Seismic Performance Assessment of Existing RC Frames with Different Ultimate Concrete Strains

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Abstract

In recent years, because of the older version code, inadequate design, lacks of construction supervision, change in loading pattern, damages and casualties of earthquakes or environmental degradation, buildings at risk need to be investigated frequently for safety purpose. To increase the strength and ductility capacities of deficient reinforced concrete (RC) beams, columns and beam-column joints, retrofitting may require. In this paper, a numerical investigation using nonlinear static pushover analysis is conducted to assess the seismic behavior of existing moment resisting RC frames. In numerical modeling, different plastic hinge lengths as well as different concrete ultimate strain conditions of RC members are considered. Pushover analysis has been carried out with the commercial software ETABS v.9.6.0 to evaluate structural behavior of RC frames located in a seismic region. Hinge properties simulating moment-rotation behavior of frame members considering different plastic hinge lengths as well as concrete ultimate strains are evaluated. Pushover curves are compared with each other to determine the plastic hinge length and strain values which provide better agreement with that of the default properties. Seismic performance criteria in terms of ductility, overstrength as well as response modification factor for frames are determined from pushover curves. From the analyses in general, the load carrying capacity as well as displacement at maximum lateral load and interstory drift index at any floor level of RC frames is assessed.

Keywords: Concrete Strain; Pushover Analysis; Seismic Performance; Ductility Factor; Overstrength Factor; Interstory Drift Index.

1. Introduction

In current context of increased urbanization, reinforced concrete (RC) structures may be considered as the base of all the structural constructions of civil infrastructures in Bangladesh due to availability of materials and cheap labor. Because of inadequate design, lacks of construction supervision, change in loading pattern, damages and casualties of earthquakes or environmental degradation, existing RC structures may require analysis whether it requires retrofitting or not. The majority of existing international and national design codes such as ACI 2014 [1]; BSI 1997[2]; BNBC 1993[3] is based upon a semi-empirical approach that derives from tests on RC members such as columns and beams. However, this approach does not allow for an easy extension of the design rules to cover changes in building practice and advances in material technology. Design of civil engineering structures is typically based on prescriptive methods of building codes. Normally in the static case, the loads on these structures are low and result in elastic structural behavior. Applying nonlinear dynamic analyses, Haselton et al. [4] investigate the risk of collapse of a set of 30 representative RC special moment-frame (SMF) buildings, designed according to ASCE 7-02 [5] and ACI 318-05 [6] and found on average, the probability of collapse (under ground motion intensities with a 2% probability of exceedance

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in 50 years) of these buildings is 11%. Seismic performance, damage, casualties and risk analysis of old buildings are also assessed by Un et al. [7], Yalciner et al. [8], Han et al. [9], Dya and Oretaa [10] and Mosleh et al. [11] with different methods. Nonlinear time history analysis (NLTHA) is a possible method to calculate structural response under a strong seismic event (e.g. near source earthquake effect [12]). Different damage levels and response parameters specially, rotation, material strains and seismic hysteretic energy of RC frames under numerous ground motions were investigated with NLTHA [13]. Hoffman and Richards [14] used a standard genetic algorithm technique with nonlinear response history analysis. Due to the large amount of data involved in such analysis, it is not considered practical. Modern building codes such as International Building Code (IBC 2006) [15] and Federal Emergency Management Agency (FEMA 356) [16] favor more accurate procedures (as pushover analysis) over traditional linear-elastic methods for a more thorough analysis. The pushover response curves relatively well for lower amplitudes of motion [17]. Nonlinear static pushover (NSP) analysis has much value in understanding important behavior characteristics that are not being explored in a nonlinear response history analysis in which engineers usually focus on a demand/capacity assessment rather than visualization of response [18-19].

A global pushover curve generated through a pushover analysis can be used to assess the seismic performance and overall lateral load behavior of a structure. The present study focuses on numerical assessment of seismic behavior of existing RC frames using nonlinear static pushover analysis. Different concrete ultimate strain conditions are considered for the RC members for evaluating hinge properties and compared with one another using pushover curves. Since mechanical behavior of the structural system is expected to change with different strain condition of the concrete and plastic hinge behavior of reinforced concrete under seismic loading [20], the effect of different plastic hinge properties with different ultimate strain of the concrete into seismic response of the RC frame structure is investigated. Moment-rotation behavior simulates better hinge properties of RC frame members are incorporated in the commercial software ETABS with an aim to assess the overall seismic performance of existing RC structures. It can direct the designer whether the structure is safe for design period and retrofitting is required or not. In summary, the main objectives of the present paper can be stated in general as (i) To analysis existing RC frames with different concrete ultimate strain conditions and (ii) To evaluate the seismic performance of existing RC frames in terms of overstrength, ductility and response modification factor.

2. Methodology: Nonlinear Static Pushover Analysis

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool to assess the performance of a structure. ATC-40 [21] and FEMA-356 [16] documents have developed modeling parameters, acceptance criteria and procedures of pushover analysis and the actions followed to determine the yielding of frame members during the analysis.

2.1. Steps for Nonlinear Static Pushover Analysis

A global pushover curve represents the relationship between a global displacement, normally roof displacement and a global strength parameter such as base shear. In this study, this curve is obtained by subjecting to a specific single lateral load pattern based on the first mode shape of the frame.

For a frame structure, plastic hinges usually form at the ends of beams mostly caused by uniaxial bending moments and columns mostly caused by axial loads and biaxial bending moments under earthquake actions. The moment-rotation behavior of frame members simulates the plastic hinge properties.

2.1.1. Plastic Hinge Assignment

The frame of a building has to be modelled to carry out nonlinear static pushover analysis. This requires the development of the moment-rotation curve for the critical sections of beams and columns. The moment-rotation in flexure is obtained from the reinforcement details and cross sectional properties and is assigned for all the beams and columns of a frame. The nonlinear properties of beams and columns have been evaluated using the section designer and have been assigned to the computer model in ETABS V.9.6.0. The flexural default hinges (M3) are assigned to the beams at two ends. The interacting (P-M2-M3) frame hinges type a coupled hinge property is assigned for all the columns at upper and lower ends of the members.

2.1.2. Lateral Load Distribution along the Height of a Frame

The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure. The distribution of these forces will vary continuously during earthquake response as portions of the structure yield and stiffness characteristics change. The extremes of this distribution will depend on the severity of the earthquake shaking and the degree of nonlinear response of the structure. Use of more than one lateral load pattern is

intended to bind the range of design actions that may occur during actual dynamic response. However, for the simplicity, a single lateral load pattern that simulates first mode shape of the building is used in the present study. In the absence of a more rigorous procedure, the total lateral force, which is the base shear V, shall be distributed along the height of the structure in accordance following Equations. (BNBC-93) [3].

$$V = F_t + \sum_{i=1}^{n} F_i \tag{1}$$

Where;

F_i = Lateral force applied at story level –i

F_t = Concentrated lateral force considered at the top of building in addition to the force and, F_n = The concentrated force,

Ft acting at the top of the building shall be determined as follows:

F_t = 0.07 TV ≤ 0.25 V, when T > 0.7 second F_t = 0.0 when T ≤ 0.7 second

The remaining portion of the base shear (V-F_t) shall be distributed over the height of the building, including level-n, according to Equation (2):

$$F_x = \frac{(V - F_t)W_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{2}$$

At each story level-x, the force F_x shall be applied over the area of the building in proportion to the mass distribution at that level.

