (2). In the Frame Number column of the table, "S" denotes the number of the stories and "B" denotes the number of the bays. The member numbers and the corresponding group numbers are given in Tables (3) and (4). In addition, cross-sectional characteristics of beam and column elements are given in these two tables. The beam element numbers and column element numbers have been showed in Figure (2) for a general frame with n bays and m stories.

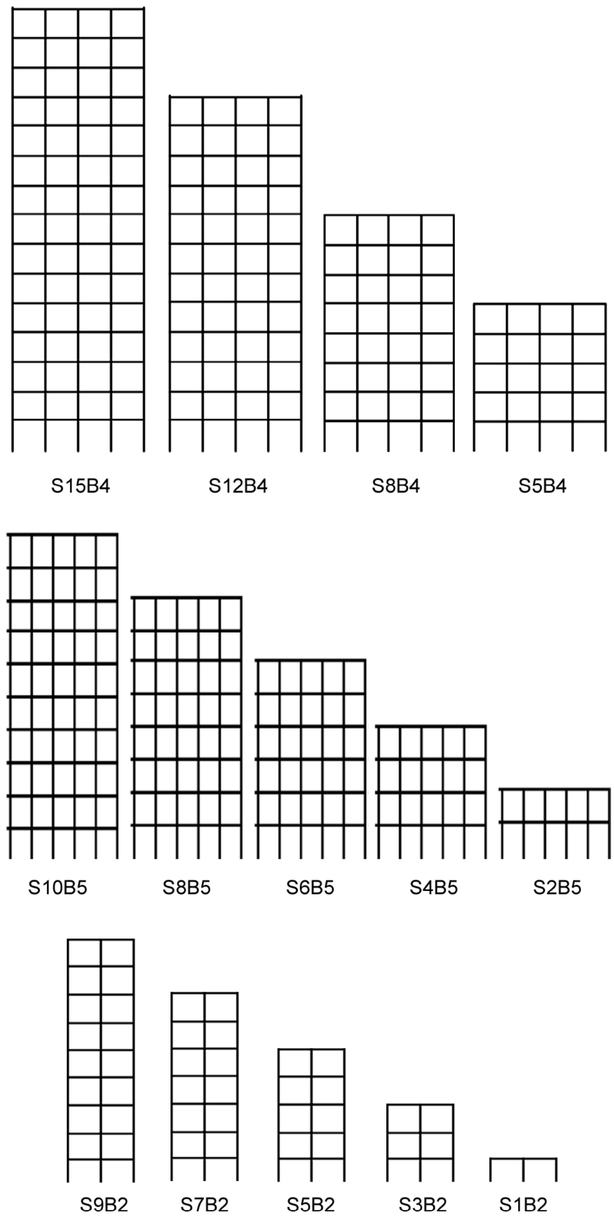


Figure 1. Geometry and names of the studied frames.

Table 2. Characteristics of the studied frames.

Frame Number	Height (m)	Period	Base Shear
S5B4	16	0.56	159.8
S8B4	25.6	0.79	202.1
S12B4	38.4	1.07	247.6
S15B4	48	1.27	276.8
S2B5	6.4	0.28	114.1
S4B5	12.8	0.47	173.5
S6B5	19.2	0.64	215.2
S8B5	25.6	0.79	248.3
S10B5	32	0.941	277.76
S1B2	3.2	0.167	22.83
S3B2	9.6	0.38	73
S5B2	16	0.56	94.3
S7B2	22.4	0.72	111.82
S9B2	28.8	0.87	105.52

Table 3. Cross-sectional characteristics of beams of the studied frames.

