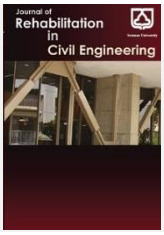


Journal of Rehabilitation in Civil Engineering

Journal homepage: https://civiljournal.semnan.ac.ir/

CNCRP Manual Based Seismic Evaluation and Retrofitting of Existing RC Buildings: A Case Study

Debasish Sen [1]; Md. Kamruddoja Shihab [2]; Md. Farhad Momin [3,*]

1. Assistant Professor, Department of Civil Engineering, Ahsanullah University of Science and Technology, Bangladesh
2. M. Eng. Student, Department of Civil Engineering, Ahsanullah University of Science and Technology, Bangladesh
3. Lecturer, World University of Bangladesh, Bangladesh
* Corresponding author: *farhadmomin.ce@gmail.com*

ARTICLE INFO

Article history:
Received: 10 January 2024
Revised: 04 May 2024
Accepted: 01 August 2024

Keywords:
Seismic assessment;
Seismic evaluation;
RC building;
RC jacket;
Retrofit.

ABSTRACT

Recent seismic incidents in Turkey have raised concerns among engineers about the potential performance of existing buildings in earthquakes. Due to its location in a somewhat active seismic region, Bangladesh is susceptible to experiencing catastrophic damage if a heavy to moderate earthquake hits. Therefore, the assessment of the seismic capacity of existing buildings is essential to investigate potential risks. Before this, Bangladesh did not have a seismic assessment or retrofitting guideline. However, the Public Works Department (PWD) of Bangladesh has now adopted the seismic assessment procedure from the Japan Building Disaster Prevention Association Manual (JBDPA, 2001) and published it as CNCRP (2015). On top of that, the building design code of Bangladesh (BNBC) has been updated, and the calculation of seismic demand has also changed. This study aims to assess an existing RC building located in Bangladesh for local seismic loads and retrofit it if found vulnerable. A six-storeyed RC building was assessed in detail. The building's seismic capacity was calculated using the suggestions of the CNCRP seismic assessment manual (2015), which depends on the strength and ductility capacity of structural members. The storey collapse mechanism was considered; therefore, the capacity of each storey was computed by considering the lateral capacities (both strength and ductility) of all columns in that storey. The columns were classified according to tributary area, and their failure mechanism was identified. The seismic index was computed utilizing the strength and ductility correlations. After evaluation, some floors were found vulnerable, and the necessary reinforcement specifications were computed following the CNCRP retrofit manual (2015). The retrofit design was established, and a comprehensive re-examination was conducted. Ultimately, the building was found safe according to the CNCRP seismic assessment manual (2015) after the retrofit.

E-ISSN: 2345-4423

How to cite this article: Sen, D., Shihab, M. K., & Momin, M. F. (2025). CNCRP Manual Based Seismic Evaluation and Retrofitting of Existing RC Buildings: A Case Study. Journal of Rehabilitation in Civil Engineering, 13(2), 187-204. https://doi.org/10.22075/jrce.2024.32938.1980

1. Introduction

The susceptibility of existing reinforced concrete (RC) structures in places with a history of seismic activity has emerged as a significant concern. A significant number of those structures were constructed without the consideration of any building codes or some obsolete building codes and inadequate design standards, rendering them vulnerable to potential damage or collapse in the event of powerful seismic activity [1]. The aforementioned phenomenon poses probable significant hazards to human lives and urban infrastructure located in regions characterized by high seismic activity.

The recent seismic events, such as the earthquakes that occurred in Turkey's Eastern Anatolian Fault Zone in February 2023, were very destructive, surpassing the effects of the 1999 Marmara earthquake. These seismic events caused extensive destruction and losses of human lives and triggered many aftershocks in 11 provinces in the eastern region [2]. A reconnaissance report [3] discussed that buildings constructed in Turkey before the year 2000 often displayed many shortcomings, such as the presence of smooth rebar, insufficient reinforcing, weak stories, exposed ground floors, and substandard beam-column connections. These problems have been identified as significant contributors to the extensive damage and collapse patterns seen in previous seismic events (Kocaeli earthquake, 1999; Van earthquake, 2011). Despite being constructed in accordance with modern codes, more than 1000 buildings built after 2000 in Turkey were severely damaged or collapsed during the 2023 earthquakes. This can be attributed to various factors such as inadequate engineering practices, construction errors, underestimation of seismic demands, weak foundations, and excessive utilization of flexible floor systems [3]. These findings suggest the necessity of reevaluating performance objectives and implementing a greater number of shear walls in future construction projects. Another study [4] argued that brittle shear failures in old Turkish structures during the 2023 earthquakes were attributed to many deficiencies, including insufficient concrete strength, smooth rebar, inadequate confinement steel, and a lack of boundary features. Additionally, some instances experienced foundation settlement and overturning due to liquefaction-induced effects [4]. Considering the substantial structural damage seen in recent earthquakes in Turkey, it is pretty clear that the seismic assessment of the existing buildings is very important.

Bangladesh is situated in close proximity to at least three of the most active tectonic plates in the world [5] and has experienced at least five major earthquakes that measured over 7.0 on the Richter scale in the last 150 years [6]. The seismic events under consideration exhibited epicenters that were found to be situated at distances not exceeding 150 km from the urban center of Dhaka [7]. Over the last several years, Bangladesh has seen a series of seismic events ranging from mild to intense, characterized by magnitudes ranging from 4.0 to 6.2. The seismic events have resulted in significant structural impairment, such as the occurrence of building collapses in Chittagong in 1997 that resulted in several fatalities. Additional consequences included the impairment of cyclone shelters during the year 1999 as well as the occurrence of structural fractures in structures in 2003. In the most recent occurrence, a seismic event measuring 6.2 on the Richter scale occurred in close proximity to Chittagong in the year 2021, resulting in some infrastructural damage. Instances of minor earthquakes have resulted in widespread fear among the affected population. The recurring occurrence of seismic activity remains a cause for concern as a result of inherent weaknesses in structural integrity, as a significant number of structures were constructed without the implementation of a comprehensive seismic catastrophe prevention strategy [6]. Several researchers [8,9] took initiatives to quantify the seismic hazards in the region near Bangladesh. Therefore, the assessment of seismic susceptibility in existing reinforced concrete buildings in Bangladesh has great significance.

Seismic assessment can be conducted by several methods (detailed seismic evaluation method, non-linear static analysis, or dynamic analysis) [10–12]. Among them, non-linear static analysis and dynamic analysis require modelling of each beam, column considering its non-linearity (i.e., moment-curvature relationship) at hinge locations. However, the modelling of each beam and column of RC buildings with the actual reinforcement configuration is very difficult in commercial software (e.g., ETABS, SAP2000, etc.). The user-defined non-linear hinge modelling of RC members needs section-specific moment-curvature relationships. The computation of such relationships is time consuming for a multi-storeyed building. On the contrary, detailed seismic evaluation methods offer alternative seismic assessment where manual computation of structural components' lateral strength and ductility is required. There are several established guidelines to assess an existing building's performance against lateral loads. Several standards, ASCE 31 [13], ASCE 41 [14], FEMA P-154 [15], JBDPA [16], etc., are well established and used frequently in the field of seismic assessment. FEMA P154 [15] suggests a rapid visual screening (RVS) method where the performance score of an existing building can be estimated, which can predict the probability of collapse of that building. In this method, the final score is provided by the summation of the basic score and the score modifier summation. The basic score of this method depends on structure types, seismic region with respect to spectral acceleration response, and soil types. On the other hand, score modifiers depend on the number of stories, building irregularities, pre-code or post-benchmark code detailing, and soil type. However, this evaluation method does not consider individual beam and column capacity. Therefore, it is not exactly computed structural capacity; therefore, the FEMA P154 [15] ultimate score has limited ability to correlate with actual seismic capacity and actual damages [17]. Nevertheless, it can be utilized for the prioritization of existing buildings for detailed evaluation. ASCE/SEI 31 [13] is a method for evaluating existing buildings using a three tier approach. The first tier is the screening phase; a second tier is the evaluation phase; and the third tier is the detailed evaluation phase. In the first-tier evaluation, the structure is identified with respect to the basic constructive parameters and assessed for potential deficiencies using the corresponding checklists. If it is determined that certain parts of the building do not meet the first-tier checklist standards, the second-tier evaluation can be done on individual elements or the entire building. If there is any doubt that the first and second tiers are too conservative for a realistic review and that a more comprehensive study is required, the third tier of evaluation is used. It can be done for the entire structure or for individual parts that do not meet the second-tier standards. These evaluation processes consider the effects of ground shaking to a minor extent, additional seismic hazards such as liquefaction, slope failure, surface fault rupture, and the impacts of surrounding structures. Although ASCE 31 [13] offers seismic evaluation guidelines, it has some limitations. The utilization of pseudo-lateral forces in the standard may not be in accordance with contemporary design methodologies [18]. JBDPA [16] offers a three-level seismic evaluation. The first-level evaluation is simple in nature, where column capacity is required to be computed to assess the building capacity. Whereas a second-level evaluation needs detailed calculations where structural drawings are required. Third-level evaluation needs numerical modelling. However, the seismic demand is considered based on the seismic capacity index of safe buildings in past earthquakes in Japan. Therefore, JBDPA [16] is difficult to utilize in other countries. In this context, a new guideline has been developed in Bangladesh, named CNCRP Assessment Manual [19], which integrates approaches that were suggested by the JBDPA [16] with some modifications to make it more relevant in the context of Bangladesh. While seismic demand in the Japanese seismic evaluation method [16] is calculated based on previous earthquake damage data, the CNCRP manual [19] can evaluate vulnerability based on local seismicity.

