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Research Article

Seismic Evaluation and Retrofit of Reinforced Concrete Buildings with Masonry Infills Based on Material Strain Limit Approach

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The seismic evaluation and retrofit of reinforced concrete (RC) structures considering masonry infills is the correct methodology because the infill walls are an essential part of RC structures and increase the stiffness and strength of structures in seismically active areas. A three-dimensional four-storey building with masonry infills has been analyzed with nonlinear static adaptive pushover analysis by using the SeismoStruct software. Two models have been considered in this study: the first model is a full RC-infilled frame and the second model is an open ground storey RC-infilled frame. The infill walls have been modeled as a double strut nonlinear cyclic model. In this study, the "material strain limit approach" is first time used for the seismic evaluation of RC buildings with masonry infills. This method is based on the threshold strain limit of concrete and steel to identify the actual damage scenarios of the structural members of RC structures. The two models of the four-storey RC building have been retrofitted with local and global strengthening techniques (RC-jacketing method and incorporation of infills) as per the requirements of the structure to evaluate their effect on the response reduction factor (R) because the R-factor is an important design tool that shows the level of inelasticity in a structure. A significant increase in the response reduction factor (R) and structural plan density (SPD) has been observed in the case of the open ground storey RC-infilled frame after the retrofit. Thus, this paper aims to present a most effective way for the seismic evaluation and retrofit of any reinforced concrete structure through the material strain limit approach.

1. Introduction

Nowadays, there is a need for seismic evaluation and retrofit of the structures due to too many problems like design deficiency, construction deficiency, and aging of structures. Retrofitting is defined as the process of modification of an existing structure like buildings and bridges to make them more resistant to seismic activity and other natural disasters. Earthquake is one of the most dangerous natural disasters to structures, so there is a need for special care, mainly in the design of structures. The aging of structures mostly causes the loss of their strength for many reasons like seismic activity, soil failure, and failure of structural members like columns, beams, and slabs due to design and construction deficiency. So, for safety purposes, there is a need for strengthening options.

Alguhane et al. [1] presented the study on the seismic evaluation of an existing 5-storied RC building with different infill configurations at Madinah city. They presented four model systems as bare frame, frame with infill from field test, frame infilled as per ASCE 41, and frame infilled together with open ground story according to ASCE 41. The response modification factor (R) for the 5-storey RC building was evaluated from capacity and demand spectra for all models. The authors concluded that the R-factor increases due to the presence of infill, and it satisfies the requirement of the code (SBC 301), but the bare frame does not satisfy the requirement of the response modification factor. Chaulagain et al. [2] evaluated the response reduction factor (R-factor) of 12 existing irregular reinforced concrete buildings in Kathmandu Valley using pushover analysis. They concluded that the computed values of the R-factor obtained for different bare frame structures were less than the suggested values in the IS 1893 (2002) code. Sadrmomtazi et al. [3] worked on the seismic evaluation and retrofitting of damaged structures in the Sarpole earthquake. In that study, a 3-storey RC building was evaluated in Iran. Initially, they have collected the general information of the sample building and, after that, conducted the NDT. Ultimately after a detailed study, they concluded that a lack of monitoring and construction mistakes such as reinforcement bending, stirrup spacing, and covers makes a more vulnerable structure. Therefore, it is necessary to monitor the building for precise construction to vanish the disaster effect. Dolšek and Fajfar [4] worked on the seismic assessment of a four-storey RC frame with masonry infills by using the N2 method. Three models have been taken for the study purpose as bare frame, partially infilled frame, and fully infilled frame. The results of that study indicate that the infills can completely change the distribution of damage throughout the structure, and it can have a beneficial effect on the structural response. Uva et al. [5] worked on the seismic evaluation of an existing RC framed building located in a high seismic risk area in Calabria. An experimental test has been carried out on the building to assess the present condition and its quality. Nonlinear static pushover analysis has been performed on the bare frame and infilled frame models. The results of that study recommended that the failure mechanism of the infilled frame was less as compared with the bare frame due to the presence of infill walls. Cavaleri et al. [6] worked on the influence of column shear failure on the seismic assessment of RC-infilled structures. The seismic performance of the RC structure was evaluated by using concentric equivalent struts for modeling infills and the level of the additional shear on the columns. Through that paper, the use of concentric struts for the infills may overestimate the structural capacity and the additional shear demand produced by infills just for the base columns. Mazza [7] evaluated a hospital structure against in-plane and out-of-plane seismic collapse of masonry infills. In that study, the base-isolation as a retrofitting technique was used for improving the IP (in-plane) and OOP (out-of-plane) response of infill walls. Mazza [8] worked on dissipative steel exoskeletons (DEX) for the seismic control of RC buildings. The dissipative exoskeleton (DEX) appears convenient from energetic and functional points of view. Three arrangements of DEX were applied externally: parallel (DEX.Pa) and perpendicular (DEX.Pe) to all façades of the existing building and a mixed (DEX.Mi) solution. The results of that study indicate that the axial load in the columns whose intensity is larger for DEX.Pa than DEX.Pe and DEX.Mi due to overturning moments induced by seismic loads and DEX.Pe is the most attractive solution for the tensile axial load transformation to the foundation. El-Betar [9] worked on the seismic vulnerability of two existing RC buildings in Egypt. The two case studies were selected for the seismic evaluation purpose as old and new school buildings. The results of the study recommended that an old school building was a more vulnerable structure under high seismic load.

Most of the residential buildings were designed for gravity loading only in a high seismic region, so there is always a need to check the seismic performance of the structures by using pushover analysis. The pushover analysis may help engineers to take the initiative for rehabilitation work by identifying the weak elements [10, 11].

Arya and Agarwal [12] explained the guidelines on seismic evaluation and the strengthening of existing reinforced concrete buildings. The preliminary and detailed evaluation process of the existing buildings was explained clearly. As per the report site visit, collection of data, configuration-related checks, and strength-related checks are required for preliminary evaluation purposes, and if all these checks are satisfied with structural integrity, then there is no need to go for detailed evaluation. In detailed evaluation, linear static or linear dynamic analysis is required to check the demand to capacity ratio of structural members. If the demand to capacity ratio is greater than 1, then the structural member is considered as a deficient member, and the different retrofitting strategies are applied (such as RC jacketing, addition of infills, shear wall, and bracings) to strengthen the structure.

The failure pattern of a reinforced concrete structure is an important aspect to be assessed. Many researchers have worked on the failure pattern of the RC frames in several ways. The "material strain limit approach" is the newly developed and most realistic method to identify the damage of the reinforced concrete structures. Based on this approach, it will help to get information regarding the actual damage in the materials of RC structures [13–15]. Reliability analysis of RC structures is very important because of the consideration of uncertainties as an important dimension of performance-based earthquake engineering, which accounts for uncertainties in modeling and design [16, 17].

The present study attempts to seismically evaluate the reinforced concrete buildings with masonry infills in a systematic approach, viz., by using the "material strain limit approach," and the retrofit is based on the deficiencies in the building. Also, we present a comparison between the values of seismic design parameters (R-factor, ductility, and overstrength factor) obtained from numerical analyses before and after the retrofit of the RC buildings.

2. Methodology

2.1. Adaptive Pushover Analysis. In recent years, the application of pushover analysis is generally used to check the nonlinear response of structures. It represents a significant alternative solution for nonlinear dynamic analysis of structures. In the case of a multistoried structure, ignoring the effect of higher modes is one of the limitations of such approaches. Some researchers proposed considering higher mode effects depending on adaptive pushover procedures, which include the increasing variation in the dynamic properties like time period and frequency [18, 19]. For this, the applied load is revised at every incremental action depending on the current dynamical properties of the structure.

Antoniou and Pinho [20] used a force-based adaptive pushover analysis, in which the lateral load is continuously revised at every single step during the eigenvalue analysis. The SRSS method was used to combine the responses of each mode. In this advanced static analysis method, the spectral amplification part is also important for updating the load vectors. According to the literature for the adaptive pushover case, one can introduce the record of earthquake ground motion and define the level of damping. In the present study, for spectral amplification, we considered the accelerogram time history of the Chi-Chi earthquake (magnitude: 7.6, location: 23.78° N and 121.09° E, and recording station: TCU045) in Taiwan (date: 20 September 1999) taken from PEER database [21] as shown in Figure 1 and its response spectrum is shown in Figure 2. In the present study, adaptive and conventional pushover analyses have been used for comparison purposes, and finally, all the seismic design parameters have been evaluated based only on the adaptive pushover analysis due to its more realistic nature as compared with the conventional pushover analysis.