Frame	Type	Elements	Dimension		Reinforcement			
			Width	Height	Left		Right	
					Bottom	Top	Bottom	Top
S9B2	1	(3-8)	300	400	724	1598	724	1477
	2	(1,2), (9-12)	300	400	913	1772	913	1772
	3	13, 14	300	400	526	1249	526	1249
	4	(15-18)	300	300	390	1266	390	1266
S7B2	1	1, 2, 7, 8	300	400	860	1605	860	1605
	2	(3-6)	300	400	982	1840	982	1840
	3	(9-12)	300	400	500	1330	500	1330
	4	13, 14	300	300	350	1016	350	1016
S5B2	1	(1-4)	300	400	730	1521	730	1521
	2	(5-8)	300	400	580	1320	580	1320
	3	9, 10	300	300	340	1093	340	1093
S3B2	1	(1-4)	300	300	620	1780	620	1780
	2	5, 6	300	300	387	1271	387	1271
S1B2	1	1,2	300	350	356	1028	356	1028
S10B5	1	(6-20)	300	450	1073	1922	1073	1922
	2	(21-35), (1-5)	300	400	926	1521	926	1521
	3	(36-50)	300	350	500	1382	500	1382
S8B5	1	1, (5-15)	300	400	840	1690	840	1690
	2	(2-4), (16-25)	300	400	570	1360	570	1360
	3	(26-35)	300	350	440	1380	440	1380
	4	(36-40)	300	300	370	1000	370	1000
S6B5	1	(1-15)	300	400	721	1600	721	1600
	2	(16-25)	300	300	500	1630	500	1630
	3	(26-30)	300	300	360	1080	360	1080
S4B5	1	(1-10)	300	400	644	1359	1359	644
	2	(11-15)	300	300	425	1381	1381	425
	3	(16-20)	300	300	352	1118	1118	352
S2B5	1	(1-10)	300	300	420	1334	420	1334
S15B4	1	(1-20), 22, 23, (25,32), 34, 35, 38, 39	300	400	1272	2660	1272	2660
	2	21, 24, 33, 36, 37, (40-51), 54, 55	300	400	1017	2280	1017	2280
	3	52, 56	300	400	764	1140	764	1140
	4	(57-60)	300	300	508	1520	508	1520
S12B4	1	(1-28)	300	400	1017	2279	1017	2279
	2	29, (32-36)	300	400	1017	1519	1017	1519
	3	(37-44)	300	300	763	2279	763	2279
	4	(45-48)	300	300	510	1520	510	1520
S8B4	1	(1-25), 28	300	400	764	1884	764	1884
	2	26, 27	300	400	764	1256	764	1256
	3	(29-32)	300	300	510	1256	510	1256
S5B4	1	(1-8)	300	400	764	1570	764	1570
	2	(9-16)	300	300	764	1884	764	1884
	3	(17-20)	300	300	508	1256	508	1256

Table 4. Cross-sectional characteristics of columns of the studied frames.

Frame Name	Type	Elements	Dimension Width	Dimension Height	Reinforcement
S9B2	1	(1-3)	300	450	2666
	2	(4-12)	300	450	1350
	3	13, 14	300	400	1200
	4	25, 26, 18	300	350	1760
	5	(15-17), (19-24), 27	300	350	1050
S7B2	1	1, 2, 3	300	400	3174
	2	5, 8	300	400	2020
	3	4, 6, 7, 9	300	400	1200
	4	11	300	350	1832
	5	14	300	350	1500
	6	10, 12, 13, 15	300	350	1124
	7	19, 21	300	300	1980
	8	(16-18)	300	300	1340
	9	20	300	300	900
S5B2	1	(1-3), 5, 8	300	350	2532
	2	4, 6, 7, 9	300	350	1232
	3	13,15	300	300	1992
	4	(10-12)	300	300	1468
	5	14	300	300	900
S3B2	1	1, 2, 3	300	300	3098
	2	5, 7, 9	300	300	2332
	3	4, 6, 8	300	300	1764
S1B2	1	1,3	300	300	1270
	2	2	300	300	750
S15B4	1	7, 9, 13	400	400	4550
	2	12, 14	400	400	3800
	3	17, 18, 19	400	400	3040
	4	16, 20, (22-25), (27-29), 1, 5, 6, 10, 11, 15	400	400	2280
	5	2, 3, 4, 8	400	500	3800
	6	(32-34)	300	400	3800
	7	21, 25, (37-39)	300	400	3040
	8	26, 30, (42-44), 47, 49	300	400	2280
	9	31, 35, 36, 40, 41, 45, 48, (52-54)	300	400	1520
	10	(56-60), (62-67), 70, 71, 75	300	300	2280
	11	46,50,51,55, 68,(72-74)	300	300	1520
S8B4	1	(1-5),8	300	400	2040
	2	7,9	300	400	1527
	3	(12-14)	300	300	3054
	4	17,19	300	300	2544
	5	11,15,18,22, 24,25,36,40	300	300	2036
	6	16,20,21,23, 25,(27-35)	300	300	1527
	7	6,10,(37-39)	300	300	1520