Several researchers [20–28] taken initiatives to assess the performance of existing infrastructure like buildings, tunnels, etc. through non-linear modeling, machine learning methods, deep learning models etc. Few researchers [29,30] studied detailed seismic evaluation methods also. However, there is a gap in the application of CNCRP-based detailed seismic evaluation. The CNCRP-based evaluation may be a useful tool for evaluating existing RC buildings with less computational requirements (i.e., no need for non-linear modelling and analysis).

This study aims to evaluate an existing RC building located in Bangladesh, designed for both gravity and lateral loads, to check its vulnerability under seismic loading according to the CNCRP seismic assessment manual [19] and to design a retrofit using the CNCRP retrofit manual [31], if necessary. The applied seismic evaluation method [19] has a unique feature of vulnerability assessment based on local seismic demand computation, while a similar seismic evaluation method [16] utilizes previous earthquake damage data for seismic demand computation. To achieve this, initially, the seismic capacity of the building was assessed using detailed structural drawings and material properties and compared to the local seismic demand in accordance with the CNCRP seismic assessment manual [19]. The building has been found seismically vulnerable, and therefore a retrofit design has been conducted considering RC column jacketing according to the provisions of the CNCRP retrofit manual [31]. Finally, the retrofitted building has been assessed to confirm its safety against local seismic loads.

2. CNCRP assessment and retrofitting manual

Recently, the Public Works Department of Bangladesh has adopted the Japanese second-level seismic evaluation procedure [16], with some improvisation, and published it as the CNCRP manual [19], [31]. Most of the procedures are the same in these two guidelines, but there are some modifications made in the CNCRP manual to acknowledge the seismic loading and construction scenarios in Bangladesh. The CNCRP manual consists of two different guidelines: one for the seismic evaluation [19] and the other for the retrofit design [31] of the vulnerable buildings. The seismic capacity assessment procedure is discussed in the subsequent sub-sections. The steps of the adopted seismic evaluation and retrofitting procedure are shown in Figure 1

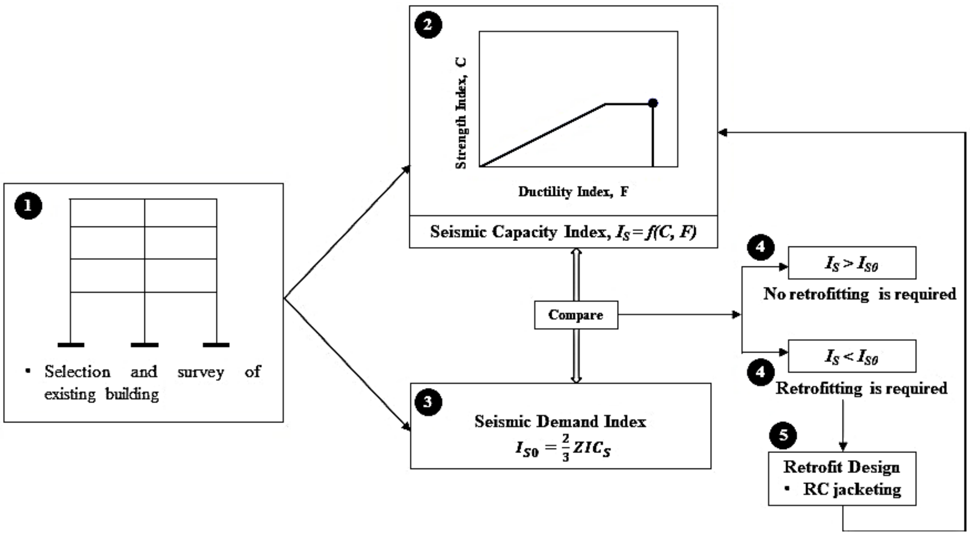


Fig. 1. Flow chart of CNCRP-based seismic evaluation and retrofitting procedure adopted in this study.

2.1. Seismic index

The seismic index, which is a representation of the seismic capacity of a building, is calculated using Equation 1 suggested in the CNCRP seismic assessment manual [19].

$$I_S = E_0 \cdot S_D \cdot T \tag{1}$$

where, I_S = Seismic index of structure, E_0 = basic seismic index of structure, S_D = irregularity index, and T = time index

2.2. Basic seismic index, e_0

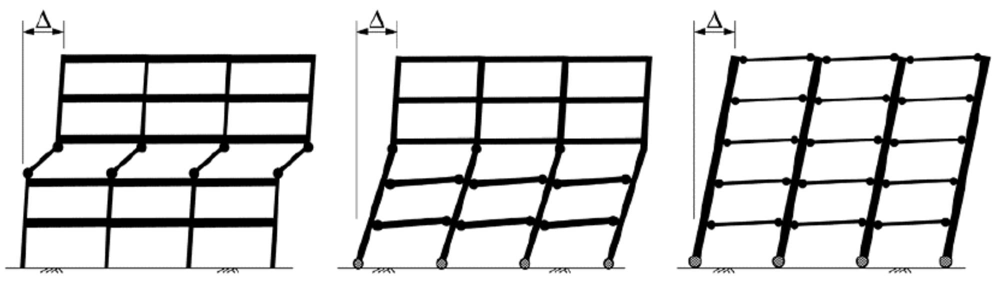
The assessment of the seismic performance of the building involves the calculation of the basic seismic index of the structure, denoted as E_0. The determination of E_0 is conducted at each floor level and in both principal directions, considering the structural element's ultimate capacity, failure mechanism, and plastic deformation capacity, etc. The basic seismic index (E_0) computation depends on the structural elements' strength capacity (C) and ductility (F). The basic seismic index for ductility-dominant and strength-dominant structures can be calculated using Equations 2 and 3, respectively. The use of a storey shear modification factor $\frac{n+1}{n+i}$ is employed to account for variations in seismic capacity at different floor levels.

$$E_0 = \frac{n+1}{n+i} \cdot \sqrt{(C_1F_1)^2 + (C_2F_2)^2 + (C_3F_3)^2} \tag{2}$$

$$E_0 = \frac{n+1}{n+i} \cdot \left(C_1 + \sum_j \alpha_j C_j\right) \cdot F_1 \tag{3}$$

where, C_1, C_2, C_3 = strength indices of 1st, 2nd, 3rd group, respectively; F_1, F_2, F_3 = ductility indices of 1st, 2nd, 3rd group, respectively; and α = effective strength factor.

In the basic seismic index calculation, an expected failure mechanism is required. An RC moment resisting frame building can experience failure through the development of a storey collapse, a beam mechanism, or an intermediate mixed mode, as illustrated in Figure 2 [32]. If the building has a weak column-strong beam configuration, it is susceptible to failure due to the storey mechanism (as shown in Figure 2a) when subjected to seismic loads. Conversely, the adoption of a strong column-weak beam formulation may increase the probability of the beam mechanism (as shown in Figure 2c), which is the desirable mode of failure for typical frame architectures [33]. Building regulations in this context require a minimum ratio of the sum of column moment capabilities to the sum of moment capacities (1.2 in ACI 318 [34]) in order to produce a strong column-weak beam. However, a prior study demonstrated that the beam mechanism can only be attained when the ratio of column-to-beam moment capacities is approximately four, which is contingent upon the number of stories in the specific building [35]. Furthermore, the existing concept of the strong column-weak beam, which is based solely on joint equilibrium, may not effectively avoid the narrative mechanism as it fails to consider the overall system or storey [33]. Thus, the occurrence of the storey mechanism is expected to happen under seismic load. This storey mechanism is taken into account in the second-level seismic evaluation conducted by JBDPA [16] and CNCRP [19] as the primary failure mechanism.