2.2. Response Reduction Factor. The R factor is generally used to minimize elastic response to inelastic response structures. In other words, the response reduction factor (R) is defined as the ratio of elastic strength to inelastic design strength. From the existing literature, it is seen that the R-factor is mainly a function of three factors, viz., the ductility factor, the overstrength factor, and the redundancy factor. It is mathematically expressed as

$$R = R_d \times R_O \times R_R, \tag{1}$$

where R is the response reduction factor, R_d is the ductility reduction factor, R_O is the overstrength factor, and R_R is the redundancy factor. Figure 3 provides an explanation of all these factors. According to the BIS (Bureau of Indian Standards) code provisions, it is mathematically represented as follows [23, 24]:

$$2R = R_d \times R_O. \tag{2}$$

According to ATC-19 [25], the product of the ductility reduction factor, redundancy, and the overstrength factor is the response reduction factor.

2.2.1. Ductility Reduction Factor. The ductility reduction factor (R_d) provides a measure of the global nonlinear response of a structure. It mainly depends on the ductility and the fundamental time period of any structure. The displacement ductility μ is expressed as

$$\mu = \frac{\Delta_{max}}{\Delta_y}, \tag{3}$$

where Δ_{max} is the maximum displacement corresponding to the peak base shear of the pushover curve and Δ_y is the yield displacement calculated by using the reduced stiffness method based on the bilinearization curve (Figure 4) [26].

The R-μ-T relationships developed by Newmark and Hall [27] were used to evaluate R_d as follows:

If time period <0.2 seconds, $R_d = 1$

If 0.2 seconds < time period < 0.5 seconds, $R_d = \sqrt{2\mu - 1}$

If time period> 0.5 seconds, $R_d = \mu$

2.2.2. Overstrength Factor. The overstrength factor (R_O) is a measure of the reserved strength present in a structure. The main sources of the overstrength factor are (i) material strength, (ii) load factors and their combination, (iii) participation of nonstructural elements like infill walls, and (iv) redundancy. It may be expressed as follows:

$$R_O = \frac{V_y}{V_d}, \tag{4}$$

where V_y is the ideal yield base shear and V_d is the design base shear.

2.2.3. Redundancy Factor. The redundancy factor (R_R) is usually defined as the gap between the local yield point to the global yield point of a structure. Any building should have a high degree of redundancy for lateral resistance. In this study, the redundancy factor is incorporated into the overstrength factor.

Recommended values of the response reduction factor (R) by the BIS (Bureau of Indian Standards) code are shown in Table 1.

3. Seismic Evaluation and Retrofitting Procedure

3.1. General Practice for Seismic Evaluation and Retrofitting. Generally, for the seismic evaluation purpose of structures, an equivalent static method (linear static method) is used. In this method, the base shear is calculated based on different seismic parameters, like zone factor, importance factor, R-factor, spectral acceleration coefficient, and seismic weight. After the calculation of the base shear, it is distributed at each floor as per IS (Indian Standards) code provisions. Subsequently, a check is done of the deficient members of structures based on different deformations criteria, e.g., check of the demand (D) to capacity (C) ratio (D/C). If D/C > 1, then the member is called a deficient member. After that, different retrofitting techniques are used for the respective deficient members, and thus, the structure is finally retrofitted.

3.2. A Systematic Approach for Seismic Evaluation and Retrofit. In current practice, a linear static method is more popular for the seismic evaluation of structures. In this

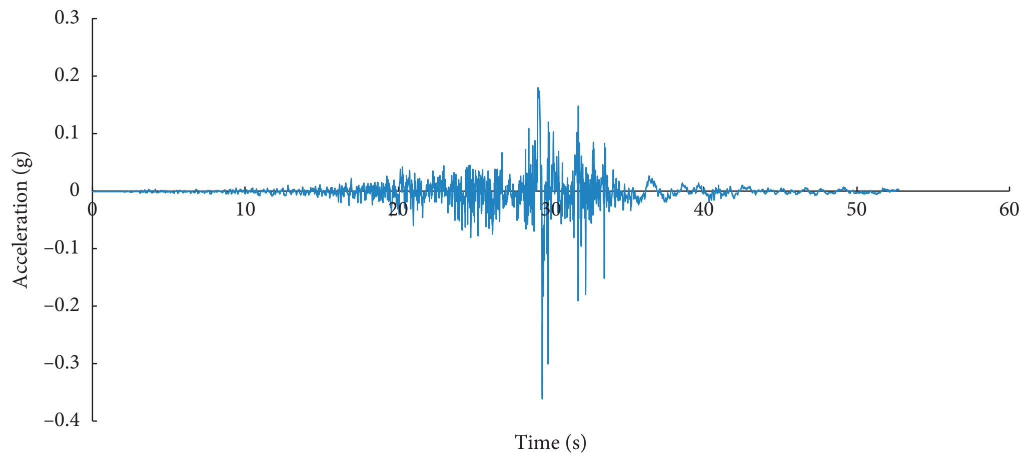


Figure 1: Acceleration time history of the Chi-Chi earthquake [21].

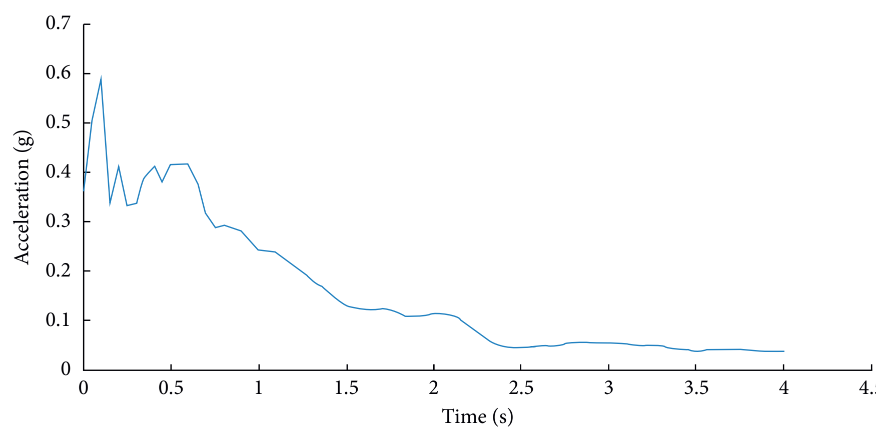


Figure 2: Acceleration response spectrum of the Chi-Chi earthquake [22].

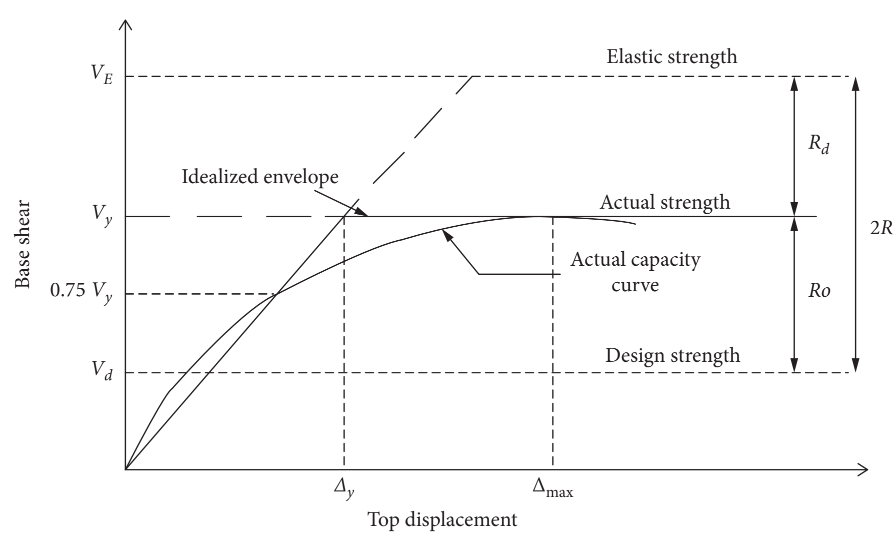


Figure 3: Interrelation between response reduction factor, overstrength factor, and ductility reduction factor [23, 24].