Frame	Type	Elements	Dimension Width	Dimension Height	Reinforcement
S10B5	1	1,6	400	400	4310
	2	7, 12, 13, 18, 19, 24, 25, 30	400	400	1600
	3	(2-5)	400	400	4600
	4	(8-11)	300	400	3940
	5	(14-17)	300	400	2920
	6	(20-23)	300	400	2020
	7	(26-29)	300	400	1600
	8	31, 32, (35-38), 41, 42, 43, 44, 47, 48	300	400	1200
	9	33, 34, 39, 40, 45, 46, (49-54), (56-59)	300	350	1100
	10	55, 60	300	350	1740
S8B5	1	2, 3, 4, 5	300	400	4000
	2	1,6,8,9,10,11	300	400	3120
	3	7, (12-18)	300	400	1200
	4	(20-23), (26-29)	300	350	1600
	5	(31-36), 19, 24, 25, 30	300	350	1200
	6	43,48	300	300	2000
	7	(37-42), (44-47)	300	300	1240
S6B5	1	1,6	350	350	2740
	2	(2-5), 20, 23	350	350	2160
	3	(8-11), 14, 17, 19, 24	300	350	1640
	4	7, 12, 13, 15, 16, 18, 21, 22, 27, 28	300	350	1220
	5	31,36	300	300	2130
	6	25, 26, 29, 30, (32-35)	300	300	1100
S4B5	1	(1-6), (8-11), 19, 24	300	300	2550
	2	7, (12-18)	300	300	1600
	3	(20-23)	300	300	900
S2B5	1	(1-7), 12	300	300	2000
	2	(8-11)	300	300	1030
S12B4	1	2, 3, 4	400	400	3040
	2	7, 8, 9, 12, 13, 14	400	400	2280
	3	17, 18, 19, 22, 24, 1, 5	300	400	3040
	4	23, 27, 29, 6, 10	300	400	2280
	5	28, (32-34), 11, 15, (37-39), 16, 20, 21, 25, 26, 30	300	400	1520
	6	56, 60	300	300	3040
	7	(42-50)	300	300	2280
	8	(51-55),(57-59),31,35,36,40	300	300	1520
S5B4	1	(1-5),21,25	300	300	2036
	2	(7-16),6,20	300	300	1527
	3	(17-19),(22-24)	300	300	1018

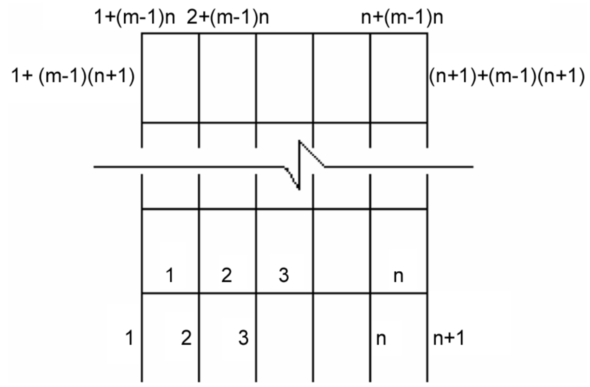


Figure 2. Element numbers for a general frame with m stories and n bays.

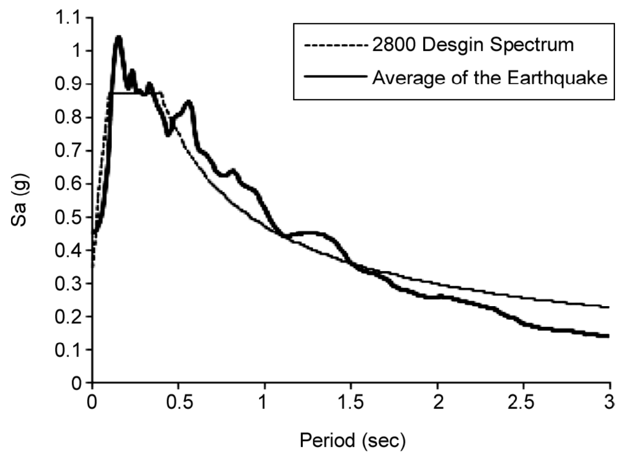


Figure 3. Average response spectrums of the scaled accelerograms.

4. Calibration of Earthquake Accelerograms

Earthquake records used in this study are a set of seven earthquakes selected from a group of twenty records used in FEMA-440 [17] for site class A that is relatively similar to soil type 1 in Iranian seismic code [16]. These records are scaled to match the Iranian 2800 standard response spectrum and scaling method is according to this standard. All records are scaled for the periodic range between 0.03 and 2.4 second to have response spectrum with minimum difference with the Iranian code response spectrum for soil type 1. These ground motion records are listed in Table (5), and average of their response spectrum together with response spectrum of standard 2800 [16] are shown in Figure (3).