(a) (b) (c)

Fig. 2. Failure mechanisms of frame structure (a) storey mechanism; (b) intermediate mechanism; and (c) beam mechanism [32].

2.2.1. Strength index, c

The strength index, C quantifies the maximum capacity of vertical components, such as columns and walls, to bear lateral loads in each storey. The calculation is performed on the assumption that beams possess significant strength, whereas vertical elements provide resistance against lateral forces. The evaluation of failure modes in vertical members involves a comparative analysis of their ultimate shear and flexural strengths. The maximum lateral load capacity of any structural element is determined by comparing its shear strength and flexural strength and selecting the minimum value as critical, which can be calculated using Equation 4. The ultimate lateral load-carrying capacity of columns depends on the flexural strength (Q_{MU}) and the shear strength (Q_{SU}). For columns, the flexural and shear strengths are determined using Equations 5-7.

The flexural strength (Q_{MU}) of a column is computed considering hinges at the top and bottom of the column. The shear strength (Q_{SU}) of a column is determined based on Hirosawa's study [36].

$$C=\frac{Q_u}{\Sigma W} \tag{4}$$

$$Q_{SU(BR)}=K_r\cdot\left\{\frac{0.053\rho_t^{\,0.23}\left(18+\sigma_B\right)}{\frac{M}{(Q\cdot d)}+0.12}+0.85\sqrt{\rho_w\cdot\sigma_{wy}}+0.1\cdot\sigma_0\right\}\cdot b\cdot j \tag{5}$$

$$Q_{SU(AR)}=\left\{\frac{0.053\cdot\rho_{t2}^{\,0.23}\cdot\left(F_{avg}+18\right)}{\frac{M}{Q\cdot d_2}+0.12}+0.85\sqrt{\rho_w\cdot\sigma_{wy}+\rho_{w2}\cdot\sigma_{wy2}}+0.1\cdot\frac{N}{b_2D_2}\right\}\cdot 0.8b_2D_2 \tag{6}$$

$$Q_{MU}=\frac{2M_U}{h} \tag{7}$$

where, Q_U = ultimate lateral load carrying capacity of the members from the concerned storey. $\sum W$ = weight of the building including live load for seismic calculation, $Q_{SU(BR)}$ = ultimate shear strength of the column before retrofit, $Q_{SU(AR)}$ = ultimate shear strength of the column after retrofit. k_r = reduction factor for low strength concrete, P_t = tensile reinforcement ratio, a_w = cross-sectional area of shear reinforcement. S = shear reinforcement spacing, P_w = shear reinforcement ratio. $_s\,\sigma_{wy}$ = Yield strength of shear reinforcement bars (N/mm^2), σ_0 = axial stress in column (N/mm^2), d = effective depth of column, h_0 = clear height off column. j = distance between centroids of tension and compression forces, the default value is $0.8D$. P_{t2} = tensile reinforcement ratio calculated by using the increased cross-section of the jacketed column, P_w = shear reinforcement ratio of the existing column calculated by the increased cross-section of the jacketed column. P_{w2} = shear reinforcement ratio of jacketing column calculated by the increased cross-section of the jacketed column.

2.2.2. Ductility index, F

The ductility index *(F)* is a measure that represents the ability of vertical structural elements, such as columns and walls, to undergo inelastic deformation beyond the point of first yielding. It is the measure of deformation at which the lateral resistance begins to deteriorate [37]. This metric assesses their capacity to endure plastic deformations while still retaining their strength. The calculation of the ductility index varies depending on the anticipated failure mechanism of the structural part. The index is correlated with the final deformation and drift angles that are linked to various failure mechanisms, such as shear or flexure. In order to restrict the maximum drift angles and plastic drift capabilities, specific measures are outlined for high-axial load columns often seen in Bangladesh. The ductility index, *F,* is calculated using the CNCRP seismic assessment manual [19]. The basic calculation of the ductility index, *F,* for columns is summarized in Table 1.

Table 1. Calculation of ductility index, F for columns [19].

Column type	F index	Condition	Notation
Shear column	$F = 1 + 0.27 \cdot \dfrac{R_{SU} - R_{250}}{R_y - R_{250}}$	-	R_y = Yield deformation in terms of inter-storey drift angle = 1/150. R_{250} = Standard inter-storey drift angle = 1/250. R_{su} = Inter-storey drift angle at the ultimate deformation capacity in shear failure R_{mu} = Inter-storey drift angle at the ultimate deformation capacity in flexural failure
Flexural column	$F = 1 + 0.27 \cdot \dfrac{R_{mu} - R_{250}}{R_y - R_{250}}$	$R_{mu} < R_y$	
	$F = \dfrac{\sqrt{\dfrac{2R_{mu}}{R_y} - 1}}{0.75 \cdot \left(1 + \dfrac{0.5R_{mu}}{R_y}\right)} \le 3.2$	$R_{mu} \ge R_y$	

2.3 Seismic demand index, is0

This is the measure of the loads a building may undergo when subjected to seismic events. This depends on some of the key structural and locational parameters for a specific building [19]. Although the method of seismic assessment is adopted from JBDPA [16], the seismic demand calculation is quite different from the one prescribed in JBDPA [16].

The seismic demand computation in the JBDPA [16] recommendations originated from the damaged data available from the previous earthquakes in Japan. According to the study of Okada and Nakano [38], the seismic capacity index (*Is*) quantifies a structure's seismic resistance by considering strength, ductility, and reduction variables. Figure 3 [38] shows the log-normal frequency distribution of 1615 Japanese reinforced concrete buildings' capacity (I_s) with damage states, and research [38] found that a minimum value of I_s > 0.6 reduces the risk of a structure incurring damage with a 95% confidence level when shaken with an acceleration of 0.23g. This I_s value (= 0.6) was adopted in JBDPA [16], as a margin, to assess the vulnerability of Japanese buildings.

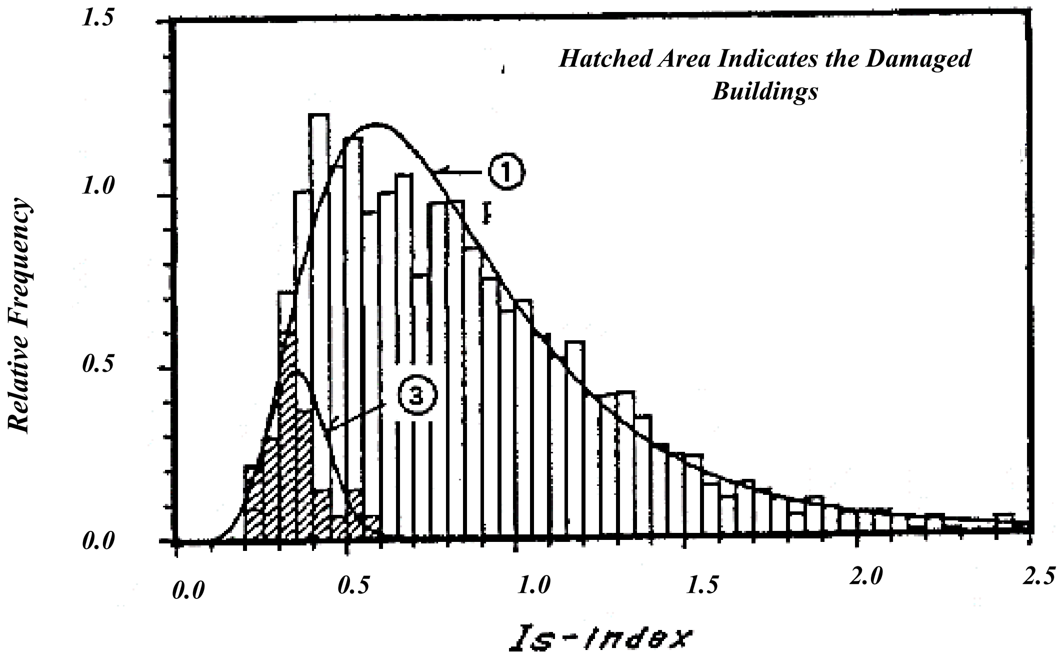


Fig. 3. Distribution of IS for existing buildings [38].

