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Original Research Article

Confidence level for Iranian Existing Moment Resisting RC Structures Based on Demand and Capacity Factored Design Method

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Abstract

The Demand and Capacity Factored Design (DCFD) method is a performance-based analytic approach used in structural engineering to assess analysis and design factors. This method involves calculating factored demand and capacity to ensure structural safety and reliability. By considering uncertainties in structural modelling and design parameters, the DCFD method provides a probabilistic framework for evaluating the performance of structures under various conditions. In recent decades, Iran has made significant advancements in the design and construction of structures following building codes. As a result, a considerable portion of residential buildings in the country now adhere to the latest building code editions. The Iranian Standard No. 2800, focusing on seismic-resistant building design, stands out as a crucial and impactful code among the various building regulations in Iran. Therefore, the question that emerges is how these regulations will impact the performance of intermediate-moment resisting reinforced concrete structures in upcoming events. In this research, the effectiveness of buildings designed according to the various versions of standard No. 2800 is assessed across operational (OP), immediate occupancy (IO), life safety (LS), and collapse prevention (CP) performance levels through the DCFD method. Structural non-linear analysis is conducted using incremental dynamic analysis. The results show that the confidence level of CP performance level, decreases with increasing height of structures, and the reliability of 8-storey structures is less than 90%. Therefore, it seems necessary to consider CP performance levels in seismic evaluations.

Keywords: Confidence level; Demand and capacity factored design (DCFD); Structure performance prediction; Incremental dynamic analysis.

Nomenclature

DCFD	Demand and Capacity Factored Design
OP	Operational Performance levels
IO	Immediate Occupancy performance levels
LS	Life Safety performance levels
CP	Collapse Prevention performance levels
LRFD	Load and Resistance Factor Design
RC	Reinforced Concrete
CMS	Conditional Mean Spectrum
IDA	Incremental Dynamic Analysis
SD	Standard Deviation
MCE	Maximum Considered Earthquake
M_W	Moment magnitude
g	Earth's gravity acceleration ($g= 9.8\ m/s^2$)
MPa	Mega Pascal

1. Introduction

The demand and Capacity Factored Design (DCFD) Method is a crucial approach used in engineering to assess the performance and reliability of structures based on probability-based load and resistance factors [1]. This method, akin to Load and Resistance Factor Design

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(LRFD), focuses on evaluating the demand placed on a structure against its capacity to withstand that demand. The DCFD method involves calculating factored demand and comparing it to factored capacity, ensuring that the structure can meet specified performance objectives. It addresses uncertainties in design parameters and aims to achieve a balance between demand and capacity to ensure structural integrity [2,3]. The iterative nature of the DCFD method involves assessing designs against performance objectives and modifying them if necessary to meet the desired reliability standards. By utilizing displacement-based parameters like demand and capacity, the DCFD method provides a systematic framework for designing and evaluating structures, offering a probabilistic approach to account for uncertainties and ensure structural safety [4].

Iran is prone to seismic activity and has a history of devastating earthquakes causing significant casualties [5,6]. This necessitates the implementation of stringent earthquake risk mitigation measures across all relevant sectors to minimize such losses [7-10]. The Iranian seismic-resistant design standard for buildings, known as Standard No. 2800, was initially established in 1987 by the Iran Building and Housing Research Center [11]. Over the years, this standard has undergone three significant revisions in 1999, 2005, and 2015 [12-14]. These revisions prompt an examination of the regulatory impact on improving building safety. Furthermore, the comparison between structures designed according to the newer and older editions of the Code highlights the evolution in seismic design practices and the enhanced safety features incorporated in the updated versions.