Where;

V = Base shear force h_x= Height at the x-level of the frame

F_x = Lateral force applied on the x-level of the frame h_n = Total height of the frame

n = Number of stories W_i= Weight of the story- i

W_x = Seismic weight of the building at the stories of building

2.1.3. Geometric Nonlinear Effects

Geometric nonlinear effects are caused by gravity loads acting on the deformed configuration of the structure, leading to an increase of internal forces in members and connections. These geometric nonlinear effects are typically distinguished between P-Δ effects, associated with deformations along the members, measured relative to the member chord, and P-Δ effects, measured between member ends and commonly associated with story drifts in buildings.

2.2. Moment-Rotation Behavior of RC Frame Members

2.2.1. Default Moment-Rotation Curves for RC Beams

Plastic hinges assigned at the end of beams characterize moment-rotation (M–θ) behavior of beams. This behavior for a member section consists of plastic rotation and corresponding moments as ratio of yield moment as shown in Figure1. This relation affects the behavior of a section once a hinge forms there. The parameters a and b for default hinges as shown in Figure1 are obtained from Table 6-7 of FEMA 356 guidelines. Static nonlinear pushover analysis of the considered frames is based on the procedures of the material, defining force–deformation criteria for the hinges properties shown in Figure 1. This figure describes the typical force-deformation relationship of any member considered in FEMA guidelines. Five points labelled A, B, C, D and E are used to define the force deflection behavior of the hinge and these points labelled A to B – elastic state, B to C- post yielding stage, C to D – post maximum capacity, D to E- residual strength. This typical hinge properties is also used by Golghate et al. [22] in their study.

2.2.2. User Defined Moment-Rotation Curves for RC Beams

For beams, yield and ultimate conditions can be determined from analytical solutions. Moment hinge properties, M3

is usually added or modified for concerned beam (actual hinge property). Individual nonlinear hinge is assigned at both ends of the individual beam according to the cross-section and material properties of the fame element.

By using the rectangular stress block of ACI 318, flexural strength may be calculated with the condition that maximum concrete strains ranging from 0.003 to 0.005 and maximum stress in concrete compression zone taken equal to 85% of the expected compression strength. Strain of concrete, εcu is considered 0.003, 0.004, and 0.005, respectively, to compare obtained hinge properties.

While doing the nonlinear static analysis, it is required to determine the moment capacity and curvature to establish moment-rotation relationship in accordance with FEMA 356. From Park and Dai [23], the curvature at yield load, φy for a doubly reinforced concrete beams as shown in Figure 2 can be written as:

$$\varphi_y = \frac{f_y}{E_s d(1 - K)}$$

(3)

Where;

f_y= Yield strength of steel E_s = Modulus of elasticity for steel

d = Effective depth of the member section   K = Depth factor and another parameter

$X_y = dK$
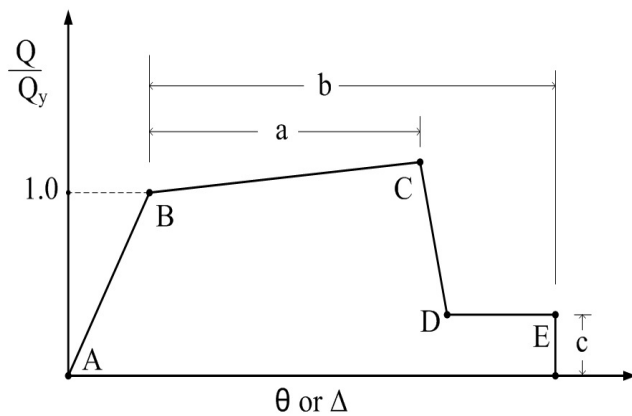
Figure 1. Force-deformation behavior of frame members   **Figure 2. Strain diagram at yield and ultimate load conditions**
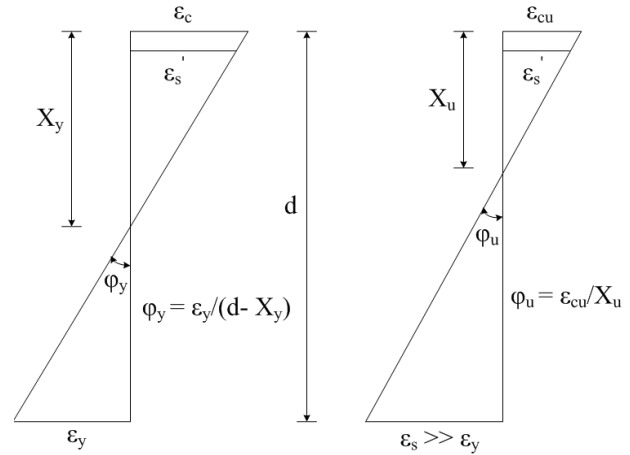
If top reinforcement is not yield in compression (εs'< εy) then, the depth factor, K can be obtained from the Equation (4):

$$K = \left[0.5 + \frac{f_y(\rho' + \rho)}{2\alpha f_c'}\right] - \sqrt{\left[0.5 + \frac{f_y(\rho' + \rho)}{2\alpha f_c'}\right]^2 - \left[\frac{f_y\left(\rho'\frac{d'}{d} + \rho\right)}{\alpha f_c'}\right]}$$

(4)

Where:

ρ'= Compression steel ratio f_c= Concrete strength

ρ = Tension steel ratio   d' = Clear cover at compression face

If top reinforcement is yield in compression (εs' > εy), then length factor, K can be evaluated from Equation (5):

$$K = \frac{f_y(\rho - \rho')}{\alpha f_c'}$$

(5)

Strain of compression steel:

$$\varepsilon_s' = \left(\frac{kd - d'}{d - kd}\right)\frac{f_y}{E_s} \tag{6-a}$$

Strain at yield condition:

$$\varepsilon_y = \frac{f_y}{E_s} \tag{6-b}$$

If compression reinforcement is not yield, ($\varepsilon s' < \varepsilon y$), then ultimate curvature, φu [24] at ultimate load can be determined as Equation (7):

$$\varphi_u = \frac{\varepsilon_{cu}}{X_u} \tag{7}$$

Distance from neutral axis to the compression face of the cross-section of the member (Xu) [24] is measured by using Equation (8) given below:

$$X_u = \left[\frac{(\rho'E_s\varepsilon_{cu} - \rho f_y)^2 d^2}{(2\alpha f_c')^2\beta_1^2} + \frac{\rho'E_s\varepsilon_{cu}dd'}{(\alpha f_c')\beta_1}\right]^{1/2} - \frac{(\rho'E_s\varepsilon_{cu} - \rho f_y)d}{(2\alpha f_c')\beta_1} \tag{8}$$