In the case of Bangladesh, there is not enough damage data from previous earthquakes that can be used to suggest a probable index for seismic capacity. Therefore, the CNCRP seismic evaluation manual [19] suggests an equation that is based on various structural and demographic parameters to calculate the seismic demand of a building. These parameters can be obtained from BNBC [39]. The seismic demand index is calculated using Equation 8.

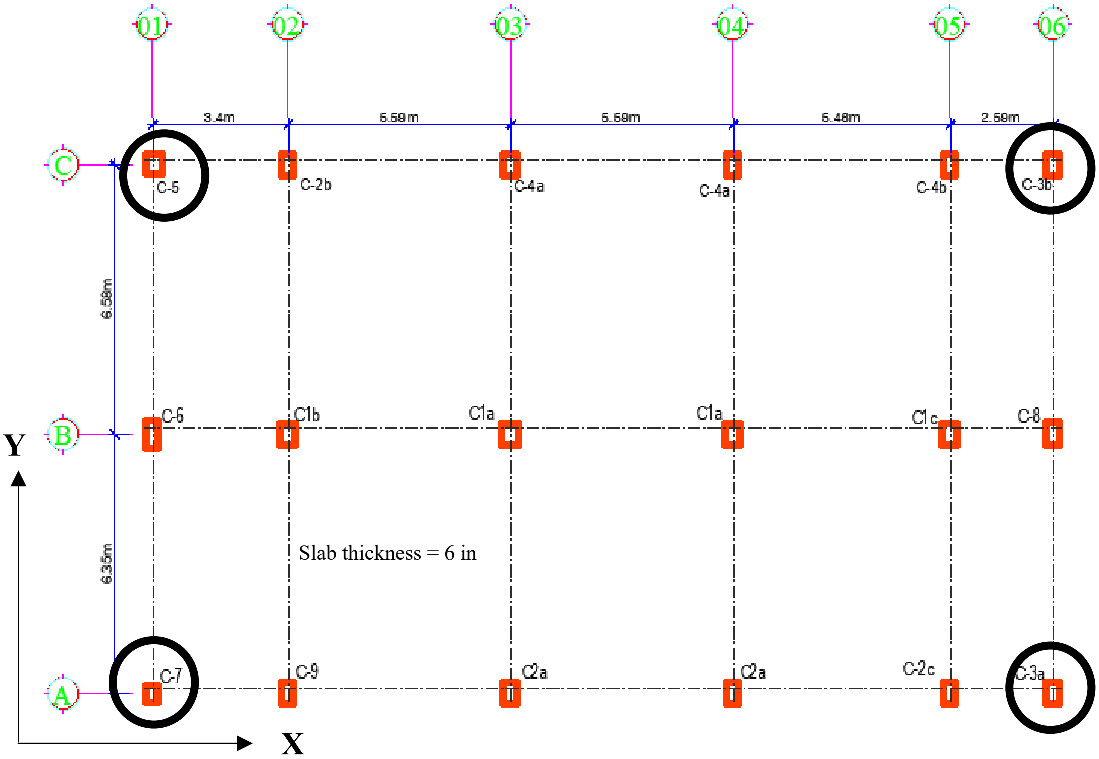
$$I_{S0}=0.8\cdot\frac{2}{3}\cdot Z\cdot I\cdot C_S \tag{8}$$

where, Z = Zone coefficient, I = Importance factor, C_S = Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class).

3. Characteristics of target building

3.1. General information about the building

An assessment was conducted on an existing six-storey RC building that was constructed in 2008. The building's structural system consists of a moment-resisting frame system, with deep foundations. The building was designed for both gravity and lateral loads using the Bangladesh National Building Code (2006) [40]. This building was selected randomly; therefore, it cannot be considered a representation of overall construction practices in Bangladesh. Each of the stories of the building was evaluated in detail, where the floor weight varied between 3838 kN and 4682 kN. The column outline of the concerned building is shown in Figure 4.



Column Size
C1 (a), (b), (c): 20″x21″), C2 (a), (b), (c): 18″x20″)
C3 (a), (b): 12″x20″), C4 (a), (b): 18″x22″)
C5: 10″x21″), C6: 15″x20″), C7: 15″x21″),
C8: 15″x21″), C9: 15″x20″)

Fig. 4. Column outline of the existing RC building (The circled columns have been retrofitted which will be discussed in the section 5.1).

3.2. Material properties

The building authority performed the concrete and re-bar tests by collecting a sufficient number of concrete core and re-bar samples from the structure in question. Then concrete equivalent strength was computed using the suggestions of ACI 214 [41] and ACI 562 [42] guidelines. It was found that the compressive strength of the concrete used in the construction is measured at 28.34 MPa, while the reinforcing bars exhibit a yield strength of 413.68 MPa.

4. Seismic evaluation of existing building using cncrp

The determination of the probable seismic risk of that particular building involved the assessment of both seismic capacity and seismic demand, which are discussed in the following sub-sections.

4.1. Seismic capacity evaluation

For the assessment of the seismic capacity of the building, each of the vertical load-carrying members (e.g., columns) was assessed. As the CNCRP [19] manual assumes the occurrence of story collapse (i.e., beams are strong enough and the columns fail during seismic hazards), the beams needed no assessment. Firstly, the columns were divided according to their tributary area, and then the relevant indices (i.e., strength and ductility index) were calculated with the help of the equations suggested by CNCRP [19]. The computation of the lateral strength and ductility indices of the vertical members was conducted using an Excel spreadsheet. Table 2 shows the calculation of the strength and ductility indices.

Table 2. Strength and ductility of different columns for storey 1.

Column ID	ho/D	Q_{mu} (KN)	Q_{su} (kN)	Selected Q(kN)	Failure Type	C	F
C1a	6.1	343	1166	343	Flexural	0.029	2.016
C1b	6.1	328	1134	328	Flexural	0.014	2.016
C1c	6.1	298	1080	298	Flexural	0.012	2.016
C2a	7.1	219	910	219	Flexural	0.018	2.016
C2b	7.1	213	898	213	Flexural	0.009	2.016
C2c	7.1	189	856	189	Flexural	0.008	2.016
C3a	10.7	75	399	75	Flexural	0.003	2.016
C3b	10.7	82	416	82	Flexural	0.003	2.016
C4a	7.1	231	876	231	Flexural	0.019	2.016
C4b	7.1	208	827	208	Flexural	0.009	2.016
C5	6.7	142	616	142	Flexural	0.006	2.016
C6	8.5	164	765	164	Flexural	0.007	2.016
C7	8.5	117	529	117	Flexural	0.005	2.016
C8	8.5	120	535	120	Flexural	0.005	2.016
C9	8.5	178	799	178	Flexural	0.007	2.02

The seismic capacity of the building can be derived using the strength index (C) and ductility index (F) curves of the building. The floor-wise strength-ductility relationships for each of the major orthogonal directions of the building are shown in Table 4. The figure shows that each of the stories deformed substantially (i.e., F indices are within a range of 1.75 to 2, which indicate lateral drift of 1% to 1.25%) before witnessing collapse, which indicates that the ductility of the stories is adequate. If the seismic assessment reveals that any of the building's stories fail to fulfill the minimal seismic safety criteria, it becomes essential to do structural strengthening in order to enhance the building's ability to withstand current seismic demands.