Several studies have delved into the seismic performance of reinforced concrete (R.C.) buildings designed based on various versions of design codes. Motaghed and Khooshecharkh reported the effects of concrete strength on the damageability of R.C. frames [15]. Kalantari and Roohbakhsh evaluated the seismic fragility of code-conforming RC moment-resisting structures under seismic events [16]. Yazdani et al. Evaluation of Seismic Effect on the concrete frames reliability [17]. Motlagh et al. Evaluated the seismic resilience index for RC structures considering corrosion effects [18]. Fallah Tafti et al. Generate new fragility curves for R.C. structures [19]. Pazuki and Tasnimi evaluated Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels for RC structures designed according to the fourth edition of standard No. 2800, emphasizing the need for further investigation at the CP level [20]. Hariri-Ardebili et al. assessed collapse risk on intermediate moment resisting RC buildings designed per Iranian codes, revealing an increased probability of instability with building height [21]. Sadeghpour and Ozay examined the design reliability and seismic performance factors in Standard No. 2800-99 and Standard No. 2800-05 for RC structures, noting vulnerabilities in building stocks designed based on the second edition [22]. Rezaei and Massumi studied the seismic performance of a multi-story reinforced concrete frame building designed according to the fourth edition of the Iranian seismic code, demonstrating compliance with code requirements [23,24]. Rahbari and Tasnimi explored the design criteria of the Iranian seismic code to enhance compatibility with performance levels, observing changes in the demand capacity ratio and structure damage index across different editions [25]. Evidence from these studies points to the fact that conducting a thorough assessment of structural behaviour across various performance thresholds and evaluating risks can provide insights into how regulations directly influence the safety of structures [26-31].

A comprehensive account of the behaviour of structures at different performance levels and risk assessment can help to understand the exact impact of regulations on structural safety. The researches as mentioned earlier use deterministic methods; contrary to deterministic methods that examine, the probabilistic method, by introducing a variety of structural and seismic uncertainties, allows for a comprehensive assessment of the behaviour of structures and the reliability of assessment. In the present study, we utilize the probabilistic demand and capacity factored design (DCFD) approach to assess how the criteria outlined in standard No. 2800 impact the seismic behaviour of twelve intermediate moment resisting reinforced concrete frames. The evaluation covers different performance and hazard levels, including Operational Performance (OP), IO, LS, and CP in the service, design, and severe earthquakes. The study employs Incremental Dynamic Analysis (IDA) to conduct non-linear structural analysis. Since the selection of ground motion records is an important issue in IDA and it can have a significant effect on the seismic uncertainties [32]; we use the conditional mean spectrum (CMS) method for record selection [33]. The results of this study can be used to design new structures and estimate the safety of existing structures, taking into account structural and seismic uncertainties, as well as examining the need for seismic retrofitting of existing structures according to Probabilistic Seismic Hazard Analysis information.

2. Methodology

The DCFD probabilistic assessment method is used for performance evaluation. The DCFD represents the confidence level that a structure satisfies the performance limit states [34-37]. In the DCFD format, seismic hazard curve, non-linear dynamic response, and structural capacity stochastic models are integrated. The DCFD formulation is expressed as Eq. (1). [34]:

$$\exp\left(-\frac{1}{2}\frac{k}{b}\left(\beta_{CR}^2+\beta_{CU}^2\right)\right)*\hat{C}\geq\exp\left(\frac{1}{2}\frac{k}{b}\left(\beta_{DR}^2+\beta_{DU}^2\right)\right)*\widehat{D}^{P_0} \qquad (1)$$

Eq. (1) is a rewrite of the famous equation as Eq. (2):

$$\Phi\hat{C}\geq\ \gamma\widehat{D}^{P_0} \qquad (2)$$

Based on Eq. (1) and Eq. (2), the confidence parameter (λ) is obtained based on Eq. (3):

$$\lambda = \frac{\gamma.\gamma_a.D}{\Phi.C} \quad (3)$$

where, γ_a is the analysis uncertainty factor that accounts for the bias and uncertainty of mathematical procedures used to estimate seismic demand (D) as the function of strong ground motion intensity. γ is the demand factor that signifies the intrinsic variability in demand prediction resulting from structural modelling assumptions and seismic record discrepancies. γ is derived from Eq. (4):

$$\gamma = \exp\left(\frac{k}{2b}\beta_{RD}^2\right) \quad (4)$$

And γ_a is calculated as Eq. (5):

$$\gamma_a = \exp\left(\frac{k}{2b}\beta_{UD}^2\right) \quad (5)$$

Φ is the resistance factor that considers the effect of randomness and the inherent uncertainty in the estimation of C. Φ is calculated as Eq. (6):

$$\Phi = \Phi_{RC}.\Phi_{UC} = \exp\left(-\frac{k}{2b}\right)(\beta_{RC}^2 + \beta_{UC}^2) \quad (6)$$