Table 5. Ground motion records.

Earthquake Number	Record	Station	Component (deg)	PGA (g)
1	Imperial Valley	286	135	0.195
2	Landers	21081	90	0.146
3	Loma Prieta	58131	270	0.06
4	Loma Prieta	58151	90	0.09
5	Loma Prieta	58338	45	0.084
6	Northridge	23590	90	0.056
7	Northridge	90019	180	0.256

5. Verification of Analysis Results

In this research, the IDARC software [18] is used to perform nonlinear dynamic and static analysis of all the studied models. In this section, two numerical examples are considered to verify the analysis results. First example is a three-story frame that is employed to verify the results of the IDARC software and second example is a five-story frame that is employed to verify the model.

5.1. Nonlinear Analysis of a Three-Story Frame

A 1:2 scale model of this frame was tested in the laboratory by Yunfei et al [19]. The structure was tested using a displacement controlled loading as shown in Figure (4). Length of all beams is 3000 mm and length of all columns is 1500 mm. Details of the member sections and the essential reinforcement used for the analysis are given in Table (6).

The frame is made of 40.2 MPa concrete and is reinforced by Grade 40 steel (400 MPa yield strength). The first three cycles of loading produced cracking and first yielding. Subsequent loading of three cycles at the same ductility were applied until the frame collapsed.

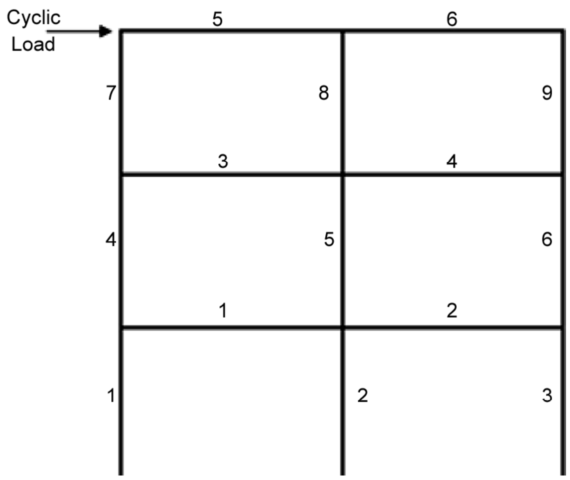


Figure 4. Half-scale model frame.

Table 6. Details of half-scale model frame.

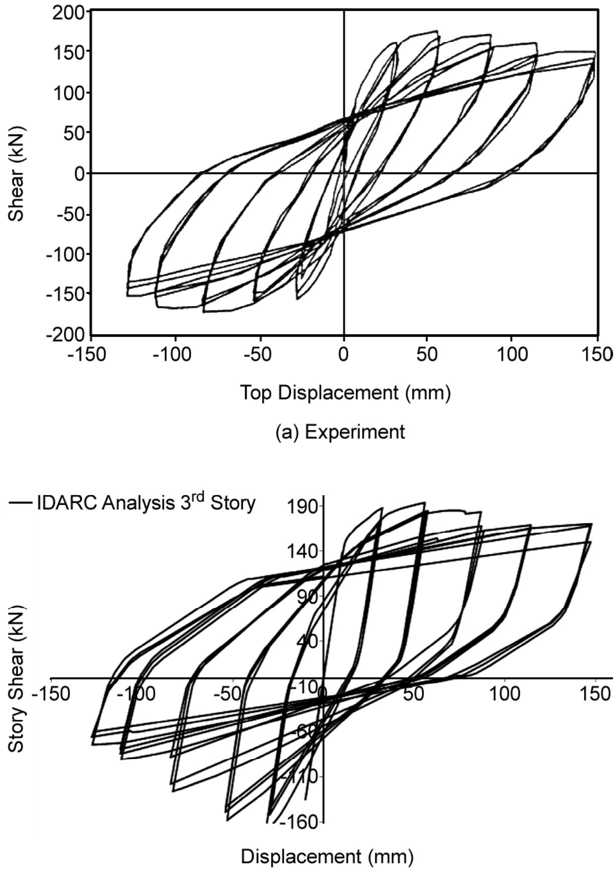
Element Ttype	Member No.	Dimension		Left		Right		Hoops
		Width	Height	Bot	Top	Bot	Top	
Beam	1 , 2	150	300	2Φ16	2Φ16	2Φ16	2Φ16	Φ 6@75
	3	150	300	2Φ16	2Φ18	2Φ16+1Φ10	2Φ16	Φ 6@75
	4	150	300	2Φ16+1Φ10	2Φ16	2Φ16	2Φ18	Φ 6@75
	5	150	300	2Φ14	2Φ14	2Φ14	2Φ12	Φ 6@75
	6	150	300	2Φ14	2Φ12	2Φ14	2Φ14	Φ 6@75
		Width	Height	Bar				
Column	1, 2, 3, 5, 8	250	250	4Φ14				Φ12@75
	4, 6, 7, 9	250	250	4Φ12				Φ 8@75