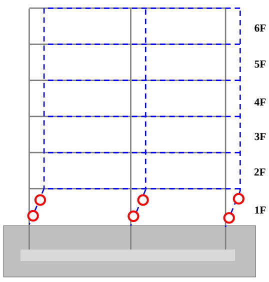
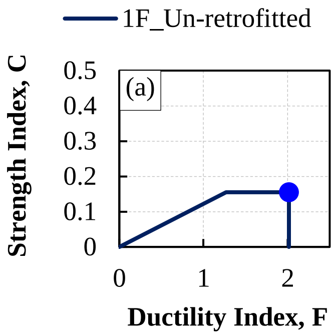
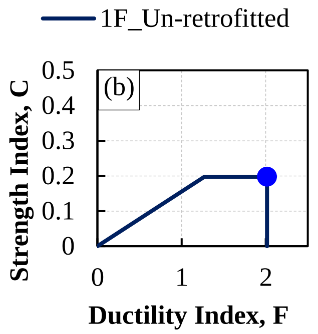
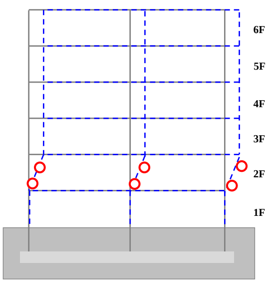
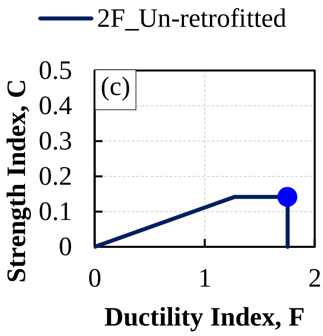
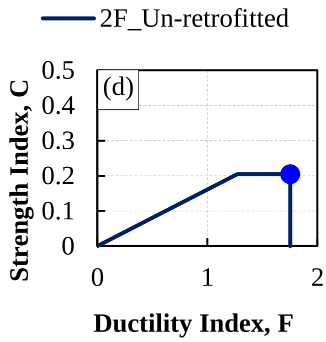
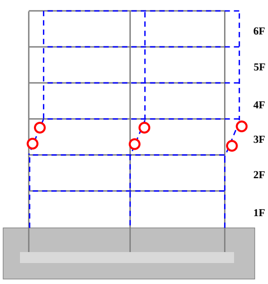
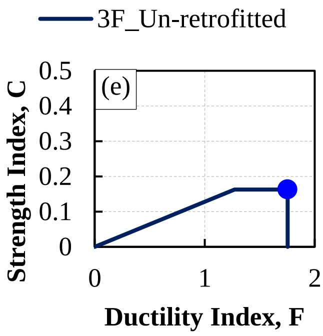
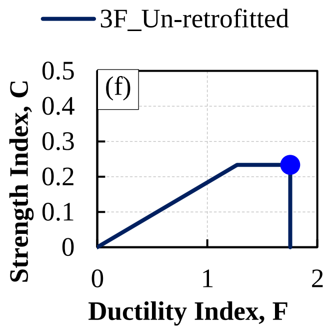
4.2. Seismic demand evaluation

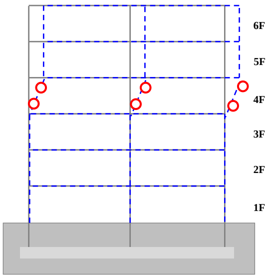
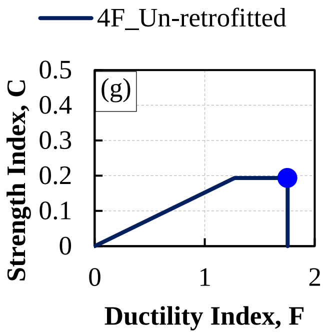
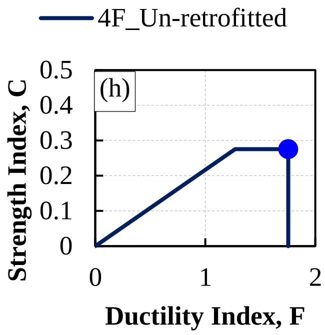
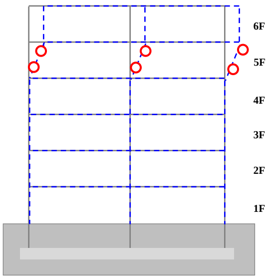
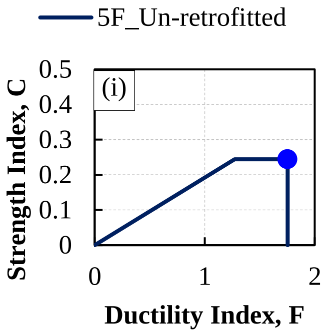
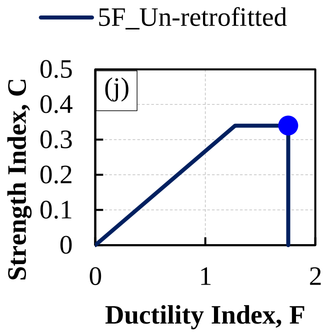
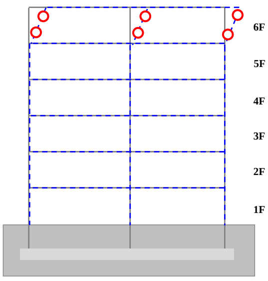
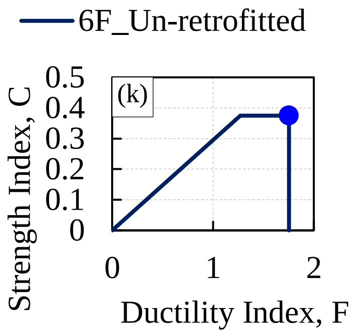
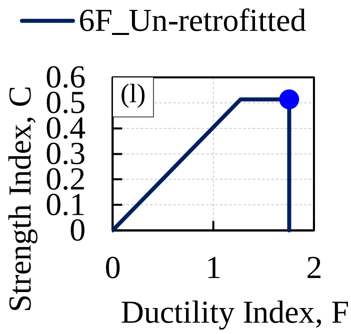
The demand index for the building of interest was calculated as suggested by the seismic evaluation manual CNCRP [19], using Equation 8. The relevant values were adopted from BNBC [39]. The value of the seismic demand index, I_{S0}, was found to be 0.27, as presented in Table 3.

Table 3. Summary of seismic demand evaluation

Normalized acceleration response spectrum, Cs	2.55
Importance factor, I	1
Zone co-efficient, Z	0.2
Seismic Demand Index, Iso	0.27

Table 4. Strength and ductility relationship of (a) 1F (long/X-direction), (b) 1F (trans./Y-direction), (c) 2F (long/X-direction), (d) 2F (trans./Y-direction), (e) 3F (long/X-direction), (f) 3F (trans./Y-direction), (g) 4F (long/X-direction), (h) 4F (trans./Y-direction), (i) 5F (long/X-direction), (j) 5F (trans./Y-direction), (k) 6F (long/X-direction), and (l) 6F (trans./Y-direction).

Assumed collapse mode	C-F relationship (Long/X-direction)	C-F relationship (Transverse/Y-direction)
		
		
		

Assumed collapse mode	C-F relationship (Long/X-direction)	C-F relationship (Transverse/Y-direction)
		
		
		

4.3. Vulnerability assessment of building

The seismic vulnerability of the building can be assessed by comparing the seismic capacity index, I_S, and the seismic demand index, I_{S0}, for each of the major orthogonal axes of the building. Table 5 shows the comparison between seismic capacity and seismic demand for each floor of the concerned building. From Table 5, it is clear that each of the floors in the transverse direction shows adequate strength compared to the current seismic demand calculated in accordance with the suggestions of CNCRP [19]. While considering the long direction, it is seen that the seismic performance of the 2nd, 3rd, and 4th floors of the building remains uncertain.

From the seismic evaluation, it is pretty clear that some floors of the building show uncertainty in terms of seismic vulnerability. To mitigate this, the specific floors needed retrofitting, which was done according to the suggestions in the CNCRP [31] retrofit manual.

Table 5. Comparison between seismic capacity and demand (before retrofit).

Storey ID	Long direction			Transverse direction		
	Seismic Demand, *Iso*	Seismic Capacity, *Is*	Remarks	Seismic Demand, *Iso*	Seismic Capacity, *Is*	Remarks
1	0.27	0.31	Safe	0.27	0.40	Safe
2	0.27	0.22	Uncertain	0.27	0.31	Safe
3	0.27	0.22	Uncertain	0.27	0.32	Safe
4	0.27	0.24	Uncertain	0.27	0.34	Safe
5	0.27	0.27	Safe	0.27	0.38	Safe
6	0.27	0.38	Safe	0.27	0.53	Safe

5. Retrofitting of existing building

Figure 5 depicts the conceptual strategic framework for the retrofitting of existing RC structures. In the provided diagram, the vertical axis is used to depict the storey shear coefficient which is a dimensionless entity. This coefficient is specifically defined as the ratio of the horizontal strength at ground level to the weight of the structure. The horizontal axis represents the ductility factor, which is shown by the storey deflection angle. Similar to the previous one, it is also a dimensionless entity. This angle is determined by dividing the storey deflection by the storey height [31].