If compression reinforcement is yield, ($\varepsilon s' > \varepsilon y$), Curvature at ultimate load can be determined by Equation (9):

$$\varphi_u = \varepsilon_{cu}\frac{\alpha\beta_1 f_c'b}{(A_s - A_s')f_y} \tag{9}$$

Rotation capacities at yield condition, θy and at ultimate load, θu can be calculated from curvature of a section; three criteria are considered in the present study:

i) From Equations. (10-a) and (10-b):

$$\theta_y = \varphi_y.\frac{L}{3} \tag{10-a}$$

$$\theta_u = \theta_y + \left(\varphi_u - \varphi_y\right)\frac{L_p.(L - 0.5.L_p)}{L} \tag{10-b}$$

Where;

L= shear span of the member (assumed half of the clear span),

$L_p = 0.08L + 0.15 f_y d_b \geq 0.3 f_y d_b$ (f_y in ksi and d_b in inch),

$L_p = 0.08L + 0.022 f_y d_b \geq 0.044 f_y d_b$ (f_y in MPa and d_b in mm) [25],

d_b= average diameter of the reinforcement.

ii) Using the half of the section depth, H as shown in Equations. (11-a) and (11-b).

$$\theta_y = \varphi_y * 0.5H \tag{11-a}$$

$$\theta_u = \varphi_u * 0.5H \tag{11-b}$$

iii) From the plastic hinge length, Lp, using Eqs. (12-a) and (12-b) [25]:

$$\theta_y = \varphi_y * L_p \tag{12-a}$$

$$\theta_u = \varphi_u * L_p \tag{12-b}$$

User-defined hinge property requires moment-curvature analysis of each frame element. The sectional analysis is used to calculate the moment capacities at yield and ultimate conditions. The curvature and rotation capacities of beam member are determined following the Equations from (3) to (12) described above.

2.2.3. Default Moment-Rotation Curves for RC Columns

Plastic hinges assigned at the end of columns also are characterized by moment-rotation (M-θ) relationships of column sections and P-M interaction surfaces. This moment-rotation behavior is the same as beam sections as shown in Figure 1. The parameters a and b for default hinges as shown in this figure are obtained from the Table 6-8 of FEMA 356 guidelines.

P-M interaction surface determines the load at which a reinforced concrete section of the column becomes inelastic

and forms a hinge. For a given section geometry, material and reinforcement, P-M interaction surface is calculated using ETABS v.9.6.0 section designer module according to ACI code (2014). The stress-strain curve for concrete suggested by Kent and Park [26] and stored in ETABS v.9.6.0 software is used to complete P-M interaction curves for the column sections in the frame.

2.2.4. User Defined Moment-Rotation Curves for RC Columns

For columns, yield and ultimate curvature and rotation capacities are determined following the subsection 2.2.2. Moment-rotation behavior as well as P-M-M interaction surface is considered for the user defined hinge properties of columns. Nonlinear hinge is assigned at both ends of the individual column according to the cross-section and material properties of the fame element.

By following the rectangular stress block of ACI 318, flexural strength may be calculated with the condition that maximum concrete strains ranging from 0.003 to 0.005 and maximum stress in concrete compression zone taken equal to 85% of the expected compression strength. Ultimate strain of concrete, ε_{cu}, is considered 0.003, 0.004, 0.005, respectively to compare the obtained hinge properties.

2.3. Seismic Performance Criteria

For the evaluation of seismic performance level of a reinforced concrete frame, following criteria are used:

2.3.1. Interstory Drift Index

The interstory drift index can be described as the lateral deflection of a floor level relative to the one immediately below it is divided by the distance between the floors of the frame. The recommendations in FEMA 356 specify interstory drift index thresholds of 1%, 2% and 4% for the immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance levels, respectively. The maximum interstory drift ratio of a frame is estimated from the pushover analysis of the frame when it reaches its maximum capacity.

2.3.2. Response Modification Factor

The seismic force reduction factor or a response modification factor is used to reduce the elastic strength demand due to an earthquake event. This factor is needed to account for overstrength and ductility inherent in the structural system considering FEMA 440 [27]. The overstrength factor, Ω_d, is defined as the ratio of the actual lateral strength (V_y) to the design lateral strength (V_d) as shown in Figure 3.

$$\Omega_d = \frac{V_y}{V_d} \tag{13}$$

Ductility factor, μ, can be represented by Equation (12):

$$\mu = \frac{\Delta_m}{\Delta_y} \tag{14}$$

The response modification factor, R can be expressed in Equation (15):

$$R = \Omega_d \mu \tag{15}$$
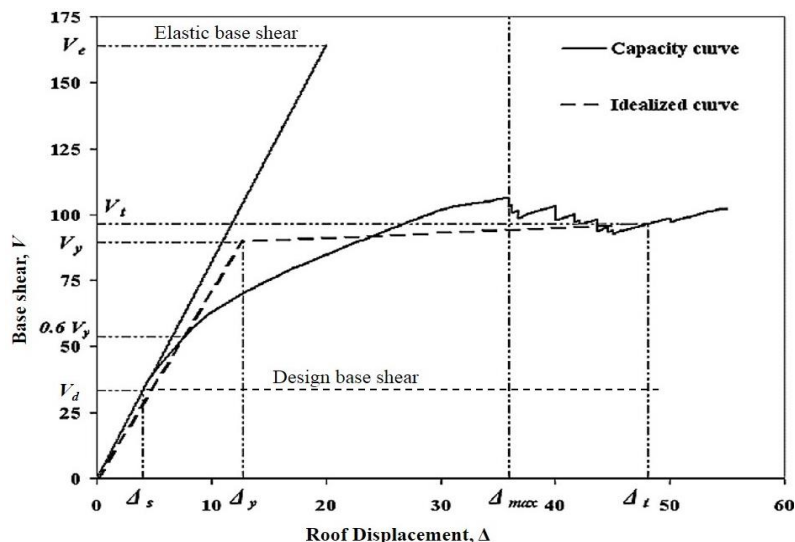
Figure 3. Description of overstrength and ductility factor for a structural response [27]

3. Pushover Analysis of a RC Frame

3.1. Bare Frame Considered in this Study

A typical frame of a 6 storied building located at Mirpur residential area in Dhaka city is considered for pushover analysis. Details of the floor plan and cross-sectional descriptions are provided in Figure 4. Material Properties of the structural elements (beam, columns) considered in the study are presented in the Table 1.