where, Φ_{RC} indicates the variability of the earthquake records and Φ_{UC} states the inherent uncertainty of C. β_{UT}, the total uncertainties in C and D are calculated as Eq. (7):

$$\beta_{UT} = \sqrt{\sum_i \beta_{ui}^2} \quad (7)$$

where β_{ui} is the standard deviation (SD) of the natural logarithm of C and D, considering all uncertainties. The confidence factor, denoted as Kx, calculates the confidence level that the structure meets the performance objective as shown in Eq. (8):

$$K_x = \left[\frac{1}{2}k.\beta_{UT}^2 - \ln\lambda\right]\frac{1}{\beta_{UT}} \quad (8)$$

where, K_x is the standard normal random variable. With K_x from Eq. (8), the confidence level is calculated as shown in Eq. (9):

$$X = \Phi(K_x) \quad (9)$$

3. Structural models

Twelve intermediate moment resisting reinforced concrete frames ranging from 3 to 8 stories (comprising three and five spans) have been designed in a highly seismic area within the Tehran metropolitan region, following the guidelines outlined in the third and fourth editions of the Iranian seismic code [13,14]. A complete description of the geometry, loading, and regulations included in the buildings is available in reference 10. To avoid repetition, here only the summary of the specifications of the structures is given. The gravity loads are expected to mirror those found in typical residential buildings in Iran, as referenced [38-41]. We assumed that these structures are situated on soil type B, according to the requirements outlined in Standard No. 2800. The height of each story is 3 meters. The design of all the structures includes two equal side spans measuring 4 meters each, along with a central span of 5 meters. Each frame is considered to be an integral part of the building's lateral load-resisting system, with rigid diaphragms assumed for all frames. Furthermore, these structures exhibit symmetry in their floor plans and regularity in their elevations. The material's properties are detailed in Table 1.

Table 1. Material characteristics of RC structures

parameter	notation	unit	value
concrete compressive strength	f_c	MPa	25
longitudinal bars yielding strength	f_y	MPa	400
shear bars yielding strength	f_{ys}	MPa	300
concrete elasticity modulus	E_C	MPa	2.5×10^4
steel elasticity modulus	E_S	MPa	2.1×10^5

Fig. (1) Shows the plans and elevations of the structures. There are significant variations in the factors influencing the calculation of equivalent static lateral forces across various versions of Standard No. 2800. This difference in S_a and the distribution of forces in height and plan and drift control requirements has caused differences in the structure elements. Additional details about the structures and elements can be found in reference 10.

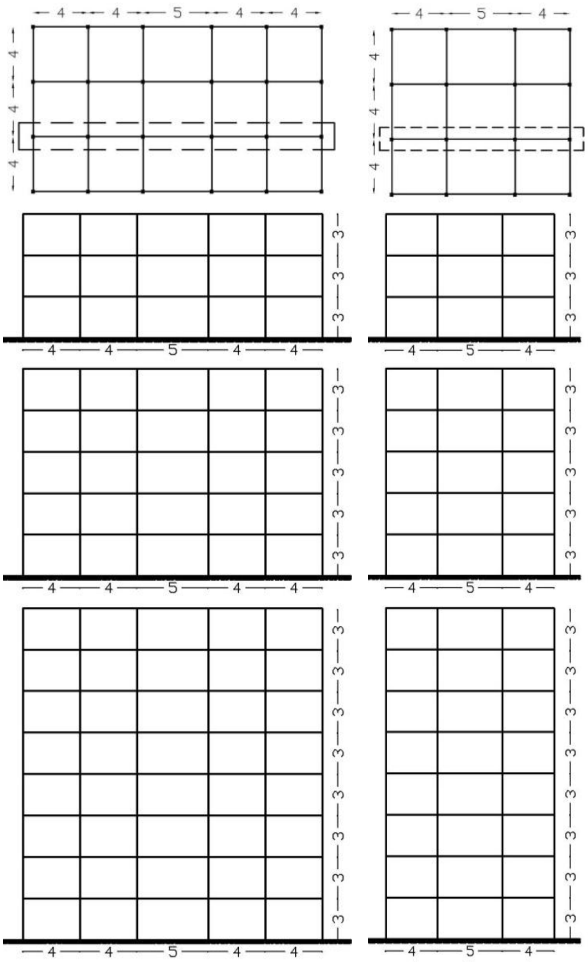

Figure 1. Plan and elevation of the structures (3, 5 and 8 stories)