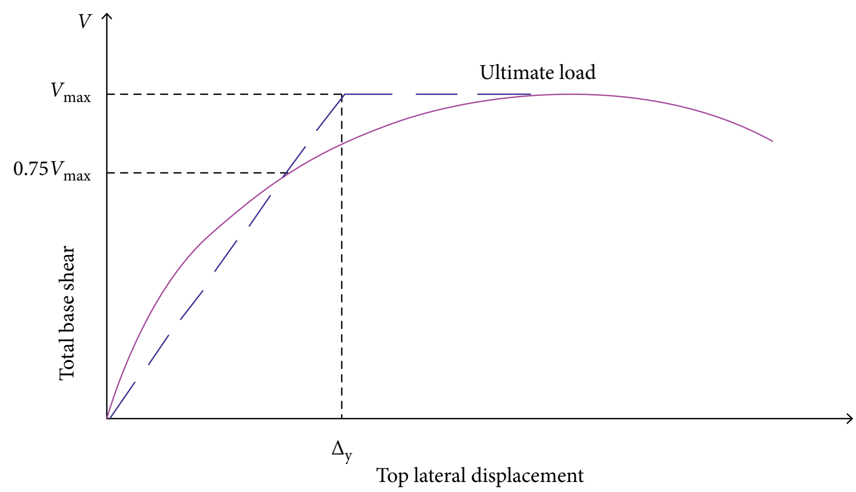


Figure 4: Reduced stiffness method [26].

Table 1: Recommended values of the response reduction factor (R) by IS1893 (Part-1): 2016 [28].

Frame system	R value
Ordinary moment resisting frame	3
Special moment resisting frame	5

article, a systematic approach, i.e., the nonlinear static adaptive pushover analysis has been used for the seismic assessment of the RC structure through the "material strain limit approach." The following steps are used:

(1) Calculate the design base shear value based on the seismic weight of the structure as per IS 1893 (Part-1):2016 [28]
(2) Distribute the design base shear value at each floor
(3) Perform an adaptive pushover analysis of the structure
(4) Based on the adaptive pushover analysis results, the pushover curves are plotted, and the seismic parameters of the structure are calculated
(5) Based on the material strain limits approach (i.e., performance criteria), the deficient members (if D/C > 1) present in the structure can be identified
(6) Thereafter, different techniques of retrofitting (local and global retrofitting schemes) can be applied to the deficient members

The "material strain limit approach" is a method for the seismic assessment of the reinforced concrete structures based on the threshold strain limit of concrete and steel to identify the actual damage state of structural members, i.e., microlevel evaluation. This method gives a more precise and reliable solution for the seismic assessment of RC structures. In this study, the "material strain limit approach" is first time used for the seismic evaluation of the RC structures with masonry infills.

4. Model Description

For this study, a four-storey three-dimensional building (floor to floor height 3 m) symmetrical on plan with 3 bays (each span 4 m) in both directions is studied. The building is considered in seismic zone "IV" and designed for gravity and lateral earthquake load. The building is modeled using the SeismoStruct software [22]. Models were studied for seismic evaluation and retrofitting of structures with an opening in infills as follows:

(1) Full RC-infilled frame in both directions
(2) Open ground storey RC-infilled frame in both directions

Figure 5 shows the building plan, while Figure 6 shows the models of the building. Table 2 provides the structural details of the building. Tables 3 and 4 show the column and beam dimensions, respectively. The structural detailing of the column and beam is shown in Figure 7.

4.1. Inelastic Infill Panel Element. The inelastic infill element is characterized by four axial struts and two shear springs, as shown in Figure 8, where X_{oi} and Y_{oi} represent the horizontal and vertical offsets, respectively, d_m is the diagonal strut length, and h_z is the equivalent contact length. Four node panel masonry elements were developed by Crisafulli [29]. This element accounts for separately shear and compressive behavior of masonry infill and adequately represents the hysteretic response. The stiffness reduction factor to consider opening effects in the infill in numerical modeling is given as follows:

$$W_{do} = (1 - 2.5A_r)W_d, \tag{5}$$

where W_{do} is the width of the diagonal strut with an opening in infill, W_d the width of the diagonal strut, and A_r is the ratio of opening area to face area of infill. Equation (5) is valid for openings in walls ranging from 5% to 40%. In this paper, the opening size is considered approximately 20%.

4.1.1. Sample Calculation of Equivalent Parameters of Masonry Infill

For bay length = 4 m, storey height = 3 m and thickness of infill (t) = 230 mm

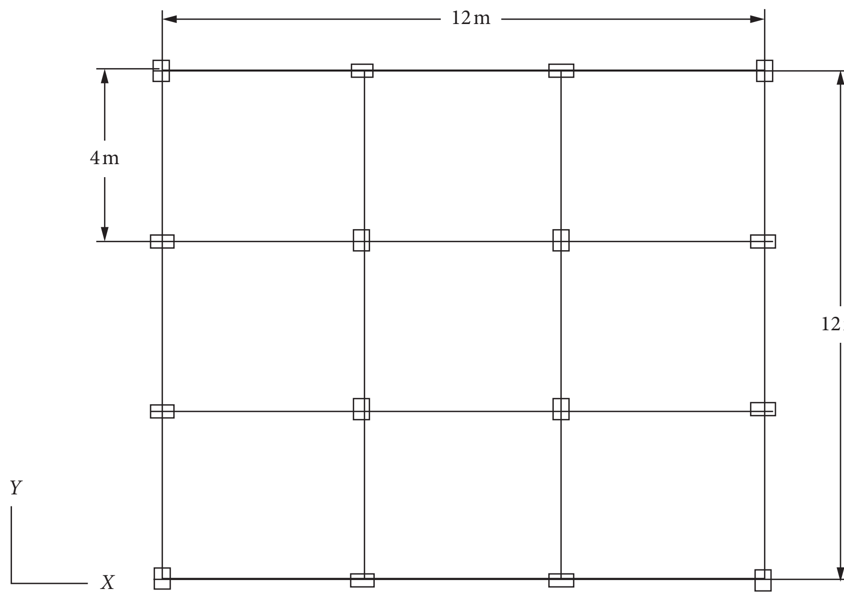


FIGURE 5: Plan of the building.

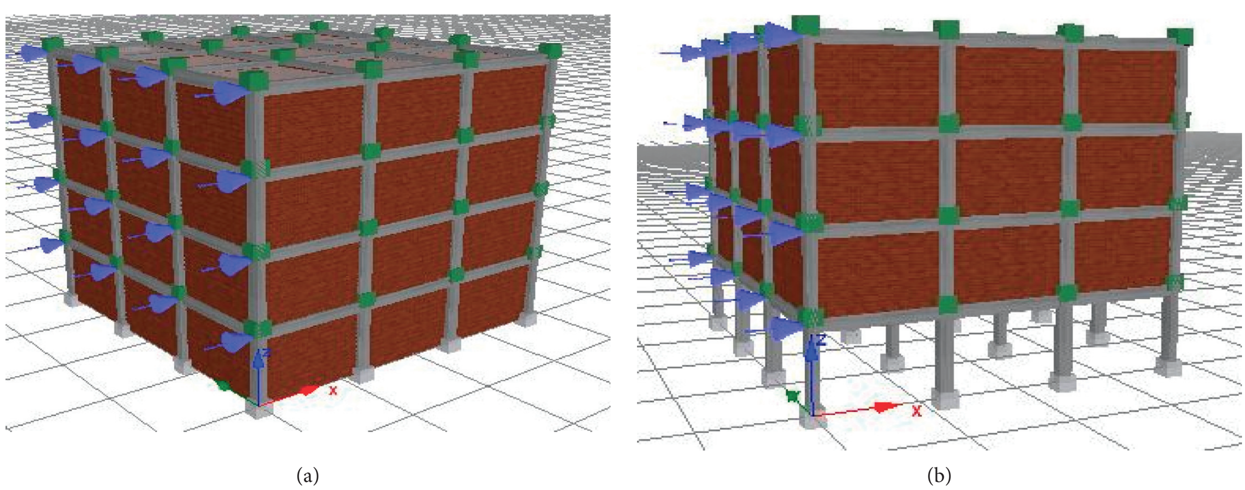


FIGURE 6: The two models of the building: (a) full RC-infilled frame; (b) open ground storey RC-infilled frame.

TABLE 2: Structural details of the building.

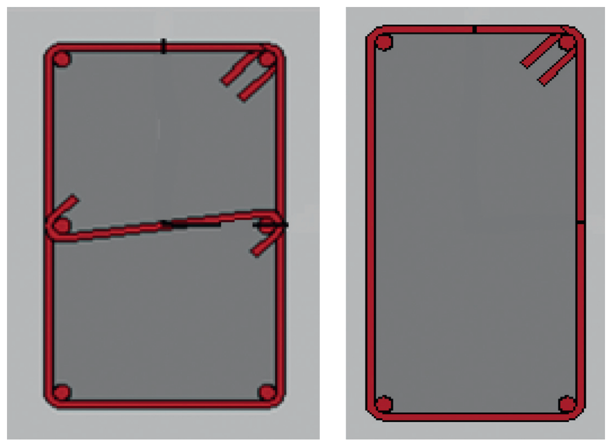
Type of structure	Special moment resisting frames
Number of storeys	4
Seismic zone	IV
Floor height	3 m
Bay span	4 m along the X direction and Y direction
Infill wall thickness	230 mm
Comp. Strength of masonry	5 MPa
Young's modulus of masonry	2750 MPa
Width of strut with opening in infill	262 mm
Equivalent contact length (h_z)	20.37%
Horizontal offset (Xo)	5.62%
Vertical offset (Yo)	7.5%
Type of soil	Medium stiff soil
Column size (mm)	300×450
Beam size (mm)	250×450
Slab depth (mm)	150
Live load (kN/m^2)	3
Material	M-25 grade concrete and Fe-415 reinforcement
Damping in structure	5%
Importance factor	1.2

TABLE 3: Column dimensions and detailing.