Table 1. Material properties of the structural elements
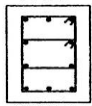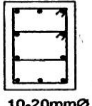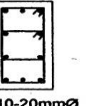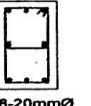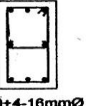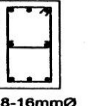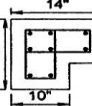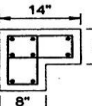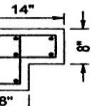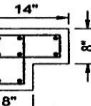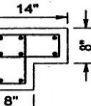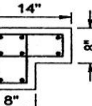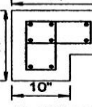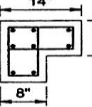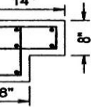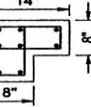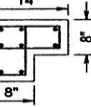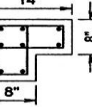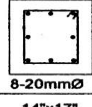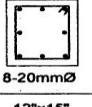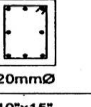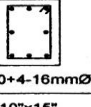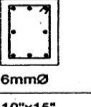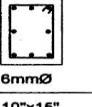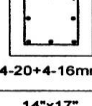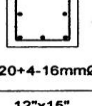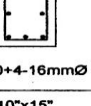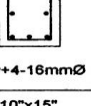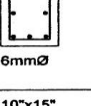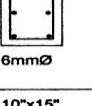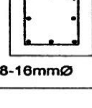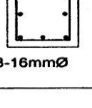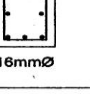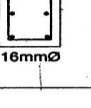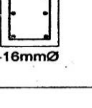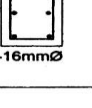
	Structural Element	Material Properties
Concrete	Columns	Compressive Strength= 24.14 MPa (3.5 ksi)
	Beams	Compressive Strength= 17.24 MPa (2.5 ksi)
Steel	Yield Strength= 413.8 MPa (60 ksi) (both transverse and longitudinal reinforcements)	

The plan of the building is divided into 5 frames- Frame A, B, C, D and E, respectively. Total height of the building is 20.42 m (67 ft), typical floor to floor height of the building is 3.05 m (10 ft) and base to ground floor is 2.13 m (7 ft). In plan, each frame consists of four columns and the span lengths of the beams 5.08 m (200 in), 5.41 m (213 in) and 4.67 m (184 in). In the present study, a two-dimensional frame E as shown in Figure 4 is considered as a representative frame for pushover analysis.

COLUMN LAYOUT PLAN AT GROUND FLOOR
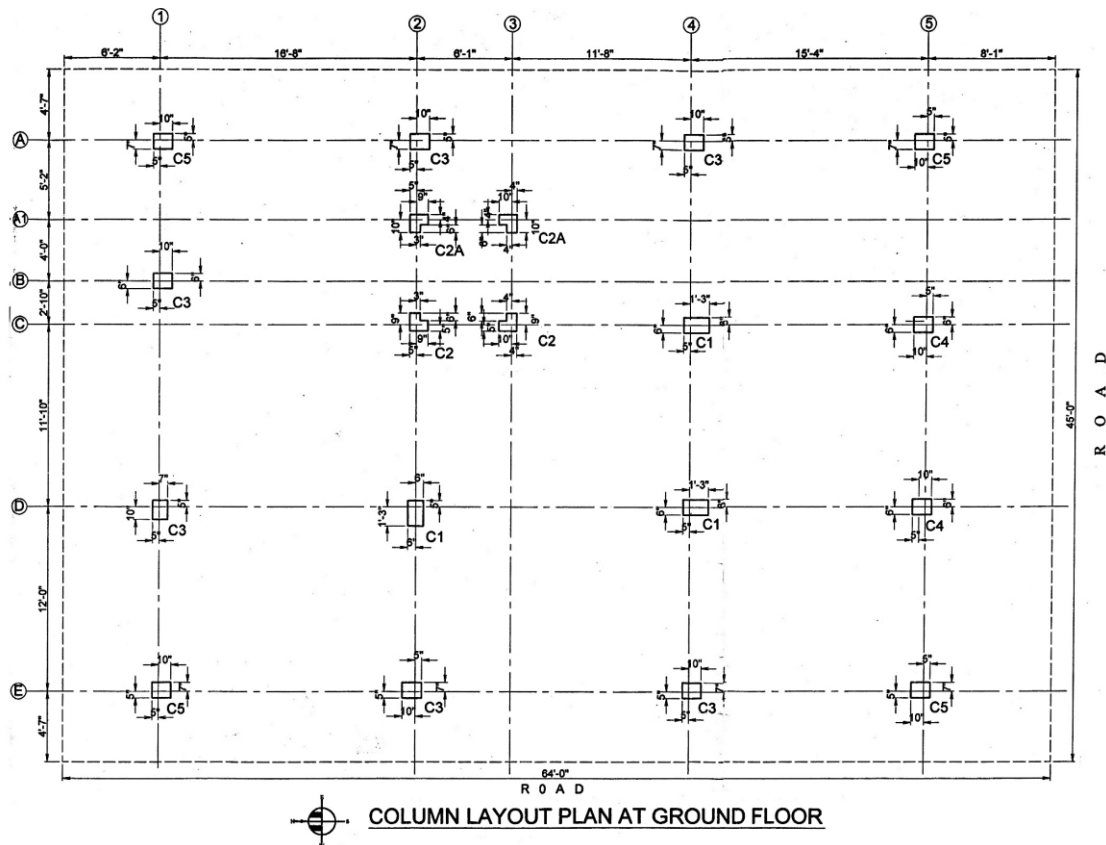
(a) Column layout plan at ground floor

COLUMN SCHEDULE

COL. MARK	BELOW GB	GR. FLOOR	1ST FLOOR	2ND FLOOR	3RD FLOOR	4TH FLOOR & 5TH FLOOR
C-1	14"x22" / 10-20mmØ	12"x20" / 10-20mmØ	10"x20" / 10-20mmØ	10"x20" / 8-20mmØ	10"x20" / 4-20+4-16mmØ	10"x20" / 8-16mmØ
C-2	8-20mmØ	8-20mmØ	8-20mmØ	8-20mmØ	4-20mmØ 4-16mmØ	4-20mmØ 4-16mmØ
C-2A	8-16mmØ	8-16mmØ	8-16mmØ	8-16mmØ	8-16mmØ	8-16mmØ
C-3	14"x17" / 8-20mmØ	12"x15" / 8-20mmØ	10"x15" / 8-20mmØ	10"x15" / 4-20+4-16mmØ	10"x15" / 8-16mmØ	10"x15" / 8-16mmØ
C-4	14"x17" / 4-20+4-16mmØ	12"x15" / 4-20+4-16mmØ	10"x15" / 4-20+4-16mmØ	10"x15" / 4-20+4-16mmØ	10"x15" / 8-16mmØ	10"x15" / 6-16mmØ
C-5	14"x17" / 8-16mmØ	12"x15" / 8-16mmØ	10"x15" / 8-16mmØ	10"x15" / 6-16mmØ	10"x15" / 6-16mmØ	10"x15" / 6-16mmØ