4. Analyses, Results, and Discussion

In this research, we employed the Incremental Dynamic Analysis (IDA) technique for conducting structural analysis [34, 42-44]. The selection of ground motion records is an important issue in IDA. Previous studies show that 20 earthquake records are required for middle-height structure analysis [43]. The selected records should assess the possibility of structural collapse in the maximum considered earthquake (MCE) of seismic zones. We use the conditional mean spectrum (CMS) method for record selection [33]. In this method, regional characteristics, including magnitude and distance, as well as the spectral shape, are considered dominant parameters in record selection. The CMS method incorporates the aleatory and epistemic uncertainties in earthquake events.

Based on the probabilistic seismic hazard deaggregation result [44], we obtain the CMS and spectra of the records (Figure 2). The magnitude and peak ground-motion acceleration of the earthquakes are from 4.5 to 7.5 (M_W) and 0.05g to 1g, respectively (Table 2).

Table 2. Selected records for IDA (soil type II)

Event	station	P	dista	Magnit
Tabas	Dayhook	0.327	13.9	7.35
Manjil	Abbar	0.514	12.5	7.37
San	Pasadena Cit	0.11	25.4	6.61
San	Lake Hughes #9	0.134	22.5	6.61
Kern	Taft Lincoln School	0.178	38.8	7.36
Morgan	San Justo Dam (L	0.081	31.8	6.19
Morgan	Gilroy - Gavilan	0.114	14.8	6.19
Hector	Twentynine Palms	0.066	42.0	7.13
Sierra	LA - City Terrace	0.091	25.6	5.61
Loma	Anderson Dam	0.244	20.2	6.93
Loma	Fremont - Mission	0.106	39.5	6.93
Loma	Gilroy Array #6	0.126	18.3	6.93
Loma	Gilroy Array #6	0.17	18.3	6.93
Loma	Monterey City Hall	0.073	44.3	6.93
Northridg	Arcadia - Campus	0.089	41.4	6.69
Northridg	Arcadia - Campus	0.11	41.4	6.69
Northridg	Alhambra - Fremont	0.08	36.7	6.69
Northridg	N Hollywood -	0.271	12.5	6.69
Northridg	La Crescenta - New	0.159	18.5	6.69
Northridg	LA - Chalon Rd	0.225	20.4	6.69

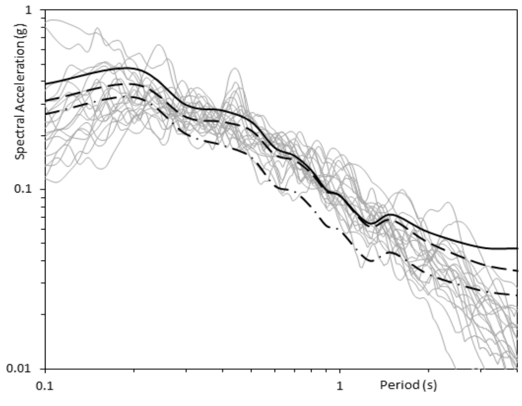

Figure 2. Regional design spectrum and spectra of selected records (CMS)

This research opts for 5% damped first-mode S_a ,(Sa.T1, 5%) as the intensity measure and maximum interstory drift ratio (θmax) as the damage measure.

We utilize IDARC2D (Version 7.0) [45] for non-linear time history analysis. The Park hysteretic model with parameters for stiffness degradation, strength deterioration, non-symmetric response, slip-lock, and a tri-linear monotonic envelope is employed. Values for hysteretic parameters (Stiffness degradation parameter (HC), Strength deterioration parameter (HBD, HBC), and slip-lock parameter (HS)) are used for intermediate moment resisting RC frames [46].