Column	Size (mm)	Main reinforcement	Shear reinforcement
All columns of the building	300 × 450	4 nos. of 16 mm diameter at the corner and 2 nos. of 16 mm on the longer side	8 mm dia. @ 100 mm c/c

TABLE 4: Beam dimensions and detailing.

Beam	Size (mm)	Main reinforcement	Shear reinforcement
All beams of the building	250 × 450	2 nos. of 16 mm diameter at the top as well as the bottom	8 mm dia.@ 100 mm c/c



(a) (b)

FIGURE 7: Structural detailing of (a) column detailing and (b) beam detailing.

Compressive strength of infill $(f_m) = 5$ MPa and modulus of elasticity of infill $= 550 \times f_m = 2750$ MPa

Infill panel net length = 4–0.45 = 3.55 m

Infill panel net height = 3–0.45 = 2.55 m

Diagonal length $(d_{\text{inf}}) = 4.37$ m

$\lambda_h = h\sqrt[4]{(E_{\text{inf}} t \sin 2\varnothing)/(4E_c I_c h_{\text{inf}})}$ (dimensionless parameter)

$Z = (\pi/2\lambda_h) \times h$

$h_z = (z/3h_{\text{inf}}) \times 100$

Equivalent contact length $(h_z) = 20.37\%$

For the evaluation of equivalent contact length of double strut, Stafford and Smith (1966) adopted the value as $Z/3$, and width of strut is calculated as follows:

Width of strut $= 0.175\ d_{\text{inf}} (\lambda_h)^{-0.4}$

Width of strut with opening $(b_w) = 62$ mm

Area of strut for infill with opening $= b_w \times t = 60260.00\ \text{mm}^2$

Horizontal offset $(X_{oi}) = 5.62\%$

Vertical offset $(Y_{oi}) = 7.5\%$

5. Results and Discussion

5.1. Pushover Curves. The utilization of nonlinear static analysis came into practice in the 1970s, but the importance of this pushover analysis has been realized primarily in the last two decades. In this study, the adaptive pushover and conventional pushover analysis have been used for the simulation of different models. For comparison purposes, we have conducted the two analyses for different models, but the calculation of several parameters like strength, ductility, overstrength factor, and R-factor is evaluated only from the adaptive pushover analysis due to the more realistic seismic analysis as compared with conventional pushover analysis. The significance of infills which play an important role in the reinforced concrete frame has been quantified.

Figure 9 shows the pushover curves of two models, namely, full RC-infilled frame and open ground storey RC-infilled frame in terms of horizontal seismic coefficient versus drift in %. As per Figure 9, there is a slight difference in adaptive and conventional pushover curves. The important parameters such as ductility, overstrength factor, and R-factor have been obtained from the adaptive pushover curves before the retrofit, which are presented in Tables 5 and 6. The base shear is lower in the open ground storey RC-infilled frame as compared with the full RC-infilled frame due to the absence of masonry infills at the ground storey.

5.2. Damage Patterns. Engineers must be capable of identifying the instants at which different performance limit states (e.g., structural damage) are reached. This can be efficiently carried out in the SeismoStruct software through the definition of performance criteria, whereby the attainment of a given threshold value of material strain is monitored during the analysis of a structure. Material strains usually constitute the best parameter for the

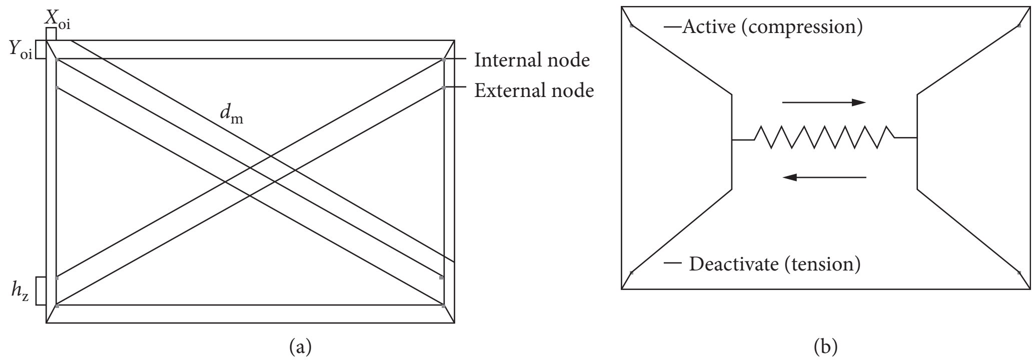


Figure 8: Inelastic infill panel element [29].

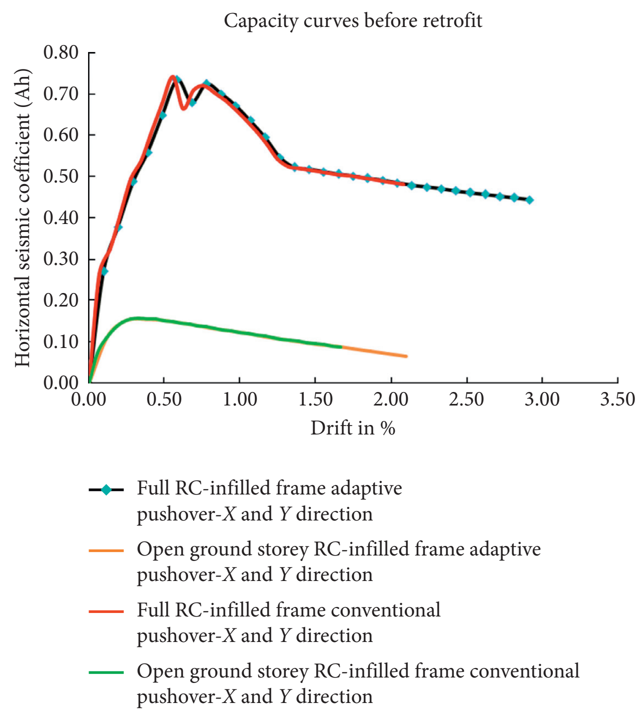


Figure 9: Pushover curves of different frames before the retrofit.

Table 5: Comparison of different parameters for the full RC-infilled frame.

Parameters	Before retrofit: Full RC-infilled frame In X-axis	Before retrofit: Full RC-infilled frame In Y-axis	After retrofit: Full RC-infilled frame In X-axis	After retrofit: Full RC-infilled frame In Y-axis	Remarks
Ultimate capacity	7056.10 kN	7063.99 kN	7887.47 kN	8072.72 kN	After retrofitting, 11.78% increases in X-axis and 14.27% increases in the Y-axis
Yield displacement	60.48 mm	60.42 mm	66.33 mm	68.5 mm	After retrofitting, 9.67% increases in the X-axis and 13.37% increases in the Y-axis
Maximum displacement	70 mm	70 mm	100 mm	100 mm	After retrofitting, 42.85% increases in X-axis and Y-axis
Ductility	1.16	1.16	1.51	1.46	After retrofitting, 30.17% increases in X-axis and 25.86% increases in the Y-axis
Ductility reduction factor	1.15	1.15	1.42	1.38	After retrofitting, 23.47% increases in X-axis and 20.00% increases in Y-axis
Overstrength factor	10.19	10.20	11.39	11.66	After retrofitting, 11.77% increases in X-axis and 14.31% increases in the Y-axis
Time period	0.32 s	0.32 s	0.31 s	0.31 s	After retrofitting, 3.125% decreases in X-axis and Y-axis
R-factor	5.85	5.86	8.08	8.04	After retrofitting, 38.11% increases in X-axis and 37.20% increases in the Y-axis

Table 6: Comparison of different parameters for the open ground storey RC-infilled frame.