Figure (5) presents a comparison of observed vs. simulated force-deformation response for the third story of the frame. This comparison shows that a proper agreement is obtained using IDARC for shear-displacement relationship.

Another feature of the IDARC software is the pushover analysis under monotonically increasing lateral loads. This feature was used to determine the correspondence with the observed collapse mechanism. The frame developed a beam side sway collapse mechanism that was clearly documented in the experimental records through measured rebar yielding in the critical beam-column interface and column-base sections, and identified by visual observations. Figure (6) shows the damaged frame with observed plastic hinge locations and computed sequence of hinge formation using IDARC. As the figure shows, there is good agreement between the experiment and analysis results.

5.2. Nonlinear Analysis of a Five-Story Frame

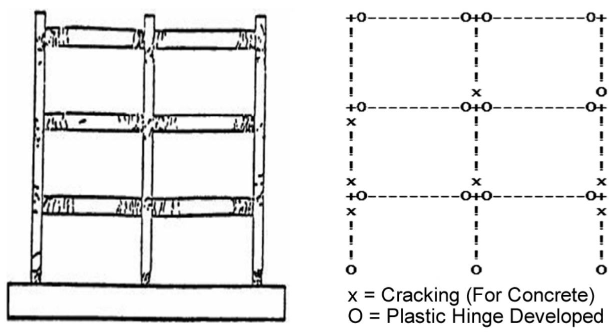
This example is concerned with the five-story frame, shown in Figure (1), which has been previously studied by Habibi et al [20]. Moreover, this frame is one model of the studied models in section 3 (model S5B4). Pushover analysis of this frame is performed by IDARC and its nonlinear static responses are calculated. The capacity curve of the structure resulting from IDARC is shown in Figure (8) and compared with that of Reference [20], as shown in Figure (7). In Figure (7), the capacity of the model M05R shows the capacity of the studied frame named S5B4 in this research. It can be seen that yielding point of the capacity curve of the structure resulting from both studies (Ref. [20] and present work) is approximately at overall drift 0.6%. In addition, it is observed that ultimate base shear is approximately obtained 15% in both researches. By



(a) Experiment

(b) Analysis

Figure 5. Comparison of observed vs. simulated force-deformation response for half-scale model frame.



(a) Experiment (b) Analysis

Figure 6. Collapse mechanism of the half-scale model frame.

comparing the capacity curves, see Figures (7) and (8), it can be concluded that the IDARC results are in good agreement with the results of Reference [20].

6. Deriving the Damage Relation

To determine correlation between the Park-Ang damage index (dynamic criterion) and the drift criterion (static criterion), pushover analysis and inelastic damage analysis is carried out on all the sample frames, which were selected in section 3. What is important in nonlinear analysis of structures, especially in reinforced concrete structures, is to employ a proper model for nonlinear behavior of elements. In this research, a spread plasticity model that has been proposed by Valles et al [18] is utilized in order to estimate the structure's nonlinear response. This plasticity model, which considers the cracking behavior of RC, can also account for material nonlinearity of RC elements with a very good approximation [18]. The IDARC software was used to perform nonlinear dynamic and static analysis of all the frames [18]. In order to increase the accuracy of the analysis results for determining capacity curves of the structures and prevent any possible numerical errors, the pushover analysis of all structures considering the different amounts of stop criteria were repeated several times. After converging, the structure's capacity was included in the later calculations. The capacity curves of the structures have been shown in Figure (8). To calculate the ultimate rotation capacity, the ultimate curvature of each element was determined from the fiber model analysis of cross-section. As a sample, the ultimate rotation values for some elements of frame S6B5 have been reported in Table (7).