Based on Yun et al. probabilistic framework [36], structural limit states need to be defined for IDA curves. The estimation of the confidence level and performance objective are based on FEMA-350 [35]. Four performance levels are interested in this study:

OP with hazard level 99.5% probability of exceedance in 50 years (service earthquake)

IO with hazard level 50% probability of exceedance in 50 years

LS with hazard level 10% probability of exceedance in 50 years (design earthquake)

CP with hazard level 2% probability of exceedance in 50 years (severe earthquake)

Based on standard No.2800, the drift ratios of OP and LS performance levels are 0.005 and 0.025, respectively. The values assigned to LS vary in different editions. In the third edition, the value is determined based on the fundamental period, with a value of 0.025 for periods less than 0.7 seconds and 0.02 for longer periods. In contrast, the fourth edition assigns values based on the number of stories; structures up to 5 stories have a value of 0.025, while taller structures have a value of 0.02. Since standard No. 2800 does not specify, this study defines IO and CP limit states following FEMA guidelines. According to FEMA-350, the drift ratio for the IO limit state is set at 0.01, while for the CP limit state, it is determined as the lower of two values: the point on the IDA curve where the local tangent drops below 20% of the initial slope or a maximum drift ratio of 10%. A visual representation of this process is provided in Fig. 3 for a 3-storey, 3-bay structure. The details of CP limit state values can be found in Table 3.

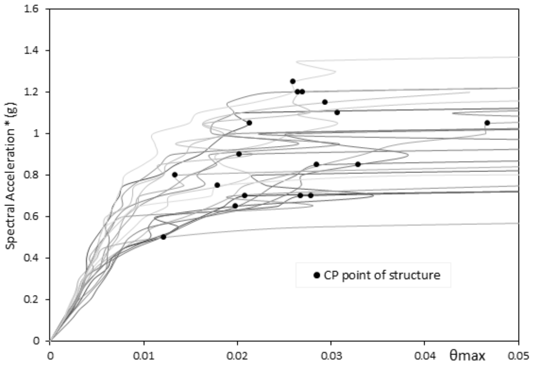

Figure 3. IDA of 3 story- 3 span structure and *CP* points based on FEMA-350

In Table 3, $\hat{C}$, β_{CR}, $S_a^{\hat{C}}$, and β_C are median capacity, SD of displacement-based capacity, S_a corresponding to median capacity and SD of S_a, respectively.

Table 3. Results of *CP* limit state based on FEMA-350

code	building	θ_{max}		$Sa_{T_1,5\%}$	
		$\hat{C}$	β_{CR}	$S_a^{\hat{C}}$	β_C
Third Edition	**3 story- 3 bay**	0.0278	0.378	0.896	0.280
	3 story- 5 bay	0.0328	0.455	0.807	0.366
	5 story- 3 bay	0.0306	0.500	0.580	0.304
	5 story- 5 bay	0.0306	0.396	0.576	0.458
	8 story- 3 bay	0.0300	0.519	0.465	0.426
	8 story- 5 bay	0.0295	0.513	0.371	0.420
Fourth Edition	**3 story- 3 bay**	0.0306	0.348	1.017	0.526
	3 story- 5 bay	0.0331	0.417	0.855	0.411
	5 story- 3 bay	0.0426	0.461	0.980	0.511
	5 story- 5 bay	0.0427	0.425	0.997	0.575
	8 story- 3 bay	0.0348	0.470	0.638	0.492
	8 story- 5 bay	0.0351	0.437	0.526	0.503

The seismic hazard parameters according to the hazard analysis (Eq. (11) and Eq. (19)) are presented in Table 4 [32,47]. In this study, the S_as corresponding to a 10% and 2% probability of exceedance over 50 years serve as the regression points.

Table 4. Seismic hazard parameters

Structures	Third Edition		Fourth Edition	
	k_0	k	k_0	k
3 storey-	2688.6	2.519	2688.6	2.519
3 storey-	2688.6	2.519	2688.6	2.519
5 storey-	992.27	2.468	1041.1	2.383
5 storey-	992.27	2.468	1041.1	2.383
8 storey-	180.16	2.275	345.19	2.297
8 storey-	180.16	2.275	345.19	2.297

Table 5 shows a and b values for considered structures.

Table 5. structural parameters based on power law method

Structure	Third Edition		Fourth Edition	
	a	*b*	*a*	*b*
3 storey- 3 bay	0.031	1.356	0.023	1.326
3 storey- 5 bay	0.035	1.404	0.027	1.261
5 storey- 3 bay	0.050	1.353	0.034	1.348
5 storey- 5 bay	0.057	1.330	0.032	1.323
8 storey- 3 bay	0.036	0.851	0.045	1.191
8 storey- 5 bay	0.038	0.846	0.050	1.214

The seismic assessment of structures in DCFD format is presented in tables (6) and (7). Also, in order to make a clearer comparison, the results of the confidence levels of the structures are shown in Fig. 4.