(b) Column Scheduling

(c) Long section of floor beam, FB1

Figure 4. Details of the existing building

3.2. Moment-Rotation Behavior of the Frame Member

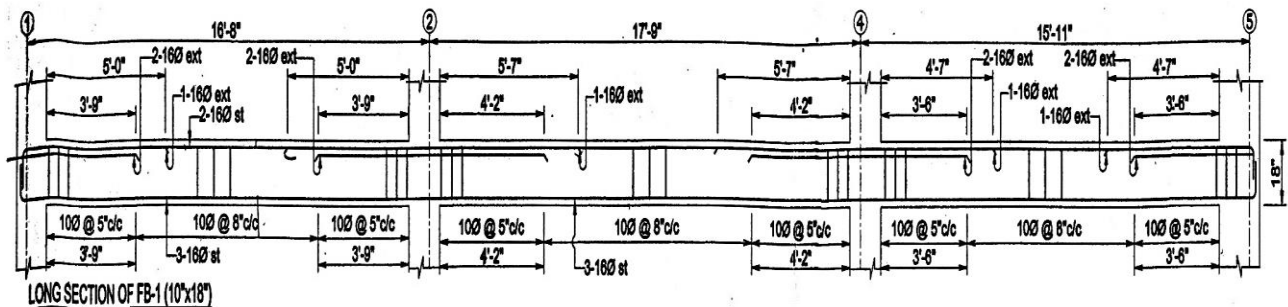Plastic hinges are assigned on both ends of columns and beams of a frame. Materials properties for each member are considered. Both the default hinge properties available in software ETABS and user-defined hinge properties obtained from moment-curvature analysis using sectional properties of beams and columns are used in the pushover analyses.

Default hinges are assigned to the beam and column members in first phase of analysis. In ETABS, default moment hinges M3, nonlinear hinge is rigid plastic and moment-rotation relation is symmetric in nature. A generalized moment-curvature characteristics of nonlinear hinge of beams in ETABS is shown in Figure 5. Point A denotes unloading condition and point B represents yielding condition of the beam. Ultimate condition is symbolized by point C. Abrupt change in moment is considered from C to D without any change of rotation. From D to E, plastic hinge rotation increases and the moment is considered remain the constant. Figure 5 shows moment capacity as ratio of yield moment and

rotation as (rad) for a section of a frame. In ETABS, default hinges for columns such as P-M-M and P-M2-M3 interaction for axial and biaxial moment are defined. A generalized moment-curvature characteristics of nonlinear hinges for columns in ETABS is similar to the default hinge property of the beam.

Axial load-displacement relationship is considered proportional to moment-rotation behavior. Interaction diagram for the column is considered doubly symmetric about moments M2 and M3. To define axial load-bending moment (P-M) interaction surface, five curves are assigned herein from biaxial moments M2 to M3 for different angle (angle 0, 22.5, 45, 67.5, 90 respectively). The P-M interaction surface at 0 degree defines the moment capacity of a section about minor axis. Whereas, P-M interaction surface at 90 degree represents the moment capacity of the section about major axis.
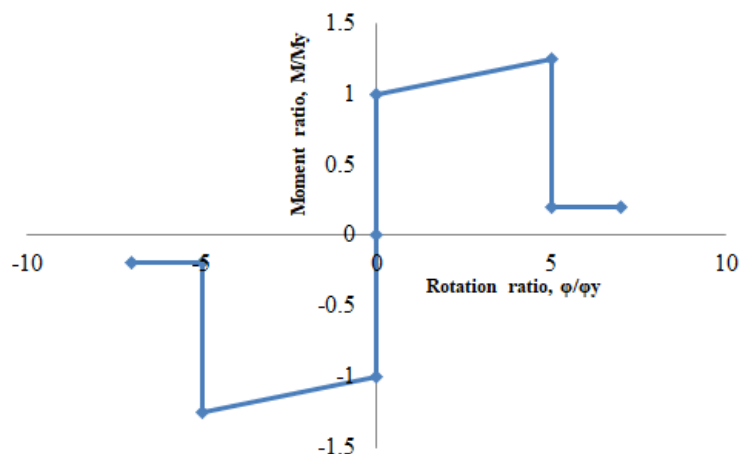

Figure 5. Typical moment-rotation curve for default hinge moment M3

In second phase of analyses, default hinges for columns and user defined hinges for beams are considered. Moment-rotation behavior of each RC beam is determined. The maximum moment capacity can be determined considering post-yield slope of 3% that of the slope of the elastic curve. The ultimate concrete strain is considered from 0.003 to 0.005. The sectional properties of beam sections and material properties are kept the same.

In the third phase of analyses, user defined hinges both for the beams and columns are considered in the analyses. Moment-rotation behavior for hinges of column member is obtained following the subsection 2.2.4. The sectional details of columns as well as material properties are the same as mentioned in section 3.1 and is provided in Figure 4(b).

3.3. Lateral Force Distribution along the Height of the Frame

The distribution of lateral forces along the height of the frame is obtained following BNBC-93. Parameters used: Numerical coefficient, $C_t = 0.073$ (Concrete structures), Height in meters above the base to level n, $h_n=20.42$ m (67 ft), Site coefficient for soil characteristics, S = 1.5, Response modification coefficient for structural systems, R=8, Seismic zone coefficient, Z = 0.15 (Dhaka), Area/floor level= 165.6 m^2 (1782.71 sft). Self-weight of the different members, floor finish, and partition walls at different floor levels are considered as dead loads for determining the total dead weight at any floor level. In calculating total seismic weight at any floor, 25% live loads (1.92 kPa) (40 psf) in addition to dead weight are used.

For the considered frame E, following parameters are used to calculate the base shear force in accordance with BNBC-93: Fundamental period of vibration of the structure in the direction under consideration, T=0.701 second, Numerical coefficient, C=2.33, C/R= 0.297> 0.075 (ok)