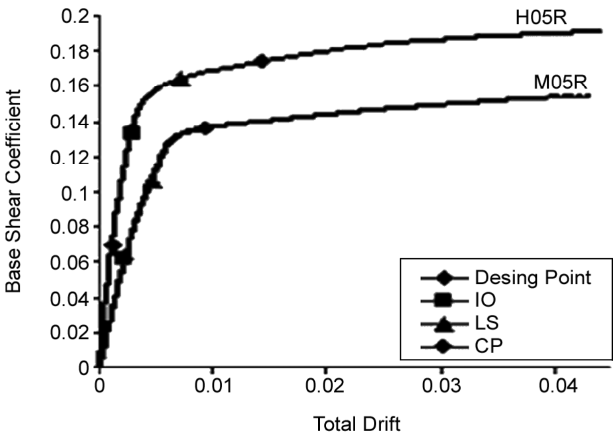
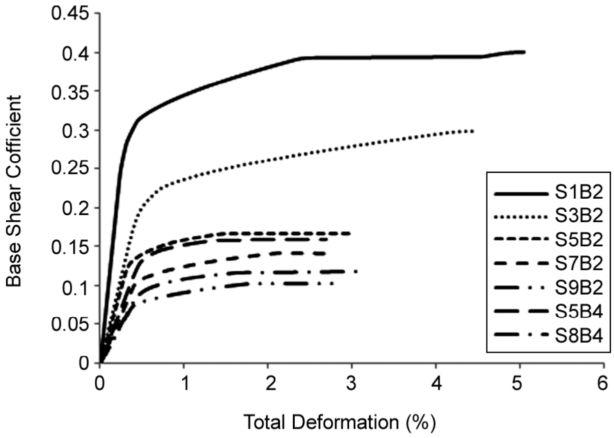


Figure 7. Capacity curve of the five-story frame [20].



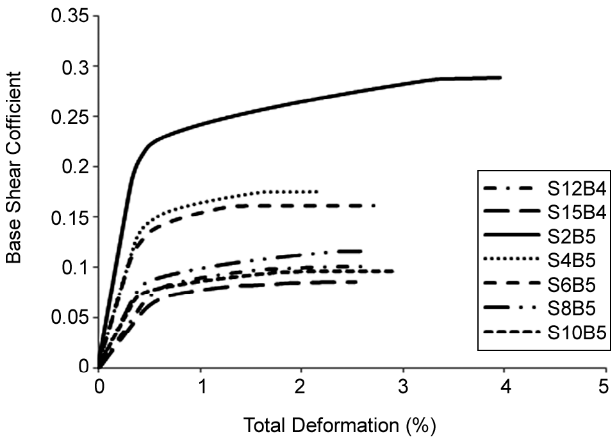


Figure 8. Capacity curves of studied frames.

Table 7. Ultimate rotation capacity for some elements of frame S6B5.

Beam Number	Dimension (mm)		Reinforcement (mm^2)		Ultimate Rotation Capacity (θ_u)	
	Height	Width	Bottom	Top	Bottom	Top
1	400	300	721	1600	0.000236	0.000384
2	300	300	500	1622	0.0003220	0.0005778
3	300	300	360	1080	0.0002828	0.0007455

Column Number	Dimension (mm)		Reinforcement (mm^2)	Ultimate Rotation Capacity (θ_u)
	Height	Width		
1	350	350	2740	0.000953
2	350	350	2160	0.001377
3	300	300	2130	0.001221

After pushover analysis of the structures, performance points of the structures were determined using capacity spectrum method. Then the static criterion (overall drift) was calculated for each structure at the performance points. To determine the relation between the dynamic and static criterion in a large range of the amounts of the damages, five performance levels were considered for each frame. These levels correspond to the average response spectrum, 1.5, 2, 2.5 and 3 times this spectrum. As an example, the calculated performance points for frame S10B5 and decreased spectrums corresponding to the five performance levels have been shown in Figure (9). The values of the static criterion were calculated at theses performance levels. In order to calculate the dynamic criterion for each one of the spectrums, the selected records were scaled and then nonlinear dynamic analysis were performed on the frames subjected to each record. This procedure can be considered a simple incremental dynamic analysis (IDA) using five levels of earthquake excitation. Corresponding to the static criterion at performance point resulting from pushover analysis, the average value of the dynamic damage index was calculated using results of the seven selected earthquakes. That is to say that for every structure, five dynamic damages were calculated. In Figure (10a) and (10b), triangle points are related to corresponding damages to average spectrum plotted in Figure (3) (the design spectrum of Standard 2800), and circle points are related to corresponding damages to one and a half times the average spectrum in Figure (3) (approximately indicating the hazard level ME in ATC40). The rest of the points are indicated by lozenge.