Figure 5 displays curve 'A,' which is a representative example of an existing reinforced concrete (RC) structure that exhibits inadequate strength and ductility. In order to rectify this deficiency, three retrofit solutions have been suggested: a) the strength-oriented retrofit method (curve S), b) the ductility-oriented retrofit method (curve D), and c) both the strength- and ductility-oriented retrofit methods (curve B). The hyperbolic curve indicates the seismic demand of a particular building floor. The region located in the top right section of the hyperbolic curve seen in the picture is often denoted as the 'Seismic Target Zone' or as the 'Target Zone.' The seismic target zone is an illustration of the preferred range of performance for a retrofitted structure, as shown on a graph plotting the strength coefficient against the ductility factor. The concept is characterized by a specified minimum strength threshold and a prescribed minimum capacity for ductility. The objective of seismic retrofitting is to increase the structural integrity of a building by implementing appropriate strengthening measures and improving its ability to deform under seismic forces. This is done in order to ensure that the structure's performance aligns with the desired criteria outlined in seismic codes. When the performance curve of a structure crosses or exceeds the designated target zone, it signifies that the building satisfies seismic safety criteria and is considered acceptable.

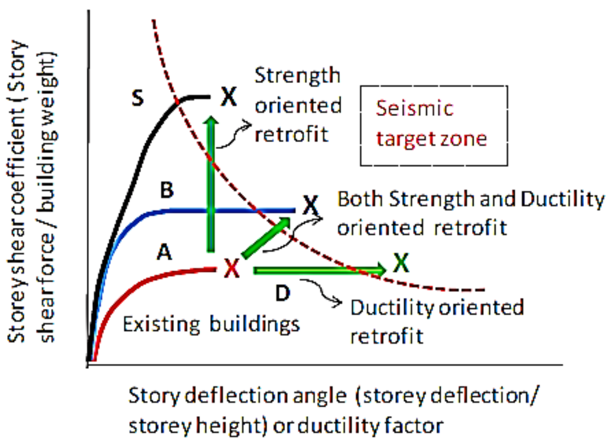


Fig. 5. Load and Deflection Curves and Concept of Retrofit [31].

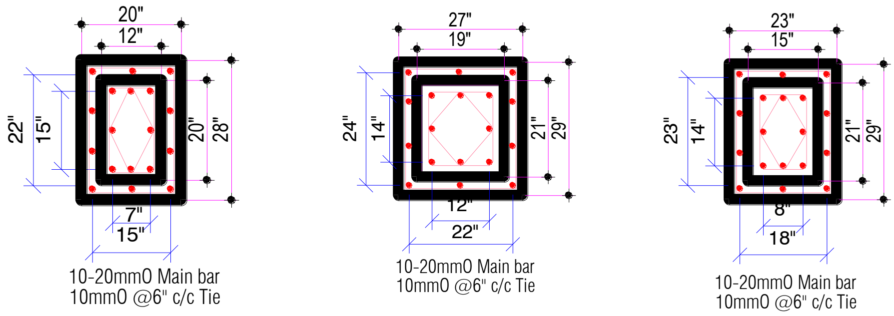
5.1. Design of RC jacket of columns

As discussed earlier, the structural members show substantial deformation before strength degradation. Column jacketing can be a viable method for seismic retrofitting in cases where a building possesses sufficient ductility but lacks the necessary strength. This technique involves casting a RC layer around the existing RC column and reinforcing the jacket with longitudinal and transverse reinforcements [43] to enhance their confinement and resistance to buckling. The efficiency of the jacketing comes from the enlarged cross section, which in turn increases load-carrying capacity and the confinement pressure induced by the jacketing in the core column. However, attention should be paid to ensure proper bonding between new and old concrete surfaces during construction. The effectiveness of RC jacketing in terms of flexural capacity can be reduced by up to 10% if the old concrete surface is not roughened [44]. The RC jacketing aligns with the principles of retrofitting that prioritise strengthening, as detailed in CNCRP [31]. The RC jacketing has been selected herein since this jacketing method was utilized by the Public Works Department of Bangladesh for some existing RC buildings in Bangladesh. In addition, the design procedure for RC jacketing is available in the CNCRP Retrofitting Guidelines prescribed by the Public Works Department of Bangladesh.

To retrofit this structure, four corner columns were selected, as marked with circles in Figure 4, up to 5F level. The selection was done in such a way that the selection remains symmetrical and the centre of rigidity does not shift. After a thorough assessment of the stories in both directions, the additional strength requirement of the stories was calculated individually according to the guidelines of CNCRP [31]. The strength requirements are shown in Table 6. The significance of designing the chosen columns on the floors that need cross-section and reinforcing in line with their observed strength deficits can be seen by the requirement for enhanced structural strength.

Table 6. Storey-wise strength requirement for the columns.

Storey ID	∑Wi (KN)	Storey shear modification factor (n+i)/(n+1)	Design Shear coefficient	Design Shear Strength Q (KN)	Original Strength X-Dir Q(KN)	Original Strength Y-Dir Q(KN)	Required addition Strength Long direction	Required addition Strength Transverse direction
6	4682	1.71	0.2639	1235.5	1755	2404	0.0	0.0
5	8520	1.57	0.2419	2060.9	2081	2896	0.0	0.0
4	12358	1.43	0.2199	2717.5	2392	3404	325.5	0.0
3	16196	1.29	0.1979	3205.3	2636	3781	569.3	0.0
2	20034	1.14	0.1759	3524.3	2836	4092	688.3	0.0
1	23872	1.00	0.1340	3197.8	3700	4713	0.0	0.0



(a) (b) (c)

Fig. 6. Column reinforcement of 1st storey details after retrofitting of column (a) C3a & C3b, (b) C5 and (c) C7.

The retrofitting process included the use of column jacketing technology as a method to enhance the structural seismic capacity. Additional reinforcement was included in accordance with the stated strength parameters, and the cross-sectional area was also increased. The reinforcement detailing of retrofitted column sections from the 1st storey is shown in Figure 6.

5.2. Seismic evaluation of the building after retrofitting

The floors, which are considered uncertain after the seismic assessment, are strengthened with the application of column jacketing. The building was thoroughly analyzed again using CNCRP [19] after retrofitting. Figure 7 shows the floor-wise strength and ductility relationship before and after retrofitting. From Figure 7, it is clear that after column jacketing, each of the floors has shown greater strength at the same ductility level compared to the building before the retrofit. It is worthy to mention that the considered columns (both before and after retrofitting) are expected to have a flexural failure; that's why the ductility level remained the same.

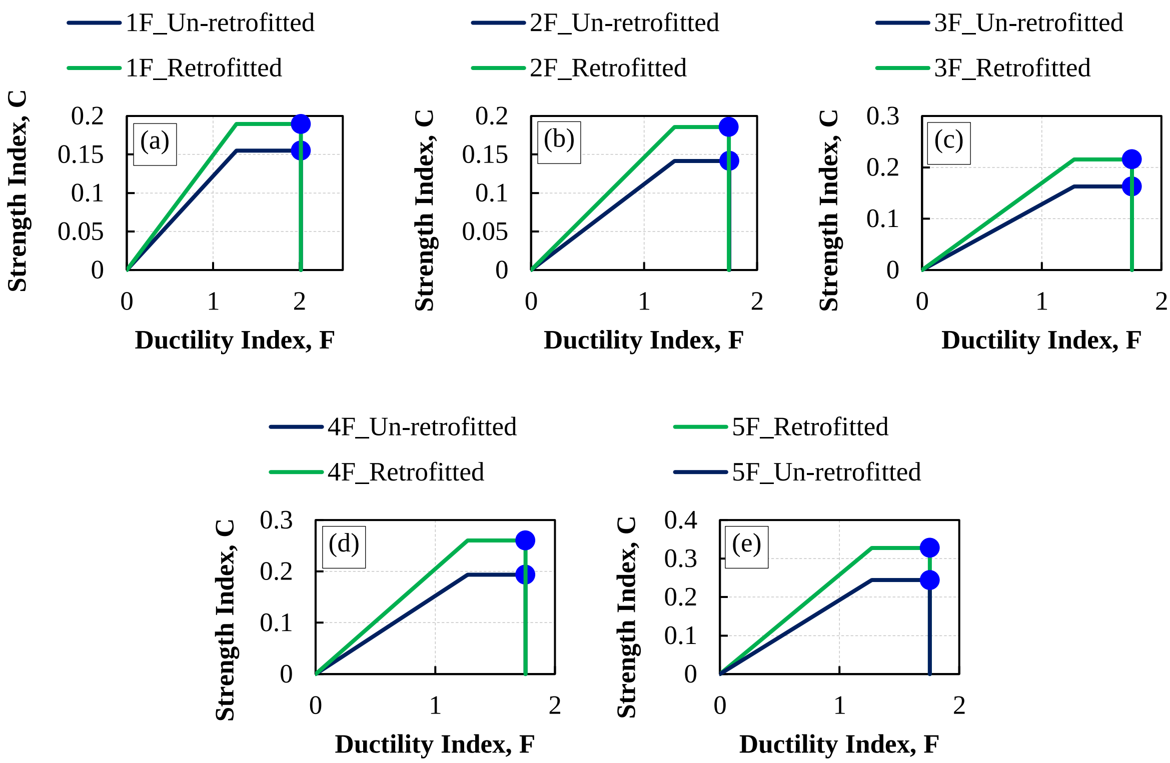


Fig. 7. Strength-ductility relationship before and after retrofit of (a) 1F, (b) 2F, (c) 3F, (d) 4F, and (e) 5F in long/X-direction.