Parameters	Before retrofit		After retrofit		Remarks
	Open ground storey RC-infilled frame In X-axis	Open ground storey RC-infilled frame In Y-axis	Open ground storey RC-infilled frame In X-axis	Open ground storey RC-infilled frame In Y-axis	
Ultimate capacity	1505.98 kN	1498.58 kN	6241.19 kN	6250.54 kN	After retrofitting, 314.42% increases in X-axis and 317.09% increases in the Y-axis
Yield displacement	22.14 mm	21.93 mm	54.62 mm	54.72 mm	After retrofitting, 146.70% increases in the X-axis and 149.52% increases in the Y-axis
Maximum displacement	42 mm	42 mm	93.33 mm	86.67 mm	After retrofitting, 122.21% increases in X-axis and 106.35% increases in the Y-axis
Ductility	1.89	1.91	1.70	1.58	After retrofitting, 10.05% decreases in X-axis and 17.27% decreases in the Y-axis
Ductility reduction factor	1.67	1.68	1.55	1.47	After retrofitting, 7.18% decreases in X-axis and 12.5% decreases in Y-axis
Overstrength factor	2.17	2.16	9.01	9.02	After retrofitting, 315.20% increases in X-axis and 317.59% increases in the Y-axis
Time period	0.48 s	0.48 s	0.32 s	0.32 s	After retrofitting, 33.33% decreases in X-axis and Y-axis
R-factor	1.81	1.81	6.98	6.62	After retrofitting, 285.63% increases in X-axis and 265.74% increases in the Y-axis

identification of the performance state of a given structure when compared with other existing methods. It is possible in the SeismoStruct program because, in this software, the distributed inelasticity (i.e., realistic phenomena) is given to each structural member, so it is easy to identify the actual damage phenomena based on the materials in a structure.

To check the damage patterns of the structures, the performance criteria based on material strain used in the present numerical simulation are (1) yield strain limit for steel: 0.0025, (2) crushing strain limit for unconfined concrete: 0.0035, (3) crushing strain limit for confined concrete: 0.008, and (5) fracture strain limit for steel: 0.06 [13–15, 30]. The different damage states have been described in detail for the different models as in Tables 7 and 8.

As per Figure 10, the first yielding of reinforcing steel occurred at base shear of 4694.47 kN and displacement of 35 mm. Thus, this frame is able to sustain more load as compared with open ground storey RC-infilled frames. First crushed unconfined concrete, i.e., spalling of cover concrete occurred at base shear of 6527.23 kN and displacement of 81.67 mm; also, the first crushed confined concrete, i.e., crushing of the core portion of concrete occurred at 5248.89 kN and displacement of 151.67 mm.

As per Figure 11, the first yielding of steel occurred at base shear of 4688.56 kN and displacement of 35 mm. This frame is able to sustain more load as compared with open ground storey RC-infilled frames. First crushed unconfined concrete, i.e., spalling of cover concrete occurred at base shear of 6529.16 kN and displacement of 81.67 mm also; first crushed confined concrete, i.e., crushing of the core portion of concrete occurred at 5254.25 kN and displacement of 151.67 mm.

As per Figure 12, the first yielding of reinforcing steel occurred at base shear of 1437.30 kN and displacement of 28.01 mm. This frame sustains less load as compared with the full RC-infilled frame. First crushed unconfined concrete, i.e., spalling of cover concrete occurred at base shear of 1498.59 kN and displacement of 42.02 mm also; first crushed confined concrete, i.e., crushing of the core portion of concrete occurred at 1152.56 kN and displacement of 125.95 mm.

As per Figure 13, the first yielding of steel occurred at base shear of 1434.11 kN and displacement of 28 mm. This frame sustains less load as compared with the full RC-infilled frame. First crushed unconfined concrete, i.e., spalling of cover concrete occurred at base shear of 1476.10 kN and displacement of 56 mm also; first crushed confined concrete, i.e., crushing of the core portion of concrete occurred at 1159.98 kN and displacement of 126 mm.

5.3. Retrofit of the Building. To check the damage patterns of different frames, the performance criteria based on the material strain used in the present numerical simulation are (i) yield strain limit for steel: 0.0025, (ii) crushing strain limit for unconfined concrete: 0.0035, (iii) crushing strain limit for confined concrete: 0.008, and (iv) fracture strain limit for steel: 0.06 [13–15, 30]. Based on the above values, the deficient members in the structure are identified by a critical

Table 7: Material strain level of the open ground storey RC-infilled frame.

Material strain level	Before retrofit				After retrofit			
	Open ground storey RC-infilled frame-*X*		Open ground storey RC-infilled frame-*Y*		Open ground storey RC-infilled frame-*X*		Open ground storey RC-infilled frame-*Y*	
	Displacement (mm)	Base shear (kN)	Displacement (mm)	Base shear (kN)	Displacement (mm)	Base shear (kN)	Displacement (mm)	Base shear (kN)
First yielding of steel	28.00	1434.11	28.01	1437.30	26.67	3817.24	26.67	3814.01
First crushing of unconfined concrete	56.00	1476.10	42.02	1498.59	80.00	6038.78	86.67	6250.54
First crushing of confined concrete	126.00	1159.98	125.95	1152.56	153.33	5237.76	146.67	5416.95

Table 8: Material strain level of the full RC-infilled frame.

Material strain level	Before retrofit				After retrofit			
	Full RC-infilled frame-*X*		Full RC-infilled frame-*Y*		Full RC-infilled frame-*X*		Full RC-infilled frame-*Y*	
	Displacement (mm)	Base shear (kN)	Displacement (mm)	Base shear (kN)	Displacement (mm)	Base shear (kN)	Displacement (mm)	Base shear (kN)
First yielding of steel	35.00	4688.56	35.00	4694.47	41.67	5282.78	33.33	4627.86
First crushing of unconfined concrete	81.67	6529.16	81.67	6527.23	83.33	7159.72	91.67	7738.15
First crushing of confined concrete	151.67	5254.25	151.67	5248.89	175.00	5917.69	175.00	6697.84

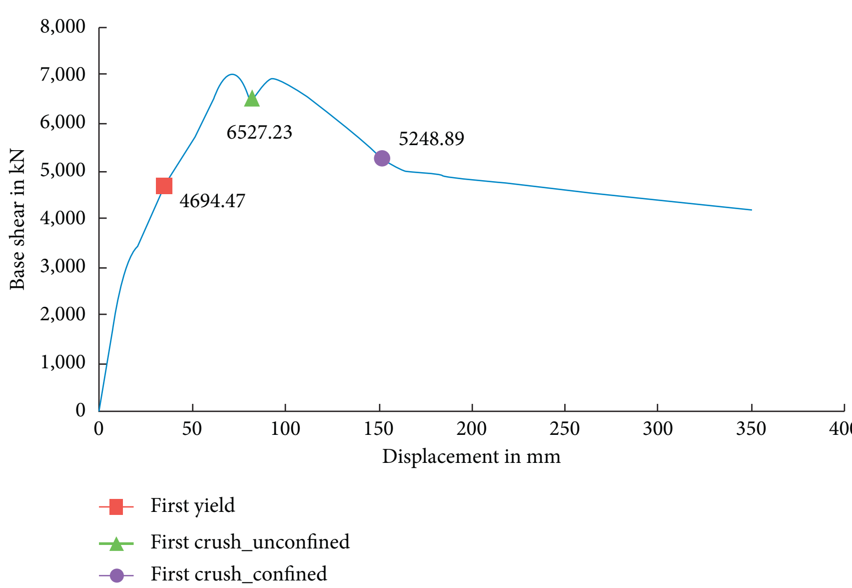


Figure 10: Damage pattern of full RC-infilled frame in *Y* direction.

combination of strain limit criteria in *X* as well as *Y* directions. There were 8 columns (at ground storey) deficient in the case of full RC-infilled frame, while all ground columns, i.e., 16 columns were deficient in the case of the open ground storey RC-infilled frame. Consequently, the full RC-infilled frame has been retrofitted with the RC-jacketing method, size of retrofitted column 450×600 mm, i.e., they are retrofitted with M30 grade of concrete around the existing member as the core of a column with the 4 numbers of 20 mm diameter of 415-grade steel, as detailed in Figure 14. The retrofitted plan of the full RC-infilled frame is shown in Figure 15.