By comparing the static drift criterion, which is introduced as a performance criterion in FEMA-273 [5] and ATC-40 [11] regulations, with the amount of dynamic Park-Ang damage index, the correlation between these two damage indices is investigated. Figure (10) shows this relationship and the fact that the range of changes of triangle and circular points is from 0.673 to 1.25 and from 0.915 to 2.1, respectively. From the achieved results, it can be seen that up to the drift of two percent (life safety definition in FEMA-273), the amount of Park-Ang damage index is less than 0.4 (structure's reparability limit); though it can be found some points higher than 0.4 in the neighborhood around this drift value. The correlation between two criteria shows a high dispersion of points in this static index. This can be the result of the exclusion of the structure's final capacity in this criterion. By fitting a curve, see Figure (10a), which has the best fit to a series of data points, damage can be estimated by a nonlinear optimum approximation

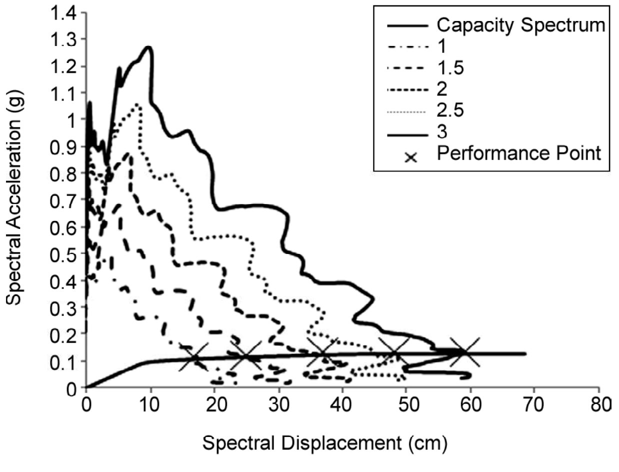
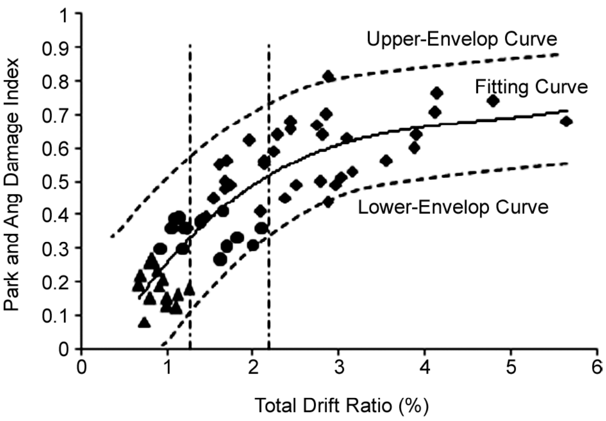
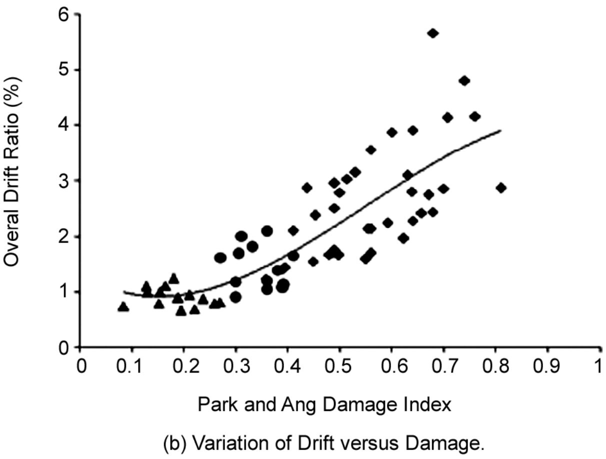


Figure 9. Performance points of the frame S10B5.



(a) Variation of Damage versus Drift



(b) Variation of Drift versus Damage.

Figure 10. Relation between drift criterion and Park-Ang damage index.

from the following equation:

$$DI_{PA} = 0.0068DI_D^3 - 0.0953DI_D^2 + 0.4687DI_D - 0.1236 \quad (3)$$

where DI_{PA} is the damage index and DI_D is the drift criterion. This equation shows that the amount of damage to the structure can be easily estimated only by knowing the drift ratio value obtained from pushover analysis. This can be highly effective and applicable in practical cases.