The findings indicate that structures conforming to both editions of standard No.2800 demonstrate satisfactory performance levels in terms of OP and IO, with a confidence level exceeding 90%.

Table 6. Calculation of λ and confidence level for structures designed according to the third edition of standard No.2800

	Structure	C	D	$\gamma * \gamma_a$	Φ	$\gamma * \gamma_a * D$	$\Phi * C$	β_{UT}	λ	K_x	C. L. %
OP	**3 storey- 3 bay**	0.005	0.0006	1.0411	0.895	0.00066	0.0044	0.2	0.149	9.692	100.0000
	3 storey- 5 bay	0.005	0.0006	1.0510	0.898	0.00065	0.0045	0.2	0.146	9.779	100.0000
	5 storey- 3 bay	0.005	0.0007	1.1001	0.879	0.00078	0.0044	0.26	0.178	6.873	100.0000
	5 storey- 5 bay	0.005	0.0008	1.0845	0.877	0.0009	0.0044	0.26	0.212	6.201	100.0000
	8 storey- 3 bay	0.005	0.0012	1.1532	0.840	0.0014	0.0042	0.26	0.341	4.465	99.9999
	8 storey- 5 bay	0.005	0.0014	1.2455	0.832	0.0017	0.0042	0.26	0.405	3.814	99.9931
IO	**3 storey- 3 bay**	0.01	0.0019	1.0411	0.895	0.0019	0.0089	0.2	0.223	7.685	100.0000
	3 storey- 5 bay	0.01	0.0019	1.0510	0.898	0.0020	0.0089	0.2	0.227	7.579	100.0000
	5 storey- 3 bay	0.01	0.0021	1.1001	0.879	0.0023	0.0087	0.26	0.272	5.250	99.9999
	5 storey- 5 bay	0.01	0.0025	1.0845	0.877	0.0027	0.0087	0.26	0.318	4.652	99.9998

	8 storey- 3 bay	0.01	0.0028	1.1532	0.840	0.0033	0.0084	0.26	0.387	3.965	99.9963
	8 storey- 5 bay	0.01	0.0029	1.2455	0.832	0.0037	0.0083	0.26	0.441	3.488	99.9756
LS	**3 storey- 3 bay**	0.025	0.0052	1.0466	0.881	0.0055	0.022	0.25	0.251	5.757	99.9999
	3 storey- 5 bay	0.025	0.0055	1.0563	0.884	0.0058	0.0221	0.25	0.265	5.529	99.9999
	5 storey- 3 bay	0.02	0.0061	1.1105	0.855	0.0067	0.0171	0.33	0.396	3.107	99.9056
	5 storey- 5 bay	0.02	0.0071	1.0949	0.853	0.0078	0.0171	0.33	0.455	2.689	99.6417
	8 storey- 3 bay	0.02	0.0061	1.1680	0.808	0.0071	0.0162	0.33	0.437	2.914	99.8218
	8 storey- 5 bay	0.02	0.0061	1.2024	0.799	0.0076	0.0160	0.33	0.456	2.812	99.7541
CP	**3 storey- 3 bay**	0.0278	0.0127	1.1760	0.812	0.0149	0.0226	0.3	0.662	1.671	95.2712
	3 storey- 5 bay	0.0328	0.0137	1.2849	0.769	0.0177	0.0252	0.3	0.702	1.468	92.8996
	5 storey- 3 bay	0.0306	0.0176	1.0895	0.751	0.0192	0.0229	0.4	0.836	0.757	77.5516
	5 storey- 5 bay	0.0306	0.0188	1.1484	0.788	0.0216	0.0241	0.4	0.895	0.621	73.2587
	8 storey- 3 bay	0.0300	0.0168	1.0972	0.745	0.0184	0.0223	0.4	0.826	0.778	78.2219
	8 storey- 5 bay	0.0295	0.0197	1.0925	0.763	0.0215	0.0225	0.4	0.958	0.389	65.1103

Table 7. Calculation of λ and confidence level for structures designed according to the fourth edition of standard No.2800