The open ground storey RC-infilled frame has been retrofitted with the RC-jacketing method as well as the

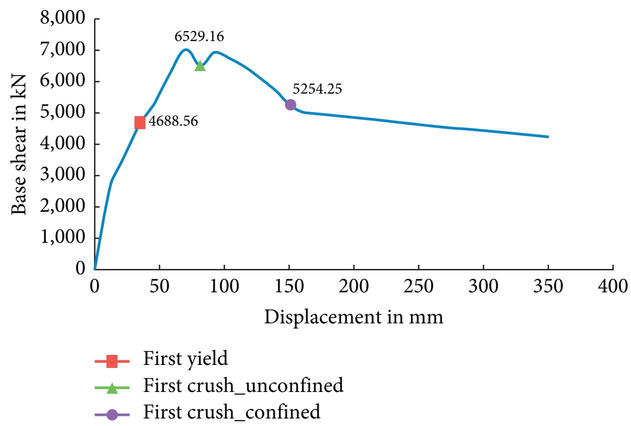


Figure 11: Damage pattern of full RC-infilled frame in X direction.

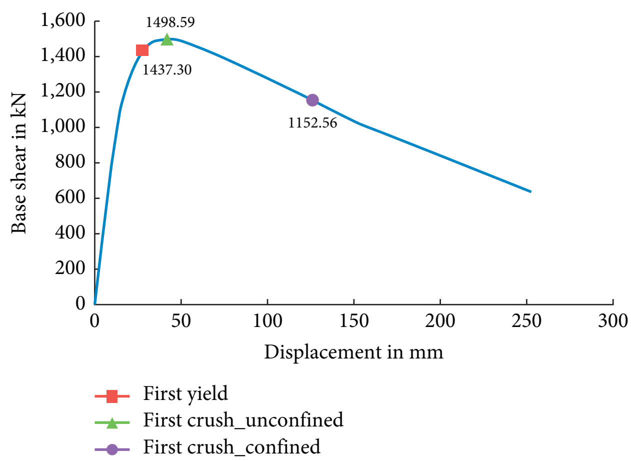


Figure 12: Damage pattern of open ground storey RC-infilled frame in Y direction.

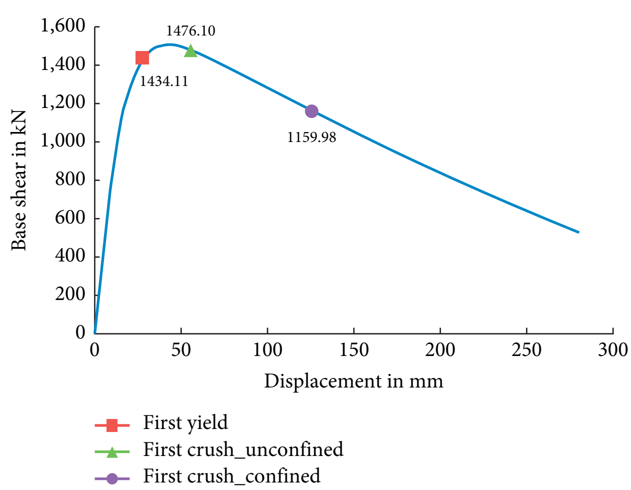


Figure 13: Damage pattern of open ground storey RC-infilled frame in X direction.

incorporation of infill panels at the corners in both directions and also provided the four infill panels at central core portion so, i.e., 12 infills are provided at ground storey adopted as the global strengthening method. For the open

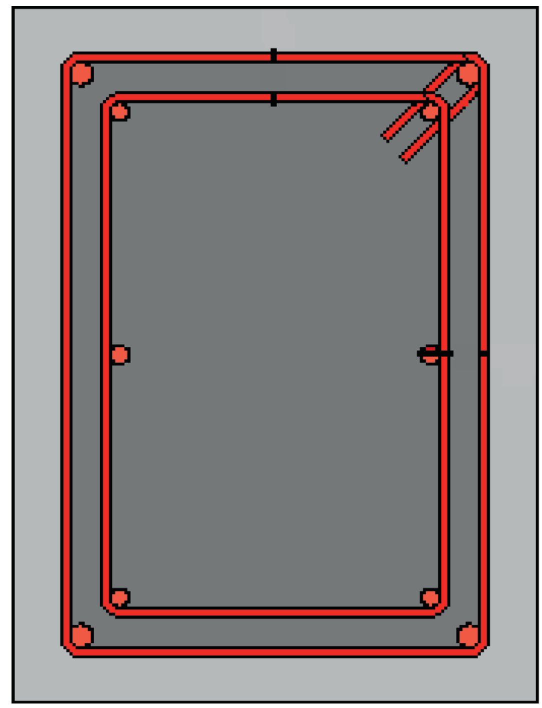


Figure 14: RC jacketed column.

ground storey case, it was adopted local strengthening as well as global strengthening method because this case is more vulnerable to lateral loads, but in the case of full RC-infilled frame, there is a need of only a few members to retrofit by local strengthening, i.e., RC-jacketing method. The retrofitted plan of the open ground storey RC-infilled frame is shown in Figure 16.

Figure 15 shows the retrofitted plan of the full RC-infilled frame. There were eight deficient column members found at the ground storey based on the material strain limit approach. The four corner columns and four middle columns at the ground storey of the building were identified as deficient, so the RC-jacketing method (local strengthening) was used to strengthen these deficient members of the building.

Figure 16 shows the retrofitted plan of the open ground storey RC-infilled frame. There were 16 deficient column members found at the ground storey based on the material strain limit approach. The RC-jacketing method (local strengthening) was used to strengthen all the deficient members of the building and also the addition of masonry infills (global strengthening) at the ground storey of the building, i.e., masonry infills were added at the middle core portion and both directions of each corner as shown in Figure 16.

5.3.1. Pushover Curves after Retrofit. Figure 17 shows the pushover curves of different frames after retrofit in terms of horizontal seismic coefficient versus drift in %. As per Figure 17, there is a slight difference in adaptive and conventional pushover curves. The important parameters such as ductility, overstrength factor, and R-factor have been obtained from the adaptive pushover curves after the

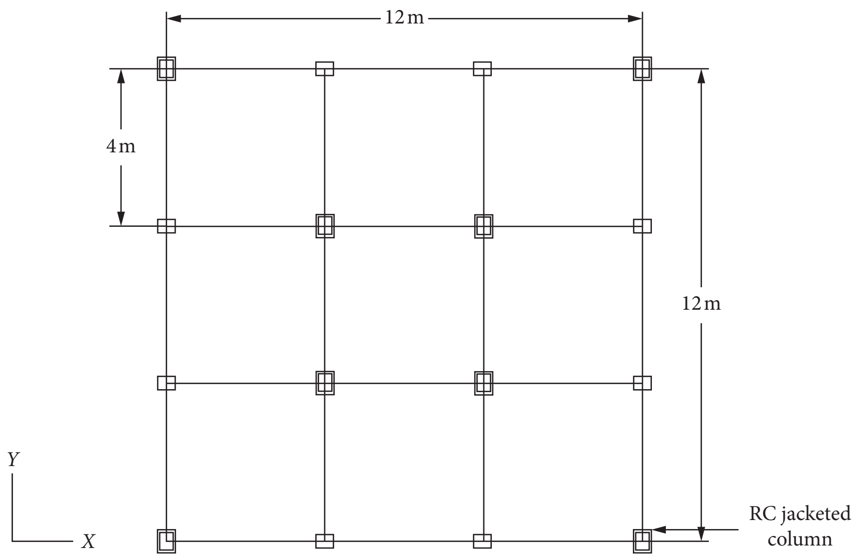


FIGURE 15: Retrofitted plan of the full RC-infilled frame.

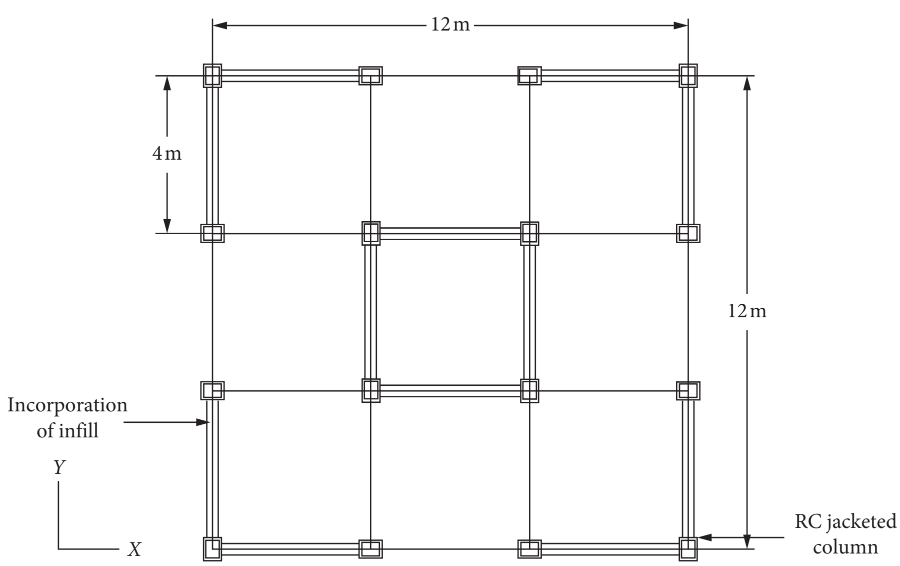


FIGURE 16: Retrofitted plan of the open ground storey RC-infilled frame.

retrofit, which are presented in Tables 5 and 6. The major study observed that the capacity of the frames has been increased significantly after the retrofit.