5.2. Assessment of the building after retrofitting

After retrofitting, the seismic capacity index for each of the floors was calculated again using the strength-ductility relationships shown in Figure 8. After the determination of the modified seismic capacity indices (as the building is retrofitted), the seismic assessment was done by comparing the modified seismic capacity with the seismic demand that was calculated earlier. Figure 8 shows the storey-wise seismic assessment of the RC building using CNCRP [19].

From Figure 8, it can be concluded that the seismic capacity index value for each of the floors of the building increased after retrofitting. Once the values exceed the seismic demand index, the building may be deemed non-vulnerable in relation to the existing seismic demands.

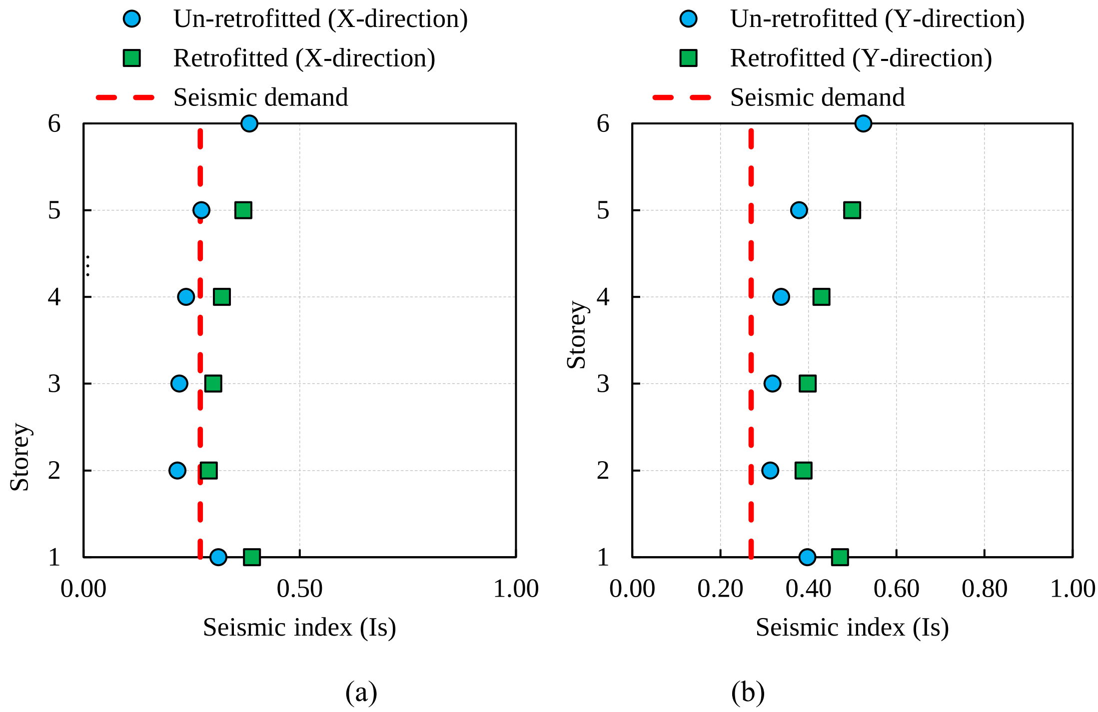


Fig. 8. Storey-wise seismic assessment of the existing RC building before and after retrofitting (a) Long/X-direction and (b) Transverse/Y-direction.

6. Conclusion

This study aimed to assess an existing building located in Bangladesh according to the CNCRP seismic evaluation manual [19] and retrofit it using the CNCRP retrofit manual [31], if found vulnerable. This evaluation method has a unique feature of vulnerability assessment based on local seismic demand computation, while similar other seismic evaluation methods [16] utilize previous earthquake damage data for seismic demand computation. To be in line with the objective, an existing RC building was assessed in detail using the CNCRP seismic assessment manual [19] and retrofitted by adopting the CNCRP retrofit manual [31]. The following outcomes were found from this study:

- The considered building possesses a substantial deformation capacity, i.e., a lateral drift capacity of 1% to 1.25%.
- The comparison of overall seismic capacity (I_s) and demand (I_{S0}) indices demonstrates that some of the floors show uncertainty in seismic performance, i.e., the floors would not be capable of resisting the local seismic demand. Thus, retrofitting the building is required.
- The adopted retrofitting design scheme, i.e., RC jacketing of corner columns, enhanced the lateral strength of the floors without altering the deformation capacity.
- Further, the re-assessment of the retrofitted building confirms that any floor is not vulnerable when seismic capacity (I_S) and demand (I_{SO}) indices are compared considering the local seismicity.

The performances of the studied RC building, both before and after retrofitting, were evaluated considering the storey collapse mechanism (i.e., column failure). However, the performance of the building should also be evaluated by considering the total collapse mechanism (i.e., combined beam and column failure) through non-linear analysis before implementing it in the real field, which would be a scope for future research. It is to be noted that since we considered only one RC building, we may not consider this a representation of overall construction practices in Bangladesh.

Funding

The authors state that they did not receive any financial support, grants, or other assistance during the development of this manuscript.

Conflicts of interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Authors contribution statement

Debasish Sen: Conceptualization; Methodology; Supervision; Visualization; Writing – review & editing.

Md. Kamruddoja Shihab: Data curation; Formal analysis; Investigation; Methodology; Resources; Software; Validation; Visualization; Writing – review & editing.

Md. Farhad Momin: Investigation; Visualization; Roles/Writing – original draft; Writing – review & editing.

References

[1] Anwar GA, Dong Y. Seismic resilience of retrofitted RC buildings. Earthq Eng Eng Vib 2020;19:561–71. https://doi.org/10.1007/s11803-020-0580-z.

[2] Ozturk M, Arslan MH, Korkmaz HH. Effect on RC buildings of 6 February 2023 Turkey earthquake doublets and new doctrines for seismic design. Eng Fail Anal 2023;153:107521. https://doi.org/10.1016/j.engfailanal.2023.107521.

[3] Çetin KO. Türkiye Earthquake Reconnaissance and Research Alliance RECONNAISSANCE REPORT 2023:1–484. https://doi.org/10.13140/RG.2.2.15569.61283/1.

[4] Wang T, Chen J, Zhou Y, Wang X, Lin X, Wang X, et al. Preliminary investigation of building damage in Hatay under February 6, 2023 Turkey earthquakes. Earthq Eng Eng Vib 2023;22:853–66. https://doi.org/10.1007/s11803-023-2201-0.

[5] Hossain ASMF, Jahan N, Ansary MA. A STUDY ON RECENT EARTHQUAKES IN AND AROUND BANGLADESH. Malaysian J Civ Eng 2021;33. https://doi.org/10.11113/mjce.v33.16339.

[6] Ahmed MZ, Islam K, Roy KS, Arafat MS, Al-Hussaini TM. Seismic vulnerability assessment of RCF buildings in old town of Dhaka city. Proc. 3rd Int. Earthq. Symp. Dhaka, Bangladesh, 2010, p. 5–6.

[7] Ali MH, Choudhury JR. Seismic zoning of Bangladesh. Int. Semin. Recent Dev. Earthq. Disaster Mitigation, Inst. Eng. Dhaka, 1994.