	Structure	C	D	$\gamma * \gamma_a$	Φ	$\gamma * \gamma_a * D$	$\Phi * C$	β_{UT}	λ	K_x	C.L.%
OP	**3 storey- 3 bay**	0.005	0.0005	1.0339	0.892	0.00052	0.0045	0.2	0.117	10.907	100.0000
	3 storey- 5 bay	0.005	0.0007	1.0527	0.886	0.00077	0.0044	0.2	0.175	8.902	100.0000
	5 storey- 3 bay	0.005	0.0006	1.0512	0.883	0.00061	0.0044	0.26	0.138	7.842	100.0000
	5 storey- 5 bay	0.005	0.0006	1.0541	0.881	0.00061	0.0044	0.26	0.139	7.816	100.0000
	8 storey- 3 bay	0.005	0.0008	1.1894	0.873	0.00096	0.0043	0.26	0.222	6.043	99.9999
	8 storey- 5 bay	0.005	0.0008	1.1426	0.875	0.0009	0.0044	0.26	0.219	6.080	99.9999
IO	**3 storey- 3 bay**	0.01	0.0014	1.0339	0.892	0.0015	0.0089	0.2	0.171	9.021	100.0000
	3 storey- 5 bay	0.01	0.0020	1.0527	0.886	0.0021	0.0088	0.2	0.243	7.281	100.0000
	5 storey- 3 bay	0.01	0.0018	1.0512	0.883	0.0019	0.0088	0.26	0.218	6.084	99.9999
	5 storey- 5 bay	0.01	0.0018	1.0541	0.881	0.0019	0.0088	0.26	0.215	6.138	100.0000
	8 storey- 3 bay	0.01	0.0023	1.1894	0.873	0.0027	0.0087	0.26	0.318	4.651	99.9998
	8 storey- 5 bay	0.01	0.0024	1.1426	0.875	0.0028	0.0087	0.26	0.321	4.610	99.9997
LS	**3 storey- 3 bay**	0.025	0.0040	1.0395	0.878	0.0041	0.0219	0.25	0.188	6.916	100.0000
	3 storey- 5 bay	0.025	0.0052	1.0587	0.872	0.0055	0.0218	0.25	0.254	5.721	99.9999
	5 storey- 3 bay	0.025	0.0053	1.0609	0.859	0.0056	0.0214	0.33	0.262	4.341	99.9993

	5 storey- 5 bay	0.025	0.0051	1.0640	0.856	0.0054	0.0214	0.33	0.254	4.442	99.9995
	8 storey- 3 bay	0.02	0.0062	1.2014	0.847	0.0074	0.0169	0.33	0.440	2.804	99.7474
	8 storey- 5 bay	0.02	0.0066	1.1538	0.850	0.0076	0.0170	0.33	0.453	2.714	99.6676
CP	**3 storey- 3 bay**	0.0306	0.0094	1.0524	0.838	0.0099	0.0257	0.3	0.388	2.816	99.9710
	3 storey- 5 bay	0.0331	0.0129	1.2362	0.786	0.0159	0.0260	0.3	0.615	1.922	97.2688
	5 storey- 3 bay	0.0426	0.0134	1.1211	0.750	0.0150	0.0320	0.4	0.469	2.240	98.7455
	5 storey- 5 bay	0.0427	0.0125	1.1320	0.751	0.0142	0.0321	0.4	0.443	2.419	99.2200
	8 storey- 3 bay	0.0348	0.0191	1.0805	0.775	0.0225	0.0270	0.4	0.833	0.755	83.5583
	8 storey- 5 bay	0.0351	0.0207	1.2218	0.794	0.0253	0.0279	0.4	0.907	0.539	70.5164