A comparison of different parameters is presented in Table 5. From Table 5, significant conclusions can be drawn. Based on Table 5, the most important parameter, the R-factor of the building, is increased by averagely (average value in X and Y direction) 37.65% due to the RC jacketing of few ground columns. In full RC-infilled frames, the probability of getting deficient members is less as compared with that of open ground storey RC-infilled frame because infills are present in the ground storey; thus, it lowers the cost of retrofitting of the building. It could be noted that the parameters, viz., ductility, ductility reduction factor, overstrength factor, and response reduction factor are slightly increased as compared with the open ground storey RC-infilled frame after the retrofit.

Based on Table 6, the most important parameter, the R-factor of the building, is significantly increased by averagely (average value in X and Y direction) 275.68% due to RC jacketing of ground columns as well as the addition of infills at the ground storey. In the open ground storey RC-infilled frame, the probability of getting deficient members is more than that of the full RC-infilled frame because infills were not present in the ground storey, which makes the soft storey building increase the cost of retrofitting the building due to such a vulnerable soft storey effect. In this case, the ductility decreases by averagely 13.66% due to the application of both local (RC jacketing of column) and global (addition of infills) retrofitting techniques, and the other parameters like the overstrength factor and the response reduction factor are significantly increased after the retrofit.

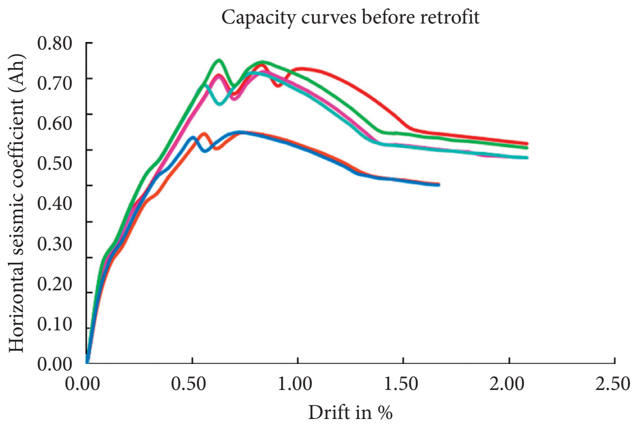


Figure 17: Pushover curves of different frames after retrofit.

Based on Table 7, after the retrofit of open ground storey RC-infilled frame, the deformations like yielding of steel, crushing of unconfined, and confined concrete have occurred late as compared with those before the retrofit. Quantitatively, after the retrofit of the building, the base shear corresponding to the yielding of steel is averagely 2.65 times more as compared with that before the retrofit. Similarly, in the same case, the base shear corresponding to the crushing of unconfined concrete is averagely 4.13 times more as compared with that before the retrofit. Also, in the last stage, the base shear corresponding to the crushing of confined concrete is averagely 4.60 times more as compared with that before the retrofit.

Based on Table 8, after the retrofit of the full RC-infilled frame, the deformations like yielding of steel, crushing of unconfined and confined concrete have occurred late as compared with those before the retrofit. Quantitatively, after the retrofit of the full RC-infilled frame, the base shear corresponding to the yielding of steel is averagely 1.06 times more as compared with that before the retrofit. Similarly, in the same case, the base shear corresponding to the crushing of unconfined concrete is averagely 1.14 times more as compared with that before the retrofit. Also, in the last stage, the base shear corresponding to the crushing of confined concrete is averagely 1.20 times more as compared with that before the retrofit.

5.4. Structural Plan Density. Structural plan density is an important structural parameter that helps to make an earthquake-resistant structure. The area of vertical members of a building has been reduced significantly from about 50–60% of the plinth area in historic masonry buildings to a small 2–4% in modern RC frame buildings. The structural plan density (SPD) is the ratio of the area of the footprint of vertical elements (at ground) resisting the lateral load to the plinth area of the building. SPD is low for gravity load design of buildings and increases for gravity plus lateral load design of buildings (to about 4% or more) [31]. The area of vertical members is necessary to be larger in order to make the buildings more earthquake resistant. In the present study, two models are as follows: (a) full RC-infilled frame and (b) open ground storey RC-infilled frame. We have evaluated the SPD in % for these two models before and after the retrofit and also demonstrated a graph between the average response reduction factor and structural plan density in % as shown in Figures 18 and 19.

5.4.1. Calculation of Structural Plan Density (SPD)

(1) Full RC-infilled frame before the retrofit is as follows:

Total area of footprint of vertical elements = column footprint + infill footprint = $2.16 + 20.01 = 22.17\ \mathrm{m}^2$

Total plinth area = $12.3 \times 12.45 = 153.13\ \mathrm{m}^2$

SPD = $(22.17/153.13) \times 100 = 14.47\%$

(2) Open ground storey RC-infilled frame before the retrofit is as follows:

Total area of footprint of vertical elements = column footprint + infill footprint = $2.16 + 0. = 2.16\ \mathrm{m}^2$

Total plinth area = $12.3 \times 12.45 = 153.13\ \mathrm{m}^2$

SPD = $(2.16/153.13) \times 100 = 1.41\%$

(3) Full RC-infilled frame after the retrofit is as follows:

Total area of footprint of vertical elements = column footprint + infill footprint = $3.24 + 19.59 = 22.83\ \mathrm{m}^2$

Total plinth area = $12.3 \times 12.45 = 153.13\ \mathrm{m}^2$

SPD = $(22.83/153.13) \times 100 = 14.90\%$

(4) Open ground storey RC-infilled frame after the retrofit is as follows:

Total area of footprint of vertical elements = column footprint + infill footprint = $4.32 + 9.57 = 13.89\ \mathrm{m}^2$

Total plinth area = $12.3 \times 12.45 = 153.13\ \mathrm{m}^2$

SPD = $(13.89/153.13) \times 100 = 9.07\%$

Figure 18 shows the graph of average *R*-factor versus structural plan density (%) for the full RC-infilled frame. As per Figure 18, after the retrofit of the full RC-infilled frame, the value of average *R*-factor and structural plan density increases by 37.77% and 2.97%, respectively. Also, we can observe that as the structural plan density increases and the value of the response reduction factor also increases.

Figure 19 shows the graph of average *R*-factor versus structural plan density (%) for the open ground storey RC-infilled frame. As per Figure 19, after the retrofit of the open ground storey RC-infilled frame, the value of average *R*-factor and structural plan density increases by 3.75 times and 6.43 times, respectively. Also, we can observe as the structural plan density increases and the value of the response reduction factor also increases.

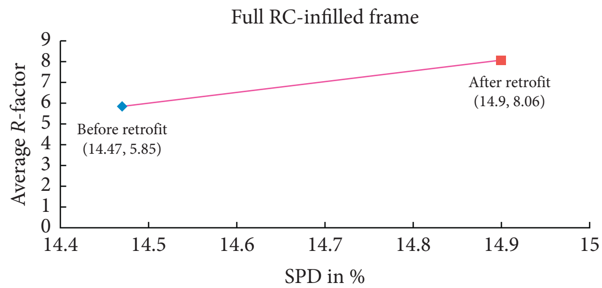


FIGURE 18: Graph of average R-factor versus structural plan density (SPD).

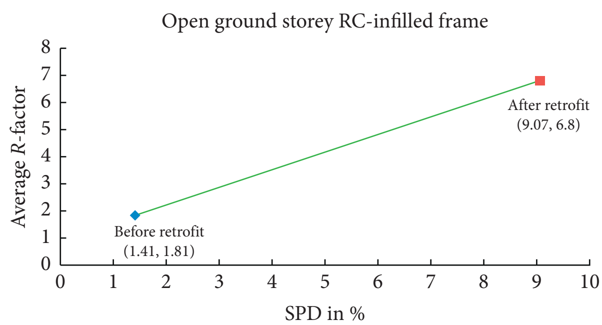


FIGURE 19: Graph of average R-factor versus structural plan density (SPD).