Design Shear, V=531.75 kN (119.5 kip). That portion of the base shear V, considered concentrated at the top of the structure in addition to F_n, F_t = 26.08 kN (5.86 kip) < 0.25V (ok)
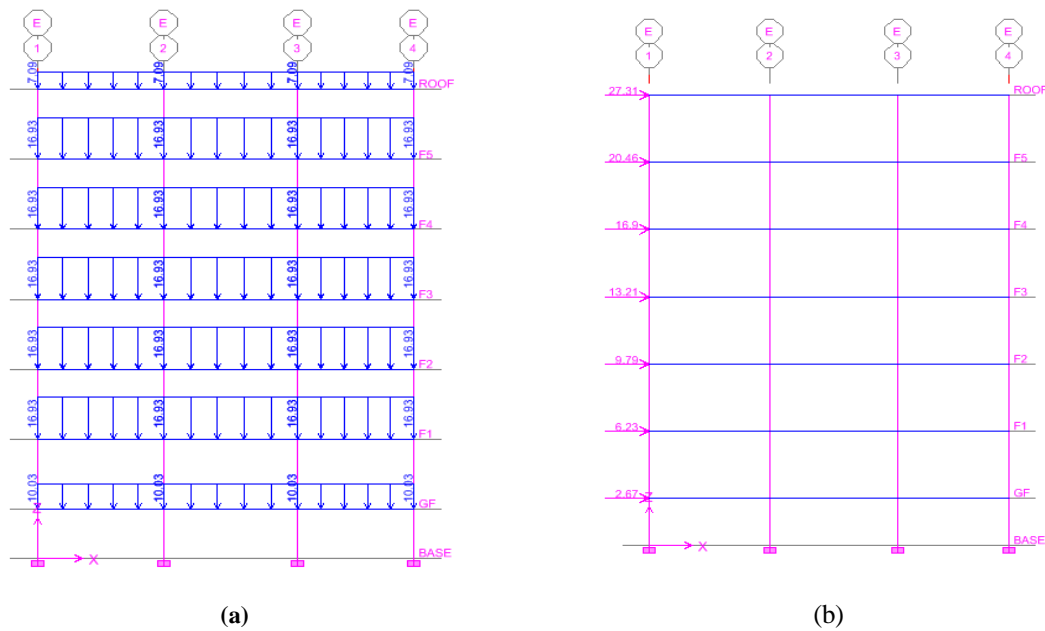
Figure 6. Load distribution on Frame E ; (a) Gravity loads (kN/m) ; (b) Lateral load distribution (kN)

In pushover analyses, hinges at the ends of different frame members are assigned and gravity loads equivalent to seismic weight are applied first on the beams of the frame. Then, the lateral force distribution on different floor level is applied gradually until the failure of the frame is occurred. For the considered frame E, gravity loads applied are used 7.09 kN/m (0.49 k/ft) on roof, 16.93 kN/m (1.16 k/ft) on different floors except the ground floor, and 10.03 kN/m (0.69 k/ft) on the ground floor. Figure 6 shows the assigned gravity loads and lateral force distribution along the height of the considered frame.

4. Results and Discussions

4.1. Comparison of Pushover Curves Obtained from Default and User-Defined Hinges

In the first phase of pushover analyses, comparison is made considering user defined hinge properties for beam elements and default hinges for columns. To determine the plastic hinge properties for beams, ultimate concrete strain conditions are considered as 0.003, 0.004, and 0.005, respectively. Figures 7(a), 7(b), and 7(c) show the base shear vs. roof displacement relationship for frames considering different ultimate concrete strains. Plastic rotation capacities at yield and ultimate conditions for user defined hinges of beam sections are determined considering the plastic hinge length following Equation (10), plastic hinge length 0.5H (H= section depth) and Equation (12), respectively.

From these curves, it is observed that user-defined beam hinges with ultimate concrete strain 0.005 provides good agreement with that of the default hinge case. It is also shown that, rotation capacities obtained using Equation (10) provides better agreement than that of the obtained results using plastic hinge length of 0.5H.

In the second phase of analyses, pushover curves of the frame with user-defined hinges for both the beam and column with different ultimate strain conditions (ε_{cu} = 0.003, ε_{cu} = 0.004, and ε_{cu} = 0.005) are presented in Figure 8 and is compared with that of the default hinge case. The ultimate moment capacity, Mu for Figures 8(a) and 8(b) is assumed to be post-yield slope of the curve as 3% and 5% of the elastic curve, respectively. For plastic hinge length, Equation (10) is used to calculate the rotation capacities from the curvatures of sections. From both figures, it is shown that ultimate concrete strain of 0.005 provides good agreement with the case of default hinges.

(a)   **(b)**

(c)

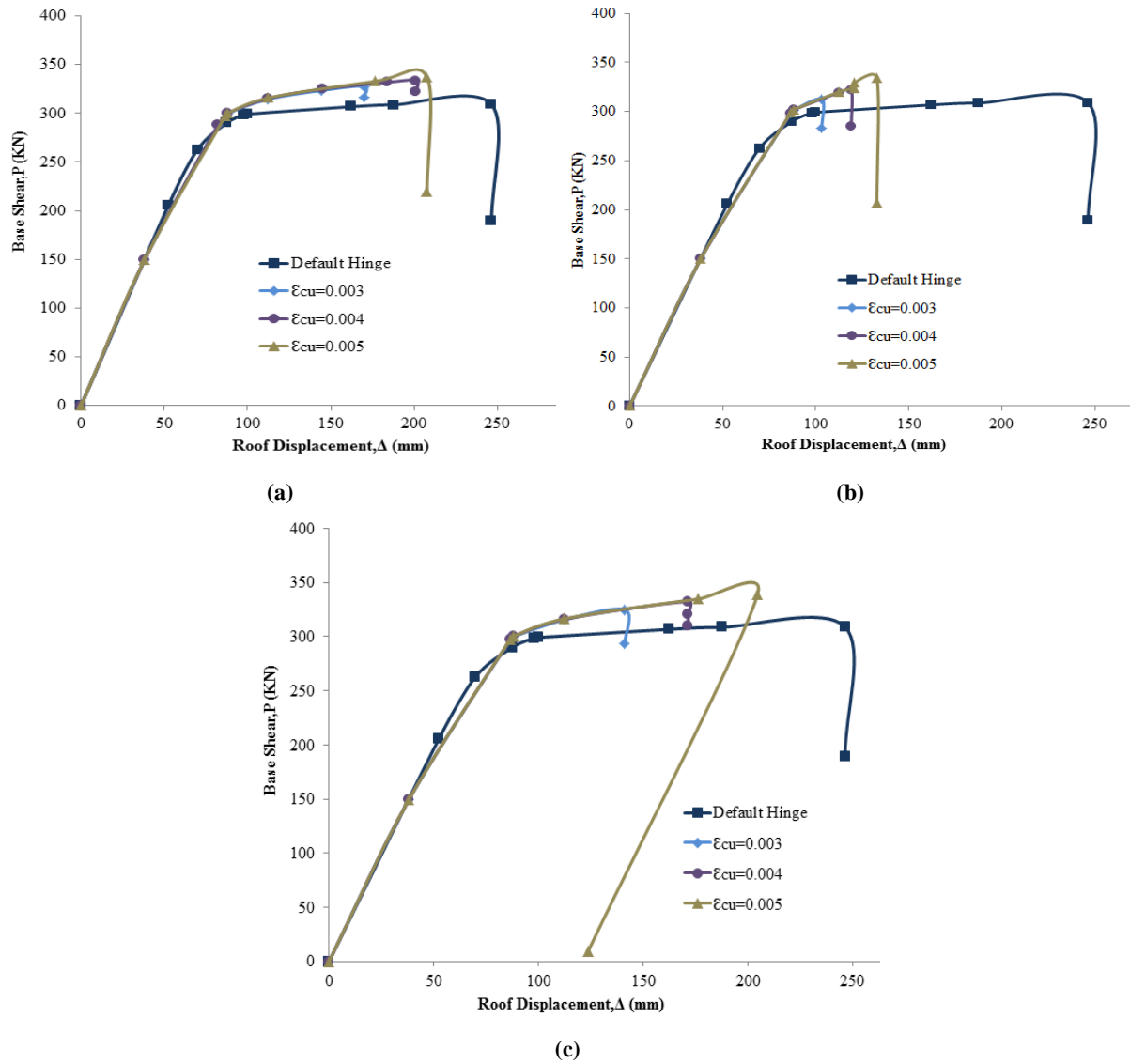**Figure 7. Pushover curves of the frame (a) for beam hinges using Equation 10; (b) considering plastic hinge length 0.5H; (c) using plastic hinge length of Equation 12 at different concrete ultimate strains**
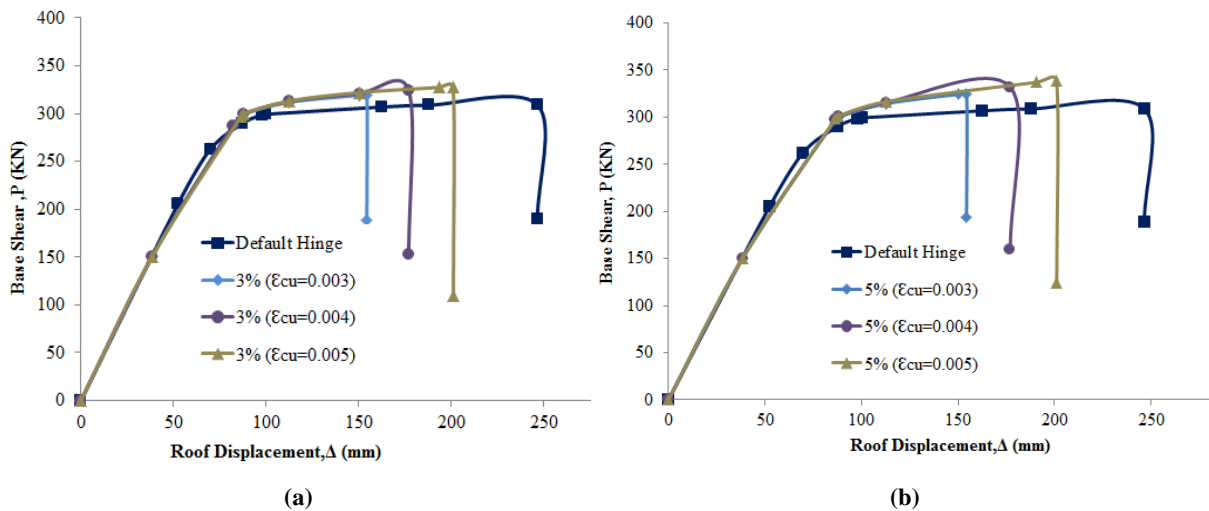
(a)   **(b)**

Figure 8. Pushover curves of bare frames using Equation 10 for different ultimate concrete strains (a) considering 3% post-yield stiffness; (b) considering 5% post-yield stiffness

Comparison of pushover curves with ε_{cu}= 0.005 and different plastic hinge lengths are also studied. Plastic hinge lengths as mentioned in Equations. (10), (11), and (12) are used to determine the plastic rotation capacities. The ultimate moment capacities of frame sections are assumed considering post-yield stiffness 3% that of the elastic slope and pushover curves of the frames are presented in Figure 9(a) for using different plastic hinge lengths.
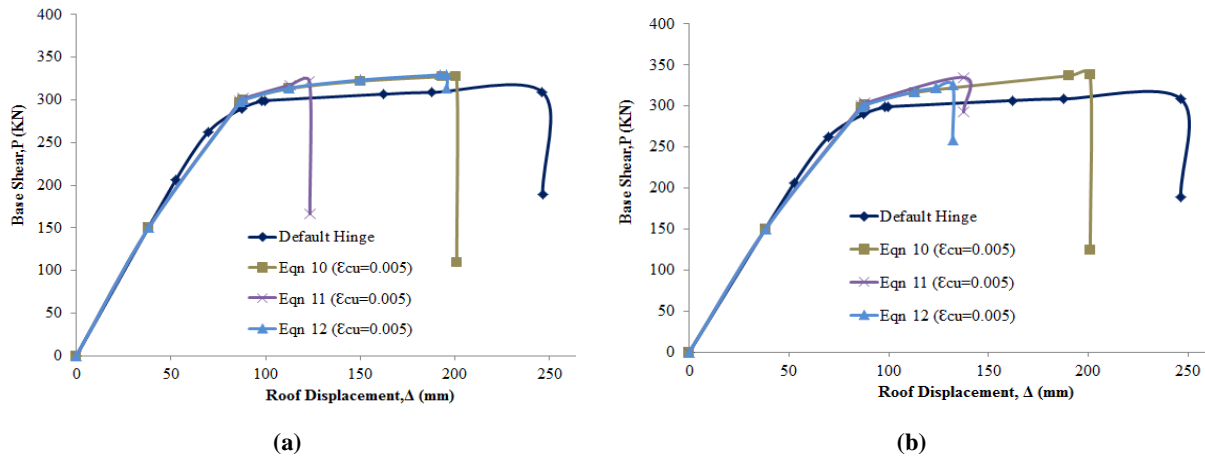

| (a) | (b) |

Figure 9. Pushover curves for frames (a) 3% post-yield stiffness; (b) 5% post-yield stiffness

Comparison is also made for the ultimate moment capacity considering 5% post-yield stiffness and is presented in Figure 9(b). From both figures, it is observed that plastic hinge length obtained from Equation (10) provides better agreement with that of the default case.

From the analyses, it is shown that pushover curves with 3% post-yield stiffness and ultimate concrete strain of 0.005 provides better agreement with that of the default hinge case. Therefore, it is considered as base pushover curve for the frame having user defined hinges for both the beams and columns. This pushover curve is used to evaluate the seismic performance parameters for the RC frame considered in this study.

4.2. Deflected Shape at Maximum Load Carrying Capacity of the Frame

In the case of analysis with user-defined hinges for both the columns and beams, plastic hinge formation starts with beam ends at grade beam (GB) and then column at below GB. Formation of hinges then gradually propagates to upper stories and continues with yielding at the base of columns. These columns experience major damage or collapse at maximum load carrying condition.

The numbers of hinges formed in the RC frame at a maximum lateral load are shown in the Figure 10. In each member of the frame, the immediate occupancy, life safety, collapse prevention and some limited stages are shown to define the force-deflection behavior of the hinges. From the figure, it is observed that base of the columns is subjected to severe damage and frame elements of the upper storied are within the elastic limits. Among 98 hinges, interior columns which are located below the grade beam of the symmetric frame undergoes to collapse. Exterior columns of the frame having the same location (below GB) are also at risk of life safety and collapse prevention, respectively. Five hinges of the beam undergoes immediate occupancy and other 22 hinges are at the stage of below immediate occupancy but above the elastic state.

5. Seismic Performance of RC Frames

To evaluate the seismic performance parameters of the RC frame, pushover curve obtained from analysis is idealized following the guidelines of FEMA 273 [28]. The yield capacity of the frame, V_y, is evaluated by idealizing pushover curve as a bilinear curve considering the same energy under the actual curve. The yield displacement, Δy, of the frame is determined at a yield capacity, V_y, in accordance with the guidelines FEMA 273. Figure 11 shows the idealized and actual pushover curves for the bare frame.
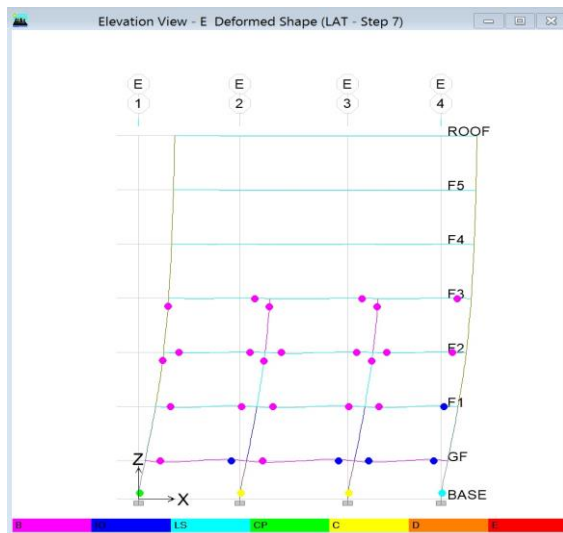
Figure 10. Hinge formation on different frame elements of the RC frame at maximum load carrying condition
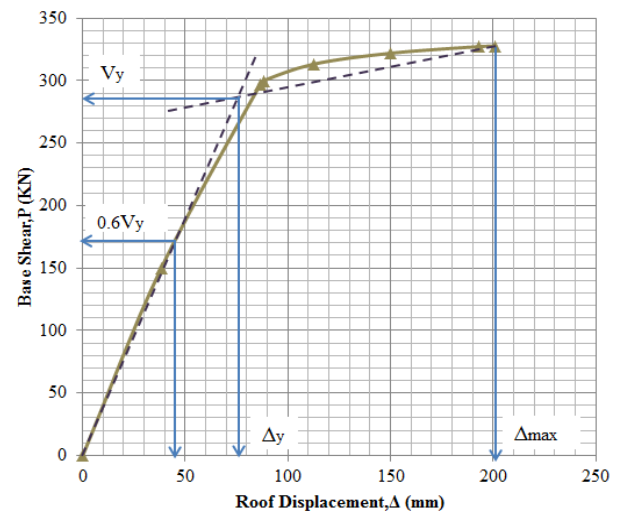
Figure 11. Pushover curves of the bare frame to evaluate the seismic performance parameters

Design base shear following BNBC-93 is determined as 132.96 kN (29.88 kip) for this frame. From the pushover curve, the yield shear capacity is determined 286 KN and the yield roof displacement is 76 mm. The maximum displacement is 201 mm. This results ductility ratio 2.64 and overstrength factor 2.15 for the RC frame. The response modification factor, R is determined to be 5.69 from the analysis.

To evaluate the interstoy drift index at different floor levels of the frame, lateral deflection at floors is considered at the maximum load carrying capacity of the frame. Interstory drift index is determined considering the relative floor displacement divided by the floor height. Figure 12shows the interstory drift index of the bare frame in comparison to acceptable performance levels. From this figure, it is observed that interstory drift index of the bare frame at maximum lateral load capacity are within the Life safety performance level (2%).
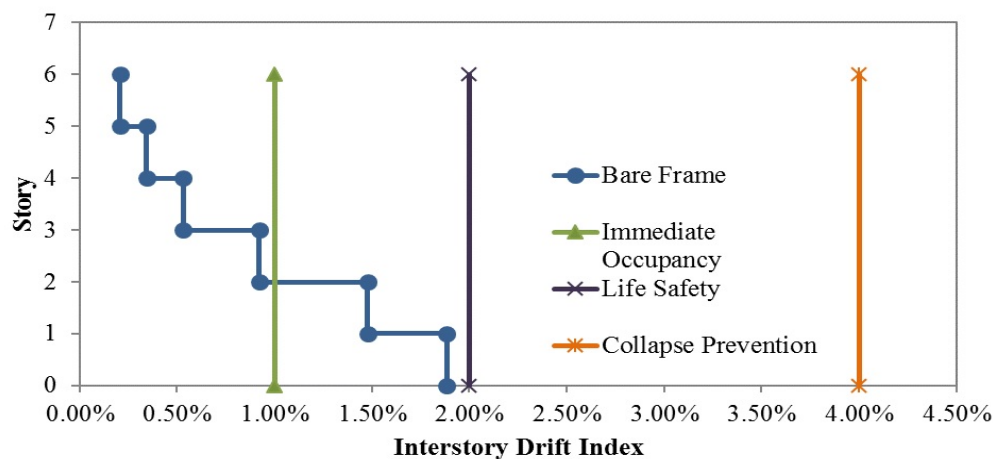
Figure 12. Interstory drift index of the RC frame

6. Conclusion

Pushover analyses are conducted on the RC frame for different plastic hinge lengths assigned at the end of beams and columns of the frame. Different ultimate concrete strains are also considered to determine the hinge properties for both the columns and beams of the frame. Comparison is made for pushover curves using different plastic hinge lengths and ultimate concrete compressive strains. The plastic hinge length and ultimate concrete strain which provide good agreement with the pushover curve using the default hinges available in commercial software ETABS v.9.6.0 are considered to assess the seismic performance of RC frames. Following conclusions can be drawn from this part of analyses:

- Ultimate concrete strain, $\varepsilon_{cu} = 0.005$ provides good agreement for the hinge properties simulating moment-rotation behavior in comparison to that of the default hinges built-in commercial software ETABS.

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- Post-yield stiffness of the section at a 3% of that of the elastic curve provides better agreement than that of the 5% post-yield stiffness of the section.

- Plastic hinge length considering Equation (10) results good agreement with the default hinge properties used in the present study.

Seismic behavior of RC frames is studied and parameter parameters such as ductility, overstrength and response modification factor are also assessed by pushover analysis.

- Response modification factor obtained from pushover analysis is less than that of the user defined factor while using equivalent static force method (R = 5.69 < 8)

- Interstory drift index at maximum lateral load capacity are within the Life safety performance level for the considered frame.

7. Conflict of Interests

The authors declare that they have no conflict of interests.

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