At the LS performance level, it is evident that different editions' 3-story structures exhibit comparable performance with a confidence level exceeding 99.99%. The 5-story structures in the fourth edition demonstrate superior performance and confidence levels due to a reduction in seismic demand and an increase in capacity compared to the third edition, with capacity values improving from 0.02 to 0.025. Analysis from DCFD results indicates that the 8-story structures in the third edition are more reliable than those in the fourth edition. Despite these variations, the results affirm the overall good performance of structures in both editions at this performance level. In terms of CP performance, the results suggest that as the height of structures increases, their confidence levels decrease. Higher seismic demand at this performance level leads to increased uncertainty in seismic demand and capacity, resulting in a convergence of demand and capacity factors (λ approaching 1) and subsequently lowering confidence levels [48]. While fourth edition 3-story structures exhibit higher confidence levels than third edition structures, both editions surpass the minimum recommended value by FEMA-350. In 5 and 8-story structures, the demand coefficient is less than the capacity coefficient ($\lambda < 1$ for both editions), although the confidence levels of these results may be subject to debate. The confidence level of 5-story structures in the fourth edition exceeds 90%, while the third edition falls below this threshold, indicating insufficient safety according to FEMA-350 due to capacity differences between the two editions. For 8-story structures, despite the improved performance of fourth edition structures, confidence levels in both editions remain below 90%, suggesting inadequate safety. Overall, the absence of consideration for CP performance in the design process of standard No.2800 implies insufficient reliability at this performance level, highlighting the need to incorporate CP control criteria in structural design standards. The study findings emphasize that intermediate moment resisting RC frames perform satisfactorily at the IO and LS performance levels as outlined in Regulation 2800. However, the CP performance is compromised by increased structure height. Therefore, it is advisable to evaluate the CP threshold, a measure of structural mortality, in the Iranian Seismic Code for enhanced structural safety and performance evaluation.

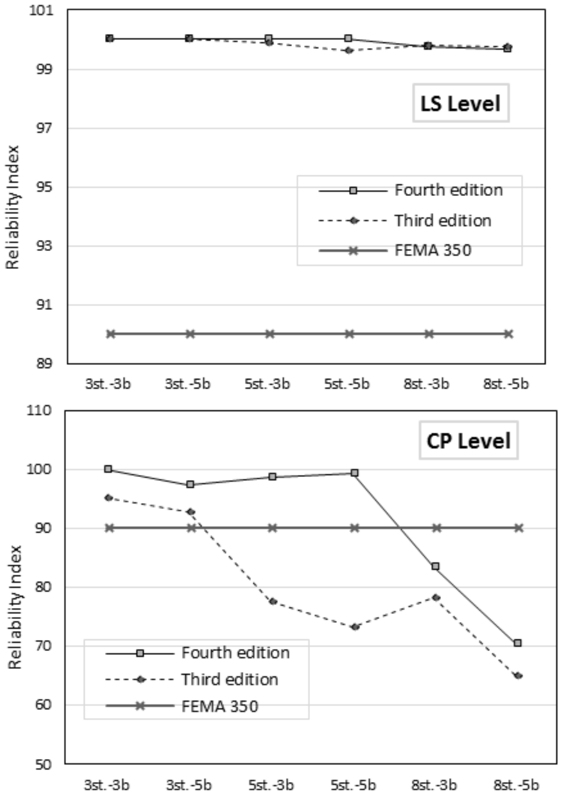

Figure 4. Confidence levels of Iranian R.C structures

5. Conclusions

DCFD format aims to balance demand and capacity factors by integrating factors like structural modelling uncertainty, limit state probability, and spectral acceleration to derive a design criterion for a given allowable probability level. The DCFD method offers a systematic way to assess and optimize structural designs based on probabilistic considerations, enhancing the overall safety and performance of engineered structures.

In this study, we have used the DCFD method to evaluate the seismic performance of existing buildings. DCFD evaluations indicate that structures designed according to standard No.2800 demonstrate satisfactory performance in OP, IO, and LS performance levels, with a minimum confidence level of 90%. The analysis further shows that structures designed based on the fourth edition generally exhibit superior performance in terms of OP, IO, LS, and CP performance levels, except for 8-story buildings at the LS level due to the influence of hazard parameters on confidence level calculations. Regarding CP performance, the study suggests that as the height of the structure increases, the confidence level decreases, potentially leading to non-compliance with FEMA-350 criteria for taller structures, indicating inadequate reliability at the CP performance level. Therefore, it is recommended to follow the guidelines outlined in Standard No. 2800 to manage CP levels during structural design. Additionally, since the CP level is associated with mortality risk, it is advised to conduct seismic assessments and implement necessary retrofitting measures for existing buildings at this performance level.

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Conflict of Interests

No conflict of interest has been expressed by the authors.

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