6. Conclusions

According to analytical results, the following conclusions can be drawn from the present study. The base shear values of the retrofitted building are slightly larger as compared with those before retrofit in the case of the full RC-infilled frame. Also, in the case of the open ground storey RC-infilled frame, the values of the base shear increase significantly after the application of local and global retrofitting techniques. In the case of the open ground storey RC-infilled frame, the ductility and ductility reduction factors are slightly decreased after the retrofit due to the application of both local and global retrofitting techniques. The overstrength factors are significantly influenced in the retrofitted building by the application of RC jacketed columns as well as the incorporation of infills. Also, as a result of it, the response reduction factors of the retrofitted buildings are significantly higher as compared with those before the retrofit. Before retrofitting, the computed values of R for the full RC-infilled building are slightly more than the value (i.e., 5 for special moment resisting frames, SMRF) recommended by the IS 1893 (Part I): 2016 code, but in the case of the open ground storey RC-infilled building, the calculated R values are significantly less as compared with the values given by the BIS code due to the soft storey effect. After retrofitting, the computed values of R for the full RC-infilled building are significantly more than the value (i.e., 5 for SMRF) recommended by the IS 1893 (Part I): 2016 code due to the application of local retrofitting technique. However, in the case of open ground storey RC-infilled building, the calculated R values are more than the values given by the BIS (Bureau of Indian Standards) code due to the application of both local and global retrofitting techniques. As the structural plan density increases, the value of the response reduction factor increases due to the higher structural footprint area. Generally, in current construction practices of open ground storey RC buildings, the structural plan density should be greater as possible to make a strong earthquake-resistant structure. Based on the present study, the material strain limit approach is the most effective way to identify the realistic damage state of reinforced concrete structures.

Data Availability

The data used to support the findings of this study are included within the article, and there are no restrictions on data access.

Conflicts of Interest

The authors declare that they have no conflicts of interest regarding the publication of this paper.

References

[1] T. M. Alguhane, A. H. Khalil, M. N. Fayed, and A. M. Ismail, "Seismic assessment of old existing RC buildings with masonry infill in Madinah as per ASCE," *International Journal of Computer and Systems Engineering*, vol. 9, no. 1, pp. 52–63, 2015.

[2] H. Chaulagain, H. Rodrigues, E. Spacone, R. Guragain, R. Mallik, and H. Varum, "Response reduction factor of irregular RC buildings in Kathmandu valley," *Earthquake Engineering and Engineering Vibration*, vol. 13, no. 3, pp. 455–470, 2014.

[3] A. Sadrmomtazi, H. Nsersaeed, and H. Vakhshoor, "Seismic performance evaluation and retrofitting scheme of damaged structures in sarpol earthquake," in *Proceedings of the 4th International Congress on Engineering, Technology & Applied Sciences*, New Zealand, Auckland, June 2019.

[4] M. Dolšek and P. Fajfar, "The effect of masonry infills on the seismic response of four storey reinforced concrete frame-a deterministic assessment," *Engineering Structures*, vol. 30, no. 7, pp. 1991–2001, 2008.

[5] G. Uva, F. Porco, and A. Fiore, "Appraisal of masonry infill walls effect in the seismic response of RC framed buildings: a case study," *Engineering Structures*, vol. 34, pp. 514–526, 2012.

[6] L. Cavaleri, F. Di Trapani, P. G. Asteris, and V. Sarhosis, "Influence of column shear failure on pushover based assessment of masonry infilled reinforced concrete framed structures: a case study," *Soil Dynamics and Earthquake Engineering*, vol. 100, pp. 98–112, 2017.

[7] F. Mazza, "Base-isolation of a hospital pavilion against inplane-out-of-plane seismic collapse of masonry infills," *Engineering Structures*, vol. 228, Article ID 111504, 2021.

[8] F. Mazza, "Dissipative steel exoskeletons for the seismic control of reinforced concrete framed buildings," *Structural Control and Health Monitoring*, vol. 28, no. 3, p. e2683, 2021.

[9] S. A. El-Betar, "Seismic vulnerability evaluation of existing R.C. buildings," *HBRC Journal*, vol. 14, no. 2, pp. 189–197, 2018.

[10] R. Suwondo and S. Alama, "Seismic assessment of RC building designed by local practice," *IOP Conference Series: Earth and Environmental Science*, vol. 426, pp. 1–8, 2020.

[11] M. Rahman and A. U. M. Shajib, "Seismic vulnerability assessment of RC structures: a review," *International Journal of Science & Emerging Technologies*, vol. 4, no. 4, pp. 171–177, 2012.

[12] A. S. Arya and A. Agarwal, *Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings-Guidelines*, National Disaster Management Division, Ministry of Home Affairs, North Block, New Delhi, India, 2012.

[13] M. Shendkar, S. Mandal, R. Pradeep Kumar, and P. R. Maiti, "Response reduction factor of RC-infilled frames by using different methods," *Indian Concrete Institute (ICI) Journal*, pp. 14–23, 2020.

[14] S. Mandal and M. R. Shendkar, "Evaluation of response reduction factor of RC-infilled frames," in *Proceedings of the 17th World Conference on Earthquake Engineering (17WCEE)*, pp. 1–12, Sendai, Japan, September 2020.

[15] M. R. Shendkar, S. Mandal, and R. Pradeep Kumar, "Effect of lintel beam on response reduction factor of RC-infilled frames," *Current Science*, vol. 118, no. 7, pp. 1077–1086, 2020.

[16] M. Grubišić, J. Ivošević, and A. Grubišić, "Reliability analysis of reinforced concrete frame by finite element method with implicit limit state functions," *Buildings*, vol. 9, no. 5, pp. 1–22, 2019.

[17] A. Rezaei Rad and M. Banazadeh, "Probabilistic risk-based performance evaluation of seismically base-isolated steel structures subjected to far-field earthquakes," *Buildings*, vol. 8, pp. 1–22, 2018.

[18] E. Kalkan and S. K. Kunnath, "Adaptive modal combination procedure for nonlinear static analysis of building structures," *Journal of Structural Engineering*, vol. 132, no. 11, pp. 1721–1731, 2006.

[19] B. Gupta and S. K. Kunnath, "Adaptive spectra-based pushover procedure for seismic evaluation of structures," *Earthquake Spectra*, vol. 16, no. 2, pp. 367–391, 2000.

[20] S. Antoniou and R. Pinho, "Advantages and limitations of adaptive and non-adaptive force-based pushover procedures," *Journal of Earthquake Engineering*, vol. 8, no. 4, pp. 497–522, 2004.

[21] Pacific Earthquake Engineering Research Center (PEER), *PEER Strong Ground Motion Databases, Pacific Earthquake Engineering Research Center*, University of California, Berkeley, CA, USA, 2020, https://peer.berkeley.edu/peer-strong-ground-motion-databases.

[22] SeismoStruct, *A Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures*, Seismosoft Ltd., Pavia, Italy, 2020, https://seismosoft.com.

[23] H. Chaulagain, "Assessment of response reduction factor of RC buildings in Kathmandu Valley using nonlinear pushover analysis," MEng thesis, Purbanchal University, Biratnagar, Nepal, 2010.

[24] M. R. Shendkar, D.-P. N. Kontoni, S. Mandal, P. R. Maiti, and D. Gautam, "Effect of lintel beam on seismic response of reinforced concrete buildings with semi-interlocked and unreinforced brick masonry infills," *Infrastructures*, vol. 6, no. 1, pp. 1–18, 2021.

[25] ATC-19, *Structural Response Modification Factors, ATC-19 Report*, Applied Technology Council (ATC), Redwood City, CA, USA, 1995.

[26] R. Park, "Ductility evaluation from laboratory and analytical testing," in *Proceedings of the 9th World Conference on Earthquake Engineering*, vol. 8, pp. 605–616, Tokyo, Japan, August 1988.

[27] N. M. Newmark and W. J. Hall, *Earthquake Spectra and Design*, Earthquake Engineering Research Institute, Berkeley, CA, USA, 1982.

[28] IS 1893, *Criteria for Earthquake Resistant Design of Structures-Part 1 General Provisions and Buildings*, Bureau of Indian Standards, New Delhi, India, 2016.

[29] F. J. Crisafulli, *Seismic behaviour of reinforced concrete structures with masonry infills*, Ph.D. thesis, University of Canterbury, Christchurch, New Zealand, 1997.

[30] P. T. Aswin, "Seismic evaluation of four story reinforced concrete structure by non-linear static pushover analysis," Bachelor of Technology thesis, National Institute of Technology, Rourkela, Odisha, 2013.

[31] A. R. Vijayanarayanan, R. Goswami, and C. V. R. Murty, *Special Class of Open Ground Storey RC Buildings Built in India UNSAFE During Earthquakes*, Indian Institute of Technology Madras, Chennai, Tamil Nadu, 2012.