Figure (10) shows that the sample points of damage are scattered around the function within a strip bounded between an upper-envelope and a lower-envelope curve. Although Eq. (3) can provide an optimum value for the damage index, it may in some situations result in an inadequate estimation of damage. For the purpose of design, considering the safety of the frame structure, the conservative upper-envelop curve is suggested according to the following equation:

$$DI_{PA} = 0.0068DI_D^3 - 0.0953DI_D^2 + 0.4687DI_D - 0.5351 \quad (4)$$

Since slop of the proposed damage equation varies in various regions of the damage curve, different damage levels can be separated. In this regard, three damage levels including low damage (overall drift: 0.6-1.25%), moderate damage (overall drift: 1.25-2.15%) and high damage (overall drift: 2.15-6%) are defined in this research. To present a Table similar to Table (1) for determining the damage state of the structure based on the drift criterion, by fitting a curve for damage considering axis x as the damage index and axis y as the drift criterion, see Figure (10b), following optimum nonlinear equation can be applied to estimate the drift criterion:

$$DI_D = -13.071DI_{PA}^3 + 21.458DI_{PA}^2 - - 5.6292DI_{PA} + 1.3269 \quad (5)$$

Now, by considering the damage limitations in Table (1) and using Eq. (5), Table (8) can be presented to detect damage status of a RCMRF using the drift criterion. This Table can present an effective procedure in predicting the structure's damage status by using proposed criterion.

FEMA-273 gives a set of acceptance criteria for reinforced concrete frames based on certain levels of desired performance. For control of global performance of reinforced concrete moment resisting frames, acceptance criterion is based on drift limits assigned to three levels of performance; IO, LS and CP. The performance levels given in FEMA-273 and elsewhere have similar definitions of physical damage to be expected in the structures. IO, LS and CP performance levels generally correspond to the minor, moderate and severe damage states described in this study. In this context, the drift limits of FEMA-273 were converted to respective damage limits using function 3. The purpose is both to check the reliability of FEMA-273 acceptance limits and recommend more reasonable ranges for drift limits when necessary. By substituting drift limits 1%, 2% and 4% corresponding to IO, LS and CP performance levels of FEMA-273 in function 3, the damage values were obtained 0.26, 0.49 and 0.66 for IO, LS and CP levels, respectively. By referring to Table (1), it is observed that these values show the moderate damage for IO and the severe damage for LS and CP. These results indicate that for the IO and CP level, FEMA-273 limit looks reasonable, whereas drift limits suggested for the LS level is high for the RCMRFs with moderate ductility. Thus, FEMA-273 limitations on the LS level seem to be un-conservative. Therefore, it seems that some existing criteria in conventional seismic regulations need to be reviewed and revised. By assuming the Park-Ang damage index values 0.25, 0.4 and 1 for IO, LS and CP levels; respectively, and using Eq. (5), the following acceptance criteria are suggested according to this study:

- Drift Limitation for Immediate Occupancy (IO) level: 1.06%

Table 8. The relation between the drift criterion and damage state.

Degree of Damage	Damage Index	Physical Appearance	State of Building
Collapse	> 4.1%	Partial or Total Collapse of Building	Loss of Building
Severe	1.7%-4.1%	Extensive Crashing of Concrete; Disclosure of Buckled Reinforcement	Beyond Repair
Moderate	1.1%-1.7%	Extensive Large Cracks; Spalling of Concrete in Weaker Elements	Repairable
Minor	< 1.1%	Minor Cracks; Partial Crushing of Concrete in Columns	Repairable

- Drift Limitation for Life Safety (LS) level: 1.67%
- Drift Limitation for Collapse Prevention (CP) level: 4.08%

7. Summary and Conclusion

In the present research, to achieve a simple and effective criterion with capability of satisfactorily estimating the damage to structure, some damage functions based on the nonlinear responses resulting from pushover analysis were derived. In this regard, conservative upper-envelop relation was proposed for the purpose of design of RCMRFs with control damage based on the drift criterion. In order to present an effective procedure in determining the structure's damage status by using the proposed criterion, a table was suggested. The table is capable of showing damage to concrete frames based on the pushover method. It was shown that FEMA-273 drift limitations on IO and CP performance levels are reasonable; whereas they are un-conservative for LS performance level. However, additional research works need to be performed to challenge the limitations of the performance levels of the codes.

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