[8] Ansari A, Seshagiri Rao K, Jain AK. Seismic Microzonation of the Himalayan Region Considering Site Characterization: Application toward Seismic Risk Assessment for Sustainable Tunneling Projects. Nat Hazards Rev 2024;25. https://doi.org/10.1061/NHREFO.NHENG-1815.

[9] Ansari A, Rao KS, Jain AK. An integrated approach to model seismic loss for the Himalayan infrastructure projects: Decision-making and functionality concept for disaster mitigation. Bull Eng Geol Environ 2023;82:393. https://doi.org/10.1007/s10064-023-03422-x.

[10] Ghasemi Jouneghani H, Nouri Y, Mortazavi M, Haghollahi A, Memarzadeh P. Seismic Performance Factors of Elliptic-Braced Frames with Rotational Friction Dampers through IDA. Pract Period Struct Des Constr 2024;29:1–24. https://doi.org/10.1061/PPSCFX.SCENG-1540.

[11] Ghasemi Jouneghani H, Nouri Y, Memarzadeh P, Haghollahi A, Hemati E. Seismic performance and failure mechanisms evaluation of multi-story elliptic and mega-elliptic bracing frames: Experimental and numerical investigation. Structures 2024;70:107658. https://doi.org/10.1016/j.istruc.2024.107658.

[12] Nouri Y, Jouneghani HG, Haghollahi A, Hemati E, Hemati SA, Mortazavi M. Experimental and numerical investigation of a steel yielding arc and ring damper. Structures 2024;68:107140. https://doi.org/10.1016/j.istruc.2024.107140.

[13] Seismic Evaluation of Existing Buildings. Reston, VA: American Society of Civil Engineers; 2003. https://doi.org/10.1061/9780784406700.

[14] Seismic Evaluation and Rehabilitation of Existing Buildings. Tall Build. Des., Taylor & Francis Group, 6000 Broken Sound Parkway NW, Suite 300, Boca Raton, FL 33487-2742: CRC Press; 2016, p. 493–620. https://doi.org/10.1201/9781315374468-9.

[15] Agency FEM. Rapid visual screening of buildings for potential seismic hazards: Supporting documentation. Government Printing Office; 2015.

[16] Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings. The Japan Building Disaster Prevention Association (JBDPA). The Japan Building Disaster Prevention Association n.d.

[17] Islam MS, Alwashali H, Sen D, Maeda M. A proposal of Visual Rating method to set the priority of detailed evaluation for masonry infilled RC building. Bull Earthq Eng 2020;18:1613–34. https://doi.org/10.1007/s10518-019-00763-5.

[18] Kehoe B. Standardizing seismic evaluation of existing buildings. 13th World Conf. Earthquaqe Eng., 2004.

[19] CNCRP Manual for Seismic Evaluation of Existing Reinforced Concrete Buildings: Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings (CNCRP). Public Works Department of Bangladesh, 2015.

[20] Aguayo R, Carvallo J, Vielma JC. Evaluating Seismic Performance in Reinforced Concrete Buildings with Complex Shear Walls: A Focus on a Residential Case in Chile. Buildings 2024;14:761. https://doi.org/10.3390/buildings14030761.

[21] Ansari A, Rao KS, Jain AK, Ansari A. Deep learning model for predicting tunnel damages and track serviceability under seismic environment. Model Earth Syst Environ 2023;9:1349–68. https://doi.org/10.1007/s40808-022-01556-7.

[22] Kazemi F, Asgarkhani N, Jankowski R. Machine learning-based seismic response and performance assessment of reinforced concrete buildings. Arch Civ Mech Eng 2023;23:94. https://doi.org/10.1007/s43452-023-00631-9.

[23] Batalha N, Rodrigues H, Sousa R, Varum H. Seismic assessment of existing precast RC industrial buildings in Portugal. Structures 2022;41:777–86. https://doi.org/10.1016/j.istruc.2022.05.040.

[24] Oyguc RA. A case study on an innovative seismic performance evaluation procedure for irregular RC buildings. Front Built Environ 2022;8. https://doi.org/10.3389/fbuil.2022.1058983.

[25] Shendkar MR, Kontoni D-PN, Kumar RP, Farghaly AA, Mandal S, Maiti PR. A refined procedure for seismic evaluation and retrofitting of reinforced concrete buildings. Curr Sci 2022;123:1020. https://doi.org/10.18520/cs/v123/i8/1020-1030.

[26] Fakharian P, Nouri Y, Ghanizadeh AR, Safi Jahanshahi F, Naderpour H, Kheyroddin A. Bond strength prediction of externally bonded reinforcement on groove method (EBROG) using MARS-POA. Compos Struct 2024;349–350:118532. https://doi.org/10.1016/j.compstruct.2024.118532.

[27] Chen L, Nouri Y, Allahyarsharahi N, Naderpour H, Rezazadeh Eidgahee D, Fakharian P. Optimizing compressive strength prediction in eco-friendly recycled concrete via artificial intelligence models. Multiscale Multidiscip Model Exp Des 2025;8:24. https://doi.org/10.1007/s41939-024-00641-x.

[28] Fakharian P, Naderpour H, Sharbatdar MK, Ghasemi SH. Advancements in structural target reliability: A comprehensive examination of theories and practical applications (2013-2024). Structures 2024;66:106930. https://doi.org/10.1016/j.istruc.2024.106930.

[29] Sulthan F, Gumilang S AA, Rusli M, Seki M. Seismic evaluation of existing building structure using United States (ASCE 41-17) and Japanese (JBDPA) standard: Case study office building in Indonesia. E3S Web Conf 2023;429:05001. https://doi.org/10.1051/e3sconf/202342905001.

[30] Mohammad Shams Wahid C, Alam MJ, Islam MR. SEISMIC ASSESSMENT OF EXISTING BUILDING IN BANGLADESH USING JAPANESE INDEX METHOD. Malaysian J Civ Eng 2020;32. https://doi.org/10.11113/mjce.v32n1.616.

[31] CNCRP Manual for Seismic Retrofit Design of Existing Reinforced Concrete Buildings: Project for Capacity Development on Natural Disaster Resistant Techniques of Construction and Retrofitting for Public Buildings (CNCRP). Public Works Department of Banglad 2015.

[32] Moehle JP, Hooper JD, Lubke CD. Seismic design of reinforced concrete special moment frames : Gaithersburg, MD: 2008. https://doi.org/10.6028/NIST.GCR.08-917-1.

[33] YE L, QU Z. FAILURE MECHANISM AND ITS CONTROL OF BUILDING STRUCTURES UNDER EARTHQUAKES BASED ON STRUCTURAL SYSTEM CONCEPT. J Earthq Tsunami 2009;03:249–59. https://doi.org/10.1142/S1793431109000627.

[34] ACI Committee 318. Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05). American Concrete Institute; 2005.

[35] Reduction of Column Yielding During Earthquakes for Reinforced Concrete Frames. ACI Struct J 2003;100. https://doi.org/10.14359/12798.

[36] Hirosawa M. Past experimental results on reinforced concrete shear walls and analysis on them. Kenchiku Kenkyu Shiryo 1975;6:33–4.

[37] Kabeyasawa T. Seismic evaluation and economical strengthening of reinforced concrete buildings 2005.

[38] Okada T, Nakano Y. Reliability analysis on seismic capacity of existing reinforced concrete buildings in Japan. Proc. Ninth World Conf. Earthq. Eng. Sci. Counc. Japan, Tokyo, Japan, vol. 7, 1988, p. 333–8.

[39] Bangladesh National Building Code. Housing and Building Research Institute, Ministry of Housing and Public Works, Government of the People's Republic of Bangladesh 2020.

[40] Bangladesh National Building Code. Housing and Building Research Institute, Ministry of Housing and Public Works, Government of the People's Republic of Bangladesh 2006.

[41] 214 ACIC. Guide to evaluation of strength test results of concrete, American Concrete Institute; 2011.

[42] Standard AACI. Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures (ACI 562-19) and Commentary 2019.

[43] Habib A, Yildirim U, Eren O. Column repair and strengthening using RC jacketing: a brief state-of-the-art review. Innov Infrastruct Solut 2020;5:75. https://doi.org/10.1007/s41062-020-00329-4.

[44] Minafò G. A practical approach for the strength evaluation of RC columns reinforced with RC jackets. Eng Struct 2015;85:162–9. https://doi.org/10.1016/j.engstruct.2014.